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DECEMBER 2016 issue 92

NZ GEOMECHANICS NEWS

Bulletin of the New Zealand Geotechnical Society Inc.

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SPECIAL FEATURE

CURRENT RESEARCH

TOE BUCKLING CENTRAL OTAGO SCHIST

SEISMIC DESIGN OF CUT SLOPES

2016 ACENZ AWARDS

15TH GEOMECHANICS LECTURE PART 3

PHOTO COMPETITION 2016

For over more than 40 years, Grouting Services has delivered some of New Zealand's most significant Ground Anchoring, Soil Nailing, Micro-Piling and Post-Tensioning contracts.

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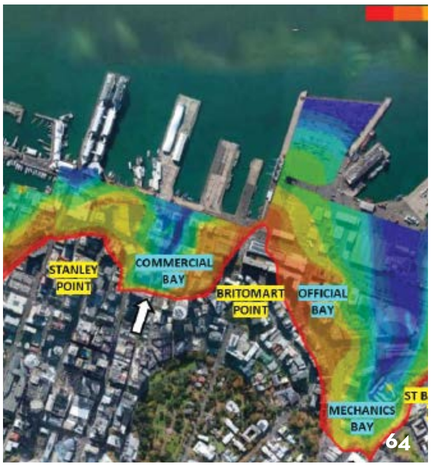
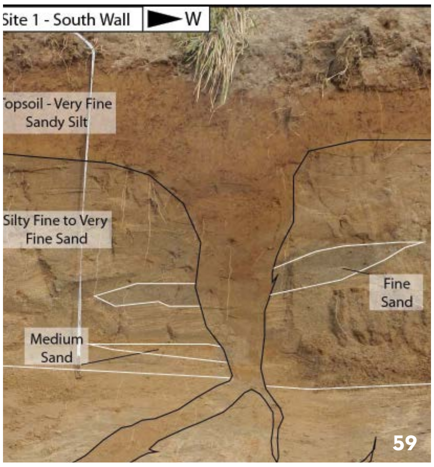




COVER IMAGE: Cliffon Hill, Sumner, Christchurch. *Image courtesy Abseil Access in Christchurch*

SPECIAL FEATURE

Charlie is the Chief Geotechnical Engineer at MWH in Christchurch. Educated as a civil engineer in Dublin and an Engineering Geologist at Imperial College in London, he has worked on dam and tunnel projects in Africa, oil and gas projects in the North Sea, hydroelectric power stations in Pakistan and the UK. He moved to New Zealand in 2003 to work on Project Aqua, and spent seven years working with URS in their Christchurch office before moving to MWH in 2011.

Charlie Price
Chair, Management Committee

AS I WRITE this we seem to be experiencing déjà vu with another series of significant earthquakes. For some it will seem like reliving the Canterbury Earthquake Sequence, only under the guise of another name: the 'Kaikoura Sequence' perhaps? Whatever we name them they are a stark reminder (if we needed it) that we are living with the ever present threat of earthquakes, and that understanding them and how to engineer for them is very much part of what we do. Indeed for some of us it is virtually all we do. So it is fitting that one of the main efforts of the society at present, in conjunction with MBIE, is the development of the Earthquake Geotechnical Engineering Guidelines, of which two modules have just been released in November. Here is a 'progress report' on the development of the series.

The documents are all published as drafts for comment. It would be good to feel that there was significant input from a wide range of users, however very few comments have been received to date, which is a disappointment, and we are now making an appeal for feedback on all those modules released so far.

The updated Seismic Assessment of Existing Buildings has now been published with the new geotechnical section, C4, for which NZGS members had substantial input over an extended period: This document will become mandatory reading for anyone carrying out seismic assessment of existing buildings, so I would encourage you to become familiar with it.

Geotechnical engineering education programme

As mentioned in the last edition of Geomechanics News, we are in the process of developing an education programme with MBIE and IPENZ covering both the Earthquake Geotechnical Engineering Guidelines and other geotech topics. The programme was launched on Sept 1st through a new MBIE website page: https://www.building.govt.nz/building-code-compliance/geotechnical-education. Details will be added to the NZGS website in due course. The website includes all training information available on the earthquake geotechnical engineering guidelines issued to date, Modules 1, 2, 3, 4 and 5A, including some introductory webinars.

The first training under this programme, dealing with Module 3 (liquefaction assessment), was held in September, presented by Misko Cubrinovski and Kevin McManus. This was well received and we are considering running it again early next year. A webinar is also being developed aimed at a more advanced level than the seminars, and should be available within the next couple of months. Webinars addressing modules 1 and 5A are available for viewing, and can be linked to from recently issued NZGS weekly emails. These are valuable and I would encourage you to register and run through them. Further training is being developed for modules 2 and 4.

AN UPDATE ON THE DEVELOPMENT OF THE EARTHQUAKE GEOTECHNICAL ENGINEERING GUIDELINES

Module 1	Overview of the Guidelines Series	Released February 2016
Module 2	Geotechnical investigation for earthquake engineering	Released November 2016
Module 3	Identification, assessment and mitigation of liquefaction hazards	Released July 2010 as 'Module 1' Updated May 2016
Module 4	Earthquake resistant foundation design	Released November 2016
Module 5	Ground Improvement of soils prone to liquefaction	Document under review
Module 5A	Specification of ground improvement for residential properties in the Canterbury region	Released November 2015
Module 6	Seismic design of retaining walls	Development to commence in 2017, expected release in 2017
Module 7	Earthquake slope stability	Development to commence in 2017

Feedback is requested to modulefeedback@nzgs.org

THE WEBSITE

I trust that members will have noticed that we have a new website. Ross Roberts is now our website manager, and has been co-opted back on to the Management Committee (he was previously co-opted as Geomechanics News editor). This site offers much improved facilities on our old site, such as a shared calendar, which we hope will assist members.

YGP 2016 CONFERENCE

The 2016 YGP conference was held in Queenstown in October, attracting a record number of 50 delegates selected from 80 abstracts and presented over a period of two days. The three mentors and judges of the best paper competition (Darrell Paul, the immediate past chair of the Australian Geomechanics Society, Nick Wharmby and myself) were highly impressed by the quality of the presentations, many of which were the presenter's first ever presentation. The best paper award for New Zealand delegates went to Phillipa Mills, for her paper on developing an understanding of the behaviour of volcanic soils, and the best Australian paper award went to Nigel Ruxton. Each of these has won a place in the 6th International Young Geotechnical Engineers' Conference, to be held in September 2017 in Seoul, South Korea. Having heard the winners' presentations I am confident that the two of them will do New Zealand and Australia proud. Papers by Romy Ridl and Hamish McEwan were also of exceptional quality and given highly commended awards. A delegates' 'most popular award' went to Mondli Magagula, the delegate from South Africa, whose attendance at the conference was sponsored by the AGS. Delegates were delighted by his depiction of a large subsidence feature superimposed on a rugby field with the Wallabies playing the springboks, which he likened to the gap in the Springboks' defence.

Thanks go to the organisers, Frances Neeson, David Buxton, Luke Storie and David Lacey, for their efforts in producing an excellent conference.

BODIES OF KNOWLEDGE AND SKILLS

Over the last six months the society has taken on the development of 'Bodies of Knowledge and Skills' for CEng level Geotechnical Engineers and for PEngGeol level Engineering Geologists. These documents will provide clear definitions of what knowledge these professionals can be expected to have at the point of becoming registered in each discipline. The Geotechnical Engineering document is now ready for issue for consultation, and the Engineering Geologists' version is well advanced.

6ICEGE

The final report on last year's 6ICEGE conference was released in August. This includes the results of the feedback survey, which showed overwhelmingly positive ratings for the conference. $72,531 of surplus from the conference has been passed to the society, on top of the return of the $20,000 seed funding, for the provision of scholarships for YGPs to the 7ICEGE conference in Rome 2019 and other NZ conferences, and for the publication of the invited papers to the conference in a Special Edition of Soil Dynamics and Earthquake Engineering.

Finally, with the extraordinary amount of ground damage from the new series of earthquakes now apparent, please take extra care of yourselves when carrying out inspections of unstable ground. Your safety is paramount.

Charlie Price
Chair, Management Committee

Marlène is Senior Lecturer at the University of Canterbury in Engineering Geology. She previously worked in tunnel design in Switzerland, the USA and Australia, having obtained her PhD in tunnelling at Queen's University in Canada. She currently works in rock mechanics applied to tunnelling, geothermal, petroleum, landslides and seismic amplification with a particular focus on lab testing and numerical modelling.

NZ Geomechanics News co-editor

Don Macfarlane has worked as an applied engineering geologist for nearly 40 years and has accumulated some knowledge, a fair bit of wisdom and a few brickbats along the way.
His real interest is dams and associated issues (seismic hazard, slope instability) but any good geohazard affecting an engineering structure will do. These days he is a Technical Director with AECOM in Christchurch.

NZ Geomechanics News co-editor

IN ROSS' OUTGOING editorial in June he talked about the great advances industry, academia and government have been making in geotechnical engineering in New Zealand. He also talked about the challenge of incorporating this research into practice. We created this special feature to highlight some of the world-class research that has been happening recently so the wider membership has a chance to see advances we have been making. This will hopefully lead to a dialogue between researchers and practitioners that facilitates incorporation of our innovations into practice. I encourage you to continue to submit research articles for future issues so we can keep this dialogue going.

We are also starting to publicise the next NZGS Symposium in Napier on 23-25 November 2017. This is always a great opportunity for industry, researchers and regulators to come together to highlight case studies, new findings, and new directions for the geotechnical industry. Get your ideas together, look out for the call for abstracts, and see you this time next year!

In addition to the special feature, this issue presents an important draft document that aims to provide guidance for assessing candidates for registration as CPEng Geotechnical Engineers. This is a large collaborative effort between IPENZ, NZGS and MBIE that is now ready for contribution from the wider NZGS membership. The NZGS is working on a similar document for PEngGeol that will be ready for feedback in a few months' time.

We want this document to reflect what we as a professional society think a Professional Geotechnical Engineer should be able to do. We are a society of practitioners, educators and researchers in geo engineering, encompassing a wide range of backgrounds, training, skills and expertise. This leads to a variety of different approaches to our discipline and it is important for all of us, geotechnical engineers and all others in this field, to read this document and provide feedback.

Finally, with the recent earthquake events in the top of the South Island, we want to remind you to keep safe while out doing field work. Consider putting your observations and project news into the next issue of Geomechanics News, which will include a special feature focussing on the early geotechnical response to the earthquakes. Of particular consideration will be: how did the natural, engineered and built environment perform, how resilient are we, what kind of field work have we done, and what have we learned so far? Given the time frame we expect to receive mostly short technical items and plenty of photos. Longer items, opinions and letters to the editor will of course also be accepted.

Tell us about your project, news, opinions, or submit a technical article. We welcome all submissions, including:

- technical papers
- technical notes of any length
- feedback on papers and articles
- news or technical descriptions of geotechnical projects
- letters to the NZ Geotechnical Society or the Editor
- reports of events and personalities
- industry news
- opinion pieces

Please contact the editors (editor@nzgs.org) if you need any advice about the format or suitability of your material.

News – In Brief

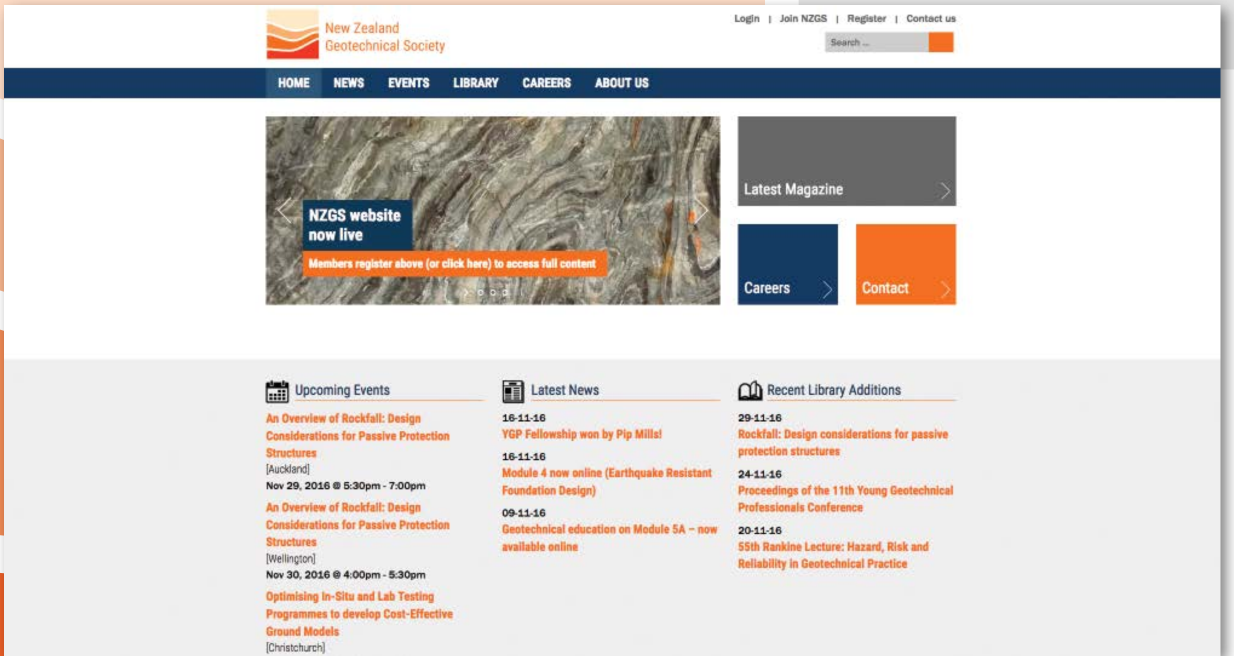
AUTOCAD CIVILS 3D GEOTECHNICAL MODULE UPDATED

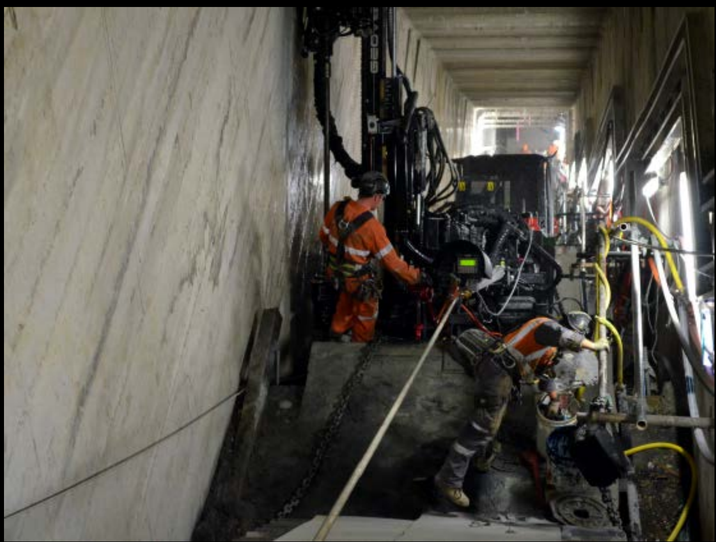
Keynetix developed the update of Autodesk's Geotechnical Module for AutoCAD Civils 3D 2017, which now allows borehole locations to be selected using the drawing filter; has redesigned menus, including colour coding of strata, for easier navigation; and automatic scaling of annotative text to match production drawings. Previous contributor to Geotechnical News and Keynetix Technical Director, Gary Morin, told us that they are offering free online training to support the launch of the new Geotechnical Module at http://geotechnicalmoduletraining.keynetix.com/. "The course is an excellent way for users to learn how to get the most out of the updated module," Morin said. "It is presented in 'bite-size chunks' that can be viewed over a few weeks. We can also tailor training for more advanced users."

NZGS New Website

Our new website is now on line. Please sign up to join as a member for full access to the site.

There is a large library available as well as a calendar to keep you up to date with all events. For more information: http://www.nzgs.org/





GROWING GEOTECHNICAL ENGINEERING PRACTICE

AN EDUCATION PROGRAMME on earthquake geotechnical engineering will help the building industry provide communities with more resilient buildings.

The programme has been jointly developed by the Ministry of Business, Innovation and Employment (MBIE), the Institution of Professional Engineers of New Zealand (IPENZ) and the New Zealand Geotechnical Society (NZGS).

It aims to provide consistent and up-to-date knowledge on earthquake geotechnical engineering practice, drawing on lessons learnt from the Canterbury earthquakes.

The information will be available in a variety of formats; including face-to-face and online resources and events. They are developed for professionals with a background in soil mechanics and earthquake engineering and will also be useful for a wider audience including engineers looking to upskill, immigrant engineers looking to move to New Zealand, architects, developers, land planners and building officials.

"The education programme is aimed at growing knowledge and understanding of earthquake geotechnical engineering practice," says Mike Stannard, MBIE's Chief Engineer.

"It's critical we understand the ground conditions on which our buildings and infrastructures are built. We live with seismic hazards and have complex and variable geological condition.

"The multi-year programme is being informed by ongoing research, both within New Zealand and in collaboration with international researchers" says Mike.

Visit the page: https://www.building.govt.nz/building-code-compliance/geotechnical-education

Release of Joint MBIE/NZGS Geotechnical Guidance Modules 2 and 4: Geotechnical Investigations and Earthquake Resistant Foundation Design Guidance.

7 November 2016: Two new joint MBIE/NZGS geotechnical engineering guidance documents have now been published as Building Act s 175 guidance:

Module 2 "Geotechnical investigations for earthquake engineering" and

Module 4 "Earthquake resistant foundation design"

These are part of the `Earthquake Geotechnical Engineering Practice' series, Refer to News at **nzgs.org.**

All of the modules are also available on both the New Zealand Geotechnical Society (NZGS) and MBIE websites. These versions are being issued for public comment. NZGS members are encouraged to make use of these documents and return comments to modulefeedback@nzgs.org within six months for consideration by the editorial committee. Comments are also welcome from structural engineers and others working in earthquake engineering.

These are the fourth and fifth of the new modules in this series to be published. Modules previously published are:

- Module 1: Overview of the Guidelines
- Module 3: Identification, assessment, and mitigation of liquefaction hazards
- Module 5A: Specification for Ground Improvement for Residential Properties in the Canterbury Region

Other modules in the series dealing with ground improvement and retaining wall design are scheduled for release in 2017.

MODULE 4

Module 4 provides the principles for good foundation design which brings practice
up-to-date and implements recommendations from the Royal Commission. We recommend that practitioners make themselves familiar with it and apply the guideline principles in their designs. Feedback during this public comment period will inform decisions about the future direction of B1/VM4 (verification method for foundation design). B1/VM4 hasn't been updated for some time and it has quite limited application. It does not address liquefaction-prone sites or those with other stability issues. Some aspects of Module 4
may be perceived as being inconsistent with B1/VM4, particularly related to strength reduction factors. However, shallow footing design is mostly governed by settlement considerations in the serviceability case, so the change will be minimal in these cases. For deep foundations, Module 4 references AS2159: 2009 to provide a risk based approach to strength reduction factors.

GEOTECHNICAL ENGINEERING EDUCATION PROGRAMME

An education programme is supporting the release of the documents. A seminar series on Module 4 on foundation design with worked examples for experienced professional will be held at various centres around New Zealand in early 2017.

Other resources are available nzgs.org, mbie.govt.nz and ipenz.nz.

AGS sponsors 11YGPC delegate from Africa

As part of a collaboration with the African region of ISSMGE, the AGS funded a young geotechnical practitioner from Africa to attend the 11th ANZ Young Geotechnical Professionals Conference (11YGPC) held recently in Queenstown, New Zealand. The African region of the ISSMGE ran a competition to select a candidate with Mondli Magagula of Jones and Wagener in South Africa selected as the winner. His attendance at the conference was extremely well received by all delegates, as evidenced by him winning the popular vote for best presentation. Mondli has extended his thanks to the AGS for the opportunity, and looks to champion future collaboration between young geotechnical practitioners across the globe. Outside of the conference, he seemed to particularly enjoy seeing and touching snow for the first time in his life.

Above: Mondli Magagula (left) experiencing snow for the first time in his life, with Darren Paul, AGS Immediate Past Chair.

A strong desire was expressed by the delegates to continue this initiative at future conferences, and we trust that this will be the case.

H&S considerations for in-field working

Ask a team member about Health and Safety Compliance and you'll get a shuffle of feet, sideways glances and perhaps a pained expression. We are going to outline one approach we've seen work in the field with geotech engineers.

We all know that implementing Health and Safety is essential for compliance, we know that the practice of H&S will get everyone home safe at the end of the day but there is a conflict because H&S can be seen as form-filling that gets in the way of getting the job done.

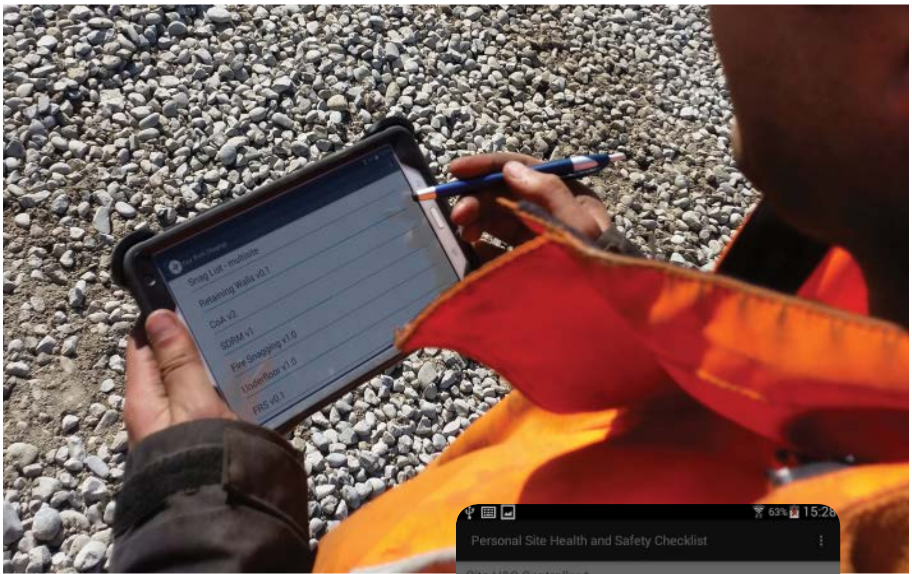
Technology should be able to help and there are many mobile H&S systems available. These mobile H&S form systems often link to a central database where patterns and trends can be reported and investigated.

These systems do remove the latency in the H&S process (the time taken for paper to move from site to office to being processed) but suffer from the same essential problem as paper forms: they are not central to the task you have been allocated, you have to remember to do them; they are a separate system.

Here's an approach that works – incorporate the H&S forms into the engineer's or rig technician's core tasks.

For example when performing a geotechnical investigation or site inspection using a mobile Site Inspection App, the first step can be designed to ensure that the Site Hazard Checklist (say) has been reviewed and a new one completed if required. This being the first compulsory item on the electronic form it cannot be ignored.

The next essential success factor for this approach is that the staff cannot complete or report their core task until the H&S information has been completed. The third is



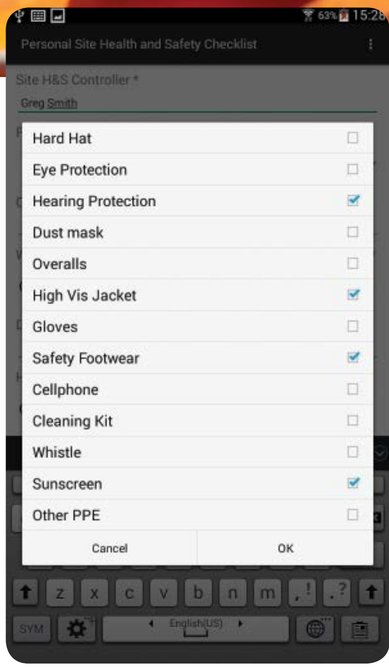
Above: Build your Health and Safety directly into your App

getting the H&S data and records delivered electronically to the relevant officer. (You can "split out" the H&S data and deliver it separately from the engineering data which goes into a report.)

This approach has driven a better rate of compliance than seen before, and has been a winner with staff who no longer have to shuffle paper around. By bringing the H&S to the fore, and by being self-managing, staff have been noticeably more comprehensive in their assessments and move on from the idea that H&S is just form-filling.

There is a fly in this technology ointment though. The H&S database you run may not accept data from another system - ie your geotech site assessment app.

The conversation to have with your H&S software supplier is to ask



them to add an API - an interface to allow data from your business systems, your mobile inspection systems and dataloggers, into your H&S system. Then you really have H&S flowing through the heart of your business.

Report by Mark English, Clarinspect

Opus Materials Testing Laboratories

Opus laboratories have a proud history dating back more than 65 years. Opus have a network of thirteen IANZ (International Accreditation New Zealand) accredited laboratories, consisting of eleven in New Zealand and two in Fiji. We provide specialist independent testing and analysis of engineering construction materials.

Through collaboration, our multi-disciplinary team of highly trained and motivated technicians have the capability to deliver large projects in a short timeframe.

Opus laboratories provide a wide range of site investigation and laboratory services, including:

- Specialised geomechanics testing (eg triaxial)
- Instrumentation installation and monitoring
- CPT (Cone Penetration Testing) services CPTu, seismic including SCPTu and temperature cones for geothermal areas
- Field investigations and soils classification

Contact us for more information:

New Zealand Laboratories Manager:
Daniel Grebenar
t. +64 9 415 4660
e. Daniel.Grebenar@opus.co.nz

Fiji Laboratories Manager:
Stephen McCone
t. +67 9 338 2751
e. Steve.McCone@opus.com.fj

Our full capabilities and services can be viewed at:
opus.co.nz/services/laboratories

ON BEHALF OF NZGS AND THE ORGANISING COMMITTEE, I AM DELIGHTED TO INVITE YOU TO NZGS2017, THE 20TH NATIONAL SYMPOSIUM OF THE NEW ZEALAND GEOTECHNICAL SOCIETY. THE SYMPOSIUM WILL BE HELD BETWEEN 24 AND 26 NOVEMBER 2017 IN NAPIER, THE ART DECO CAPITAL OF THE WORLD.

THE NZGS SYMPOSIUM HAS NOT PREVIOUSLY BEEN HELD IN HAWKE'S BAY, AND WE LOOK FORWARD TO THE OPPORTUNITY TO EXPLORE THE EXCITING GEOLOGICAL, CULTURAL AND VITICULTURAL OFFERINGS THE REGION HAS TO OFFER.

Pierre Malan
Conference Convenor

The symposium theme will be What In Earth Is Going On: *balancing risk, reward, regulation and reality.* International keynote speakers will lead a single streamed event over two full days, complemented by top quality presenters from around New Zealand. You can look forward to engaging presentations, technical papers and rigorous discussions that consider the inherent risks of dealing with variable subsurface conditions, and the regulatory, ethical and practical challenges associated with this.

With an emphasis on practical and pragmatic geotechnics we expect this to be an extremely valuable event for all our members. This will be an interesting and exciting symposium, and I look forward to seeing you in Napier next year.

A welcome reception will be held on the Thursday evening, followed by the main symposium on the Friday and Saturday. A conference dinner on Friday night will be complemented by an optional field trip to local geological (and viticulture) features on Sunday. We are also intending to run a series of workshops and short courses on the Thursday.

A call for abstracts will be made in the next few months, with registration opening early in 2017. These dates, as well as further information will be posted on the conference website **www.nzgs2017.co.nz** so please keep an eye out for this. Sponsorship and exhibition opportunities are still available, and you can find further information in the prospectus available on the website.

WHY SHOULD YOU ATTEND THIS SYMPOSIUM?

This is New Zealand's prime forum for sharing geotechnical knowledge. A huge volume of technical content is distilled into a few days by leading professionals in your industry, providing a high quality, condensed learning experience. With the emphasis on the practical and pragmatic side of our work you will be using your learnings in your every-day practice.

The symposium will be attended by industry icons, consultants, contractors, suppliers, academics and students. You will make lasting contacts who can help you with technical and career advice.

This symposium will be held in sunny Napier, one of the best destinations in the country. You'll get to stay in accommodation with sea views, eat fine fresh local produce while drinking locally made wine or beer with your colleagues. You're bound to enjoy the experience while you're learning and networking. You could arrive early, stay late and enjoy all the offerings of Hawke's Bay.

WHY SHOULD YOU SUBMIT A PAPER?

Submitting a paper is a great way to share your knowledge. It also helps increase your profile and gives you opportunities to present your work. If you have something to share around our topics of risk, reward, regulation and reality, with a focus on the practical and pragmatic, it's time to start preparing your abstract.

OUR PROPOSED SUB-THEMES ARE:

- **Communicating and managing risk**
- **Natural hazard and resilience**
- **Innovative tools and techniques**
- **Standards, guidelines and regulation**
- **Putting theory into practice**
- **Best practice in design and construction**
- **Driving efficiency from investigation to construction**

WHY SHOULD YOU SPONSOR?

This event, held only once in every four years, is one of the few opportunities to get your message across to the leading geo-professionals in New Zealand. With a totally New Zealand focussed symposium you will have the opportunity to promote yourself as a supplier or future employer to the best in our industry. You will have a chance to interact on a personal level with nearly 300 people representing around a third of the national NZGS membership.

NEW ZEALAND GEOTECHNICAL SOCIETY INC
www.nzgs.org

Letters to the Editor

The Editor
New Zealand Geomechanics News

DON TAYLOR'S CONTRIBUTION TO THE SOCIETY

In the very good obituary to Don Taylor published in NZ Geomechanics News, Issue 91 (June 2016) I noted that there was only a very brief mention of the very important role he played in the development of our Society during the 1970's decade, when he was Chairman from 1974/77.

The Society was first established in the late 1950's as the New Zealand National Committee of the International Society for Soil Mechanics and Foundation Engineering. It acted as a communication point between the International Society and the local engineering institution (then called the New Zealand Institution of Engineers). For the first ten years of its life, there was only a very limited amount of local activity generated by the Society and it had a very small membership. Don Taylor's colleague, Ralph Tonkin was the fourth Chairman of the Society for two years in 1967/69.

The big change occurred in 1972 when the name was changed to the New Zealand Geomechanics Society which also became the National Committee for communication with the International Association of Engineering Geology (IAEG) and also for the International Society for Rock Mechanics (ISRM). The rest of the 1970's decade was the time for very rapid expansion as the Society moved from being a small technical group of NZIE towards being a fully fledged technical society by the end of that decade. Don became the sixth Chairman of the Society from 1974/77 and, during this time, the activities of the Society greatly expanded, with local groups being established and membership of the Society greatly increased.

Don Taylor's wise counsel and leadership of this Society during those three years was a major contribution to the ongoing development of the Society towards the organisation we know today. I was fortunate to be able to succeed Don as the seventh Chairman of the Society from 1977/80 and helped continue the process through to the end of the decade.

John Blakeley
Life Member
New Zealand Geotechnical Society

To the Editor

REDEFINING THE GEOTECHNICAL SPACE

Com'on we all know Geotechnical Engineering is highly technical and let's be honest, not the most exciting of engineering professions – or is it?!

Having just joined CMW I was very surprised with my first impressions of the company: a whole lot of younger Geotechnical Engineers very much setting the tone for engineering as a profession, which made me rethink why such talented and motivated people found Geosciences appealing and would consider this an interesting and long term career proposition.

The image we might still have of highly experienced but 'stuffy' older men, is something I can see being revolutionised as we attract the brightest minds into construction and alongside a booming construction sector it looks like the future of New Zealand Engineering Sciences is in a very good place with vibrant prospects ahead.

What does this come down to, I asked myself, thinking the best place to get answers is to ask the very people who make Geosciences look good as a lifelong career choice.

Coming into the organisation as business development manager it is my job to snoop around and get a sense of how staff talk to each other and how this translates into thinking about the business overall, and couldn't help overhearing a conversation regarding how our customers saw us and what they thought were the key qualities, which made us attractive to do business with, summed up in 3 words:

'Attentiveness and responsiveness'

PETE: GEOTECHNICAL ENGINEER

'It means we are responsive and personally involved with the customer, easily accessible and on hand at any time to answer questions and give assurances, and basically available when and where the customer needs it.'

Pete is one of the new breed of young Engineers, quick, smart minded and very customer focussed.

Pete explains, "the whole process from the initial call, quoting, doing the work, reporting, customer follow up and even debt collection is largely handled by each individual Geotechnical consultant from woe to go."

The upshot from what I was hearing is the Geotechnical engineer becomes very familiar with the 'whole of project,' so much so they intimately understand the process of dealing and doing business with customers in all situations, helping them to develop as all rounded and 'grounded' consultants.

This also helps develop a unique insight in how to handle a variety of project issues and personalities, as well as effectively deliver a great result having gone through the correct peer review and signoff procedure.

KIERAN: ENGINEERING GEOLOGIST

Moments after talking to Pete I was approached by Kieran recently arrived from Perth to be part of the New Zealand construction scene.

He started by explaining his background in Science emphasising his interest and 'love' for the business side of Geosciences, passion for client interaction and building long term and successful relationships with customers.

We also got onto the subject of technology and the future of Geosciences, and how this ultimately would be led by young Geoscientists coming into the profession with skills not previously part of the sector, now naturally applied as a 'normal' way of doing their work.

"Oh, by the way I am really keen on demonstrating 3D Engineering models as I have a passion for art and design and would love to get into the BIM animations for our clients as well."

"As the construction industry above the ground has been revolutionised by 3D modelling in all their design development, CMW uses the technology in their Geotechnical investigation and reporting with the advantages of digital animation to demonstrate ground conditions in a way that is easily understood by construction companies and their non-construction clients."

OMG I thought, a man of science with a bent for the 'creative side' of business with a Geology Degree as well, perfect balance of 'left brain and right brain,' unheard of I thought until now,

---

CHELSIE: ENGINEERING GEOLOGIST

I was also struck by another young Geologist who seemed to stand out in what is, let's face it, still a male dominated profession, so sat down for a chat with Chelsie and got straight into asking what attracted her into something, which may still not be a natural career choice for many women of her age.

"From the start I liked the idea of a job with variety, the outdoors and the whole science behind the Geotechnical industry."

"But, for most 'girls' (her words) I think the physical aspect might be a little daunting especially as it does require drilling holes and that is a bit of a challenge!"

"So," I asked, "how did you get around that?"

"Well you have to be tough, but I can cope and with the whole team helping with heavy lifting we get the job done"

"And overall, how do you like the industry."

"I love it, it is such a challenge, highly skilled, technical and a great career choice for women, if only they understood what Engineering is really all about and how they can get into it and enjoy it as a great job with plenty of challenges and lots of opportunity"

We finished by agreeing she really was a great ambassador for 'girls', as she put it, to follow her lead and join an industry with great career prospects, a buoyant future and great remuneration.

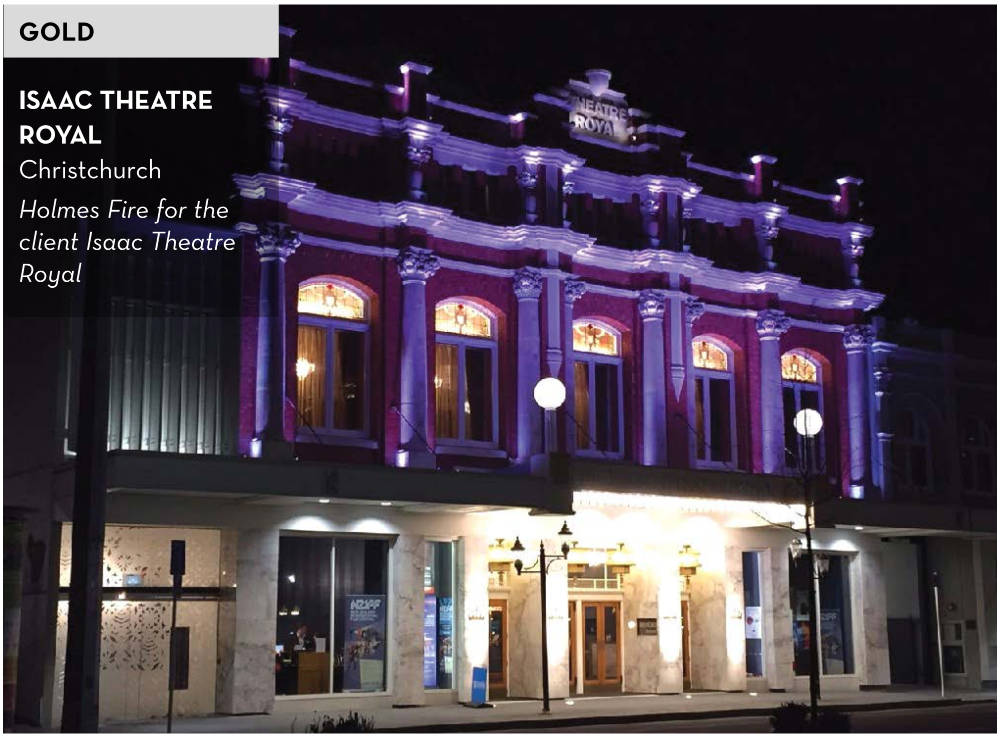
CMW is headed by a senior team of Consulting Geotechnical Directors and a dozen or so senior management Consultants across the group in Australia and New Zealand representing a vast amount of construction experience and depth and breadth in Geotechnical expertise, which has generated phenomenal growth for the company.

However, it is the sheer energy and vitality the younger engineers bring to the job, something to be encouraged and harnessed, as a positive and exciting way of doing business with new ideas and fresh perspectives.

The focus on young people as they graduate and seek opportunities in the workforce should be a real catalyst for building a dynamic culture where not only their professional skills are fostered and developed but their enthusiasm and exuberance is heard, listened to and acted on.

Frans Huysmans
Business Development Manager
CMW Geosciences

Top Major NZ Engineering Projects were recognised at Annual Awards



GOLD

ISAAC THEATRE ROYAL
Christchurch
Holmes Fire for the client Isaac Theatre Royal

***THE ACENZ* (Association of Consulting Engineers) INNOVATE Awards acknowledge and honour innovative and exceptional consulting engineering projects in New Zealand and Australasia. This year's Awards of Excellence, held on Friday, 2nd September at the Ellerslie Events Centre in Auckland recognized 20 award winning projects from New Zealand and international locations. The INNOVATE Awards acknowledge and honour innovative and exceptional consulting engineering projects. Additionally, the ACENZ Awards (informally known as the people awards) recognised 7 individuals and presented 1 ACENZ Special Award. The awards welcomed over 240 guests to the black tie dinner, sponsored by HEB Group.**

ACENZ President Keryn Kliskey says "Those who receive Innovate Awards can be proud of their significant contribution in delivery, innovation and excellence in their projects for their clients, customers and end users. These outstanding projects also recognise the effective multi-disciplinary team work and collaboration between project partners including clients and contractors. We are reminded of the pressure that our communities and customers are facing through population and climate change with associated economic, environmental, and cultural impacts. Addressing such change is a common issue in most of the featured projects including those responding to catastrophic events such as the Christchurch Earthquakes."

ABOUT INNOVATE AWARDS:
The INNOVATE NZ Awards differ from other awards as each of the projects are evaluated individually on the merit of the project alone, so there may be more than one award in any of the given categories or none at all. A project will not be awarded a prize (being Merit, Silver, or Gold) for simply being a good project. The work, technology and service must go above and beyond what is considered standard operating procedure for the industry. It is this peer-reviewed, high standard of evaluation that credits the INNOVATE Awards above all others. Each year a panel of 25-35 judges evaluates all of the submissions, conducts client interviews and tours many of the project sites. This attention to detail accompanied by a thorough investigation by a panel of peers is what makes the INNOVATE NZ Awards of Excellence the pinnacle of industry achievement.

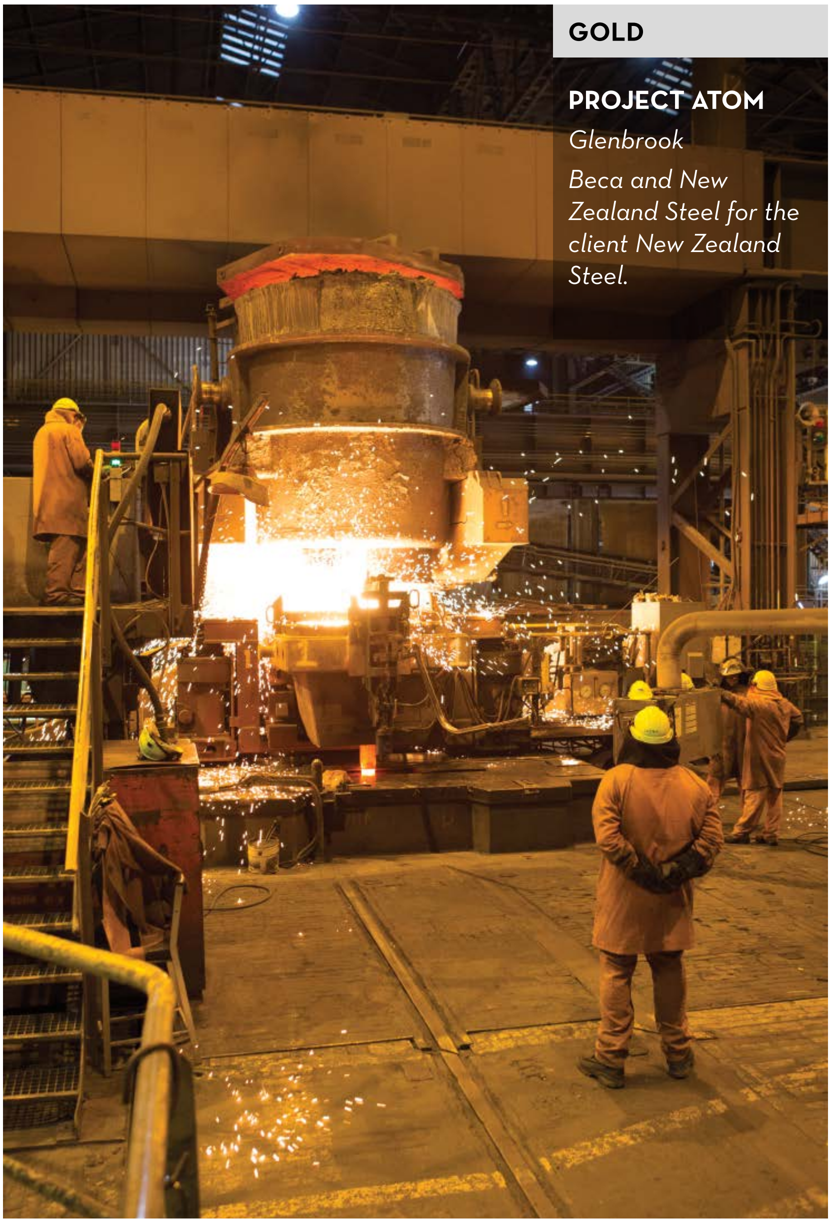
SUMMARIES OF THE GOLD WINNING PROJECTS ARE INCLUDED BELOW:

Project ATOM jointly produced by Beca and New Zealand Steel for the client New Zealand Steel.

Project ATOM was an upgrade to New Zealand Steel's existing Glenbrook site allowing flexible production and expanded capabilities for the plant. The project had to be completed in a tight timeframe in the middle of an existing and fully operational steel plant, with a completely boutique solution. This project was determined to be Gold worthy for the outstanding consulting performance working under time, capability and physical constraints to produce a custom solution for the client. The Beca team fully integrated themselves into New Zealand Steel and is a prime example of what a great collaborative working agreement looks like, raising the profile and value of the consulting engineering industry.

Isaac Theatre Royal by Holmes Fire for the client Isaac Theatre Royal

This iconic heritage building, built in 1908, sustained extensive damage during the earthquakes of 2011 and in subsequent aftershocks. Holmes Fire, working with the client and within the specific NZ Building code requirements, tailored a building specific solution which incorporated the buildings heritage status and significant artistic elements. The consultants worked within the confined spaces to produce a solution which had to be flexible enough for multi-purpose functions that the Isaac Theatre Royal required. This project was determined to be Gold worthy for the customised strategy which was developed for the client which incorporated many site challenges to produce the best solution for the client. Not only is the Isaac Theatre Royal a good example of what excellent consulting services look like, they worked with the artistic and site restrictions of the project and produced an excellent solution for the client. This project has helped raise the bar for consultancy services in fire engineering.

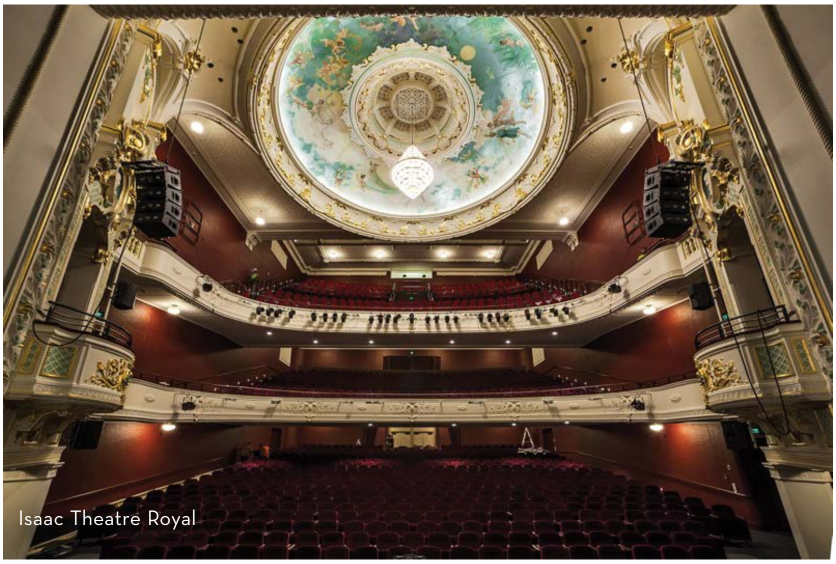


GOLD

PROJECT ATOM

Glenbrook

Beca and New Zealand Steel for the client New Zealand Steel.



Isaac Theatre Royal

GOLD

TUVALU BORROW PITS REMEDIATION
Tuvalu, Soth Pacific
Calibre Consulting for the client the Ministry of Foreign Affairs and Trade

After

Before

Tuvalu Borrow Pits Remediation by Calibre Consulting for the client the Ministry of Foreign Affairs and Trade

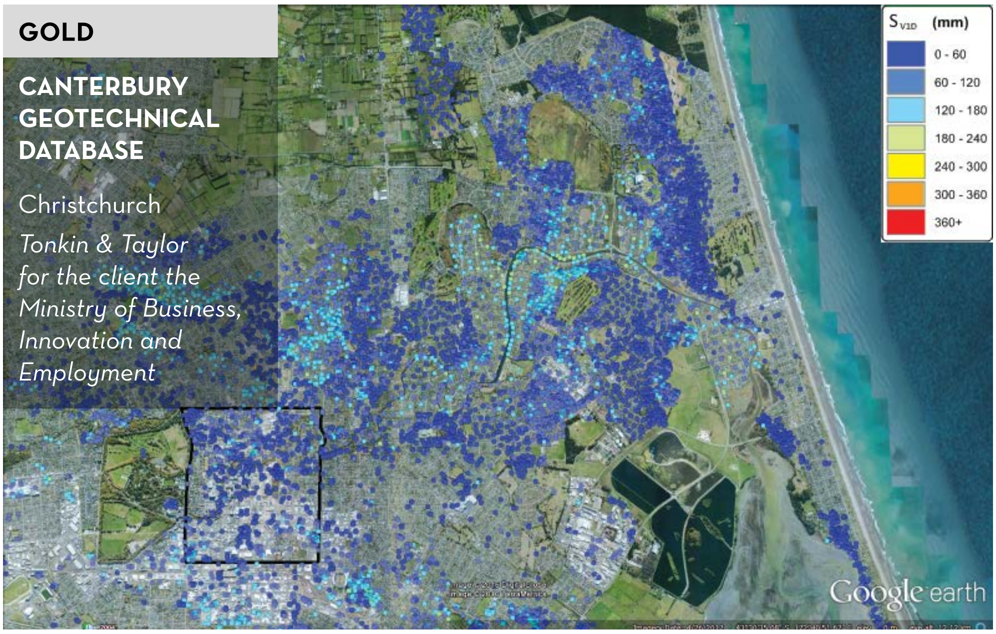
Tuvalu had many "borrow pits" that were excavated during WWII to build an airfield. Over time, these pits filled with rubbish and effluent which led to substantial environmental and health issues for the small island nation. This project was determined to be Gold worthy for outstanding consulting service amid numerous and critical challenges. Some of the challenges included multiple private ownership of land, Tuvalu's isolated location, the tight construction timeframes outside Tuvalu's cyclone season, and understanding Tuvalu's cultural needs and legislative requirements. The product of Calibre's outstanding consultancy work has had a huge impact on the health and wellbeing of the Tuvalu community, as well as setting a benchmark for cost-effective climate change response in the Pacific.

Canterbury Geotechnical Database by Tonkin & Taylor for the client the Ministry of Business, Innovation and Employment

The Canterbury Geotechnical Database is an online database developed initially to assist in the rebuild following the 2010-2011 Canterbury Earthquake Sequence. Through its evolution, the team at Tonkin & Taylor shaped it's functionality and capabilities to be progressive and adaptive in order to handle the growing use within New Zealand and worldwide. Prior to this database, information was scattered and limited. This project was determined to be Gold worthy for the innovative method of data collection and reporting, creating an IT platform that encourages collaboration, for both day to day consulting as well as world leading research. The interactive nature of the platform has changed how consultants share information for the betterment of the industry and the public good. The results have been so successful; this system is being modeled in the US, and will become a permanent national system within New Zealand.

ABOUT ACENZ:
The association that represents business services and advocacy for consulting professionals in the built and natural environment. We are the trusted advisor providing business leadership in matters relating to the consulting and engineering industry. ACENZ offers firm based membership with a current total of 179 Member Firms which represent over 10,500 staff.

Full project summaries (200 words+) can be found online here: http://acenz.org.nz/Content_8.aspx (each project has a PDF write up with a selection of photos we can provide).



GOLD

CANTERBURY GEOTECHNICAL DATABASE

Christchurch

Tonkin & Taylor for the client the Ministry of Business, Innovation and Employment

INNOVATE AWARDS:

From a total of 30 finalist INNOVATE projects, 4 Gold awards, 7 Silver awards, 8 Merit awards, 1 Community award, was made.

They are:

GOLD WINNERS:

1. **Isaac Theatre Royal** by Holmes Fire for the client Isaac Theatre Royal
2. **Tuvalu Borrow Pits Remediation** by Calibre Consulting for the client the Ministry of Foreign Affairs and Trade
3. **Canterbury Geotechnical Database** by Tonkin & Taylor for the client the Ministry of Business Innovation and Employment
4. **Project ATOM** by Beca and New Zealand Steel for the client New Zealand Steel

SILVER WINNERS:

1. **15 Stout Street Development** by Holmes Fire for the client Argosy Property
2. **Nelson Street Cycleway Te Ara I Whiti** by GHD, Novare Design, Monk Mackenzie, and NZ Transport Agency for the client NZ Transport Agency
3. **Wairau Road 220 kV GXP Substation** by AECOM for the client Transpower New Zealand
4. **Timaru's District Wide Wastewater Strategy** by CH2M Beca for the client Timaru District Council
5. **Southern Response Archaeological Investigations** by Opus International Consultants for the clients Southern Response Earthquake Services and Arrow International
6. **Christchurch Art Gallery Te Puna O Waiwhetu Base Isolation Retrofit** by Ruamoko Solutions and Aurecon for the clients Christchurch City Council and Fulton Hogan
7. **Ferrymead Bridge Replacement** by Opus International Consultants for the client Christchurch City Council

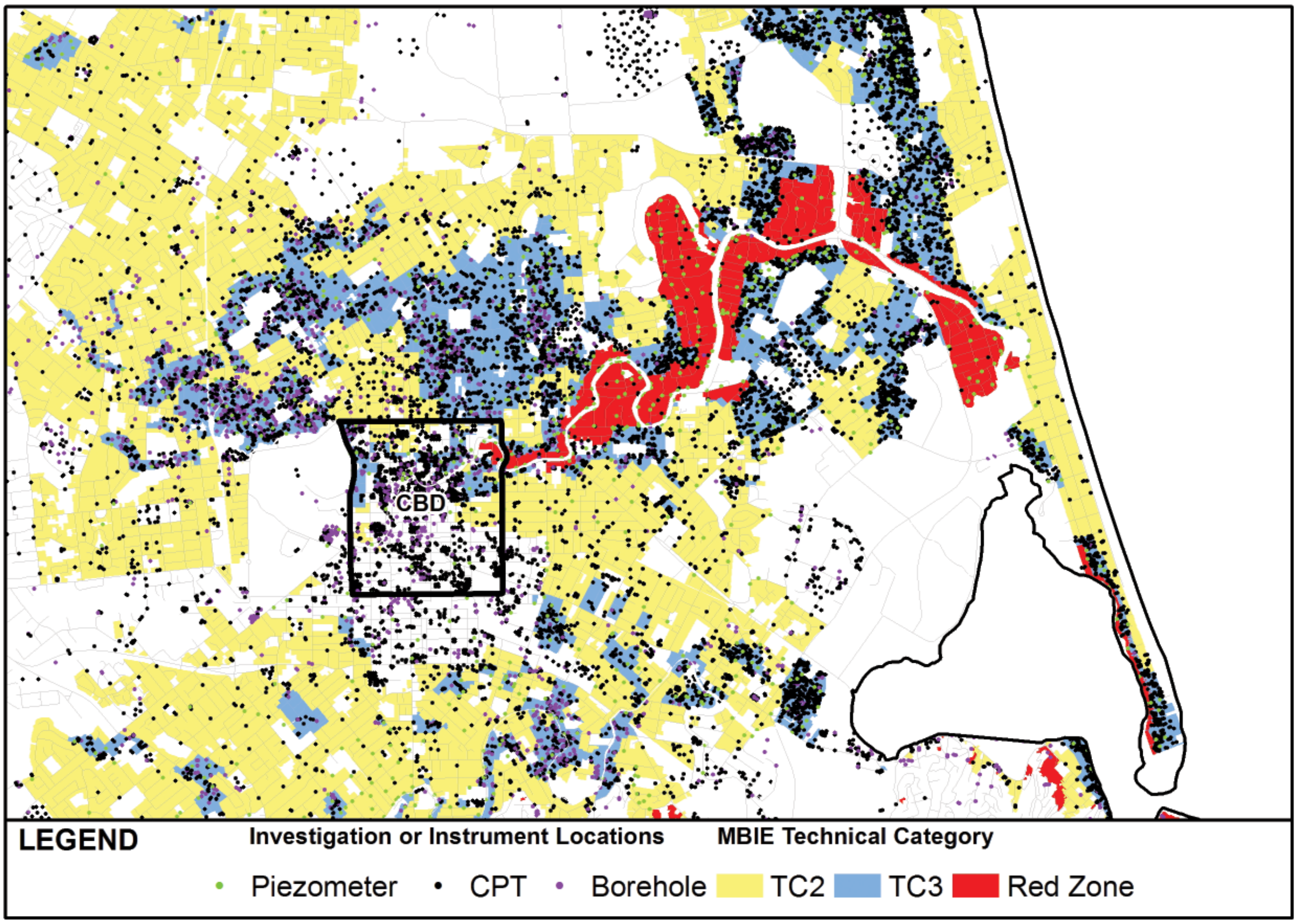
MERIT WINNERS:

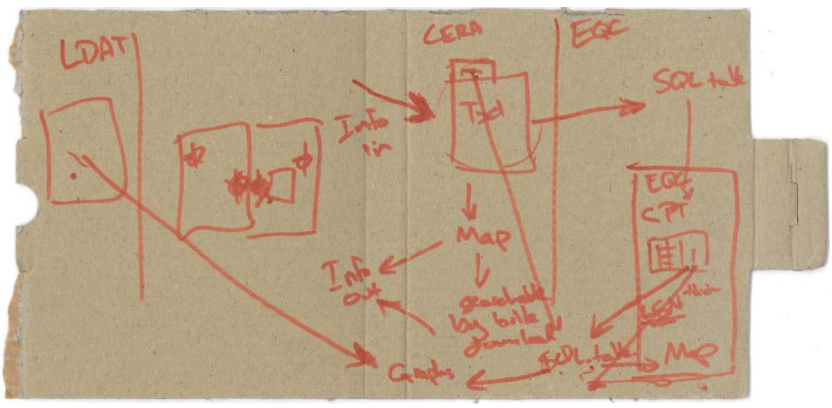
1. **Pukeahu National War Memorial Park and Underpass** (Arras Tunnel) by the Memorial Park Alliance (alliance members include: NZ Transport Agency, Downer, HEB Construction, Tonkin & Taylor, and AECOM) for the client NZ Transport Agency
2. **Pedestrian Facility Selection Tool** by Abley Transportation Consultants for the client Austroads
3. **Hamilton Southern Links** by AECOM and Opus International Consultants for the clients NZ Transport Agency and Hamilton City Council
4. **Auckland Harbour Bridge Outcomes Based Asset Management** by alliance members NZ Transport Agency, Beca and Total Bridge Services (TBS composed of Opus International Consultants, TBS Farnsworth, and Fulton Hogan) for the client NZ Transport Agency
5. **University of Canterbury Angus Tait Building** by GHD for the client the University of Canterbury
6. **Cathedral Grammar Junior School** by Ruamoko Solutions for the client the Cathedral Grammar School
7. **Sumner Surf Life Saving Club Clubhouse Rebuild** by Aurecon for the client Sumner Surf Life Saving Club
8. **Sapper** by Cook Costello for the client IAG New Zealand

COMMUNITY AWARD:

1. **Sumner Surf Life Saving Club Clubhouse Rebuild** awarded to Aurecon, Sumner Surf Life Saving Club, and Christchurch City Council

Sketch on a cardboard box results in world first and Gold Award



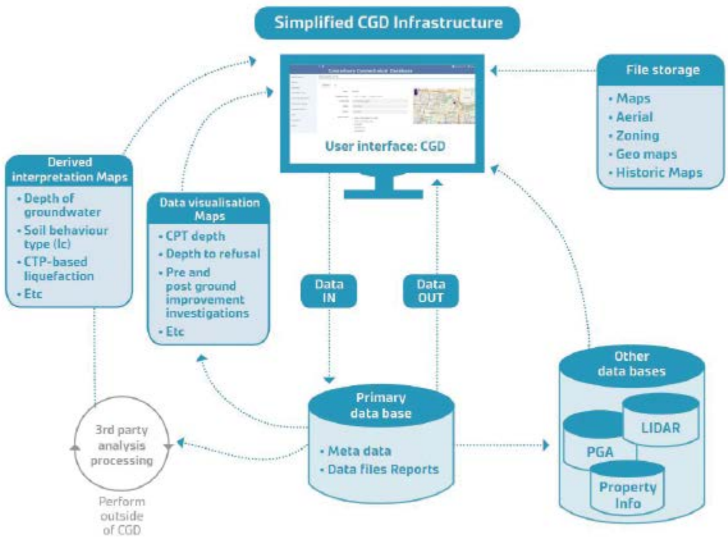


AN IDEA THAT began as a sketch on a cardboard box to expedite the Canterbury rebuild, then developed into an important world first, has just won a prestigious gold award.

From September 2010, EQC commissioned thousands of geotechnical investigations to better understand the soil and determine its resilience in Canterbury. Working out how to share that data, and in a way that others could organise, was important to the recovery. CERA had determined it could take 30 years and cost millions of dollars, if property owners had had to commission their own geotechnical investigations before consent and rebuild.

Tonkin + Taylor engineer Dr Sjoerd van Ballegooy sketched out a solution that developed into an open source, cloud-based platform to upload, download and share geotechnical investigation data to help facilitate the rebuild.

The Canterbury Geotechnical Database was born. The online database was designed by Tonkin + Taylor engineers



and IT developers with the collaboration of CERA, DBH (now MBIE) and EQC. EQC seeded the database with the $30M worth of data it had commissioned and this contribution was integral in the CGD gaining traction as a collaborative tool for professionals.

The professional engineering community, councils, insurers, academics and researchers began to experience the benefits of being able to access and share geotechnical data which had not previously been possible.

"The data sharing model benefits the whole community with significant environmental, sustainability, safety and cost advantages. Having a large database of information that continues to grow, means we are able to develop a much richer understanding of the environment and our response to it is more informed", says Tonkin + Taylor's Natural Hazard Resilience Leader, Richard Reinen-Hamill.

The success of the CGD encouraged MBIE and T+T to extend the model nationwide and, earlier this year the New Zealand Geotechnical Database (NZGD) went live.

"The NZGD has saved and will continue to save New Zealanders who are building, rebuilding, developing and redeveloping property thousands of dollars per site in geotechnical investigations and provides a lot more reassurance about the ground under our feet, and we know from the Canterbury Earthquake Sequence just how important that is to us all".

"It's a world first. Other countries are trialing the model and looking at building their own country specific cloud-based open-source geotechnical databases."

As of this week, 60,000 geotechnical investigations have been uploaded to the NZGD. The database includes cone penetration tests, boreholes, piezometers results and laboratory tests. It helps engineers determine localised ground conditions, appropriate foundation strengths and enables close examination and modelling of ground and built infrastructure performance.

"At the moment, the majority of those are from Canterbury, making it easy to see why Christchurch has been described as the most investigated and researched city in the world".

GOLD AWARD

The Canterbury Geotechnical Database was awarded Gold at the ACENZ Innovate Awards on Friday night in Auckland. (Details in this issue).

"We are rapt the tool has been recognised by our peers and sincerely hope the award will draw more attention to the NZGD so all geotechnical professionals in NZ begin using it as a matter of routine, and more countries take the model and develop their own", says Richard Reinen-Hamill.

"Ultimately, it has multiple uses including for strategic purposes such as increasing community resilience, planning for future natural disasters, catastrophe loss modelling and informing regulatory processes".

"The CGD is one of the positive lasting legacies to come out of the Canterbury Earthquake Sequence. It's incredibly rewarding to think that together with CERA, MBIE and EQC we may have made a lasting contribution to the understanding of NZ's complex ground conditions in an effort to build a more resilient future and that these practices are now being applied internationally".

Clifton Hill and Moa Bone Point Rockfall Protection

Leon Gerrard
Leon is a Project Manager with Abseil Access in Christchurch. Leon has completed a BSc in Geology and PG Diploma in Engineering Geology at the University of Canterbury. He worked for Aurecon prior to joining Abseil Access and has been involved with a range of projects in the North and South Island, including Port Hills Slope Stability work for the past 5 years.

DURING THE CHRISTCHURCH Earthquakes in 2011, the cliffs at Clifton Hill and Moa Bone Point experienced significant ground shaking and as a result a large amount of material was lost from cliff collapse and mass movement. The sites at Clifton Hill and Moa Bone Point were part of the Sumner to Lyttelton Corridor works, which includes mitigating the risk to traffic along Main Road below Clifton Hill and Moa Bone Point.

Abseil Access was contracted by Fulton Hogan to carry out a significant portion of the cliff stabilisation works, designed by Golder Associates in Christchurch. The cliff stabilisation works carried out at each of the sites included the following,

CLIFTON HILL

- Removing vegetation to gain access to the site and cliff face
- Scaling of loose rocks and protrusions that might affect the integrity of the draped mesh
- Supply and installation of approximately 3400 m² of draped Geofabrics PVC coated double twist mesh
- Supply and installation of face anchors at locations determined by the Engineer

MOA BONE POINT

- Removing vegetation to gain access to the site and cliff face
- Heavy rock scaling of isolated columns of rock using multiple airbags, in an area approximately 1200 m²
- Moderate rock scaling of the talus slope to stabilize large blocks using airbags and crow bars, in an area approximately 1600 m²
- Light rock scaling of the slope directly above the Moa Bone cave, in an area approximately 450 m²
- Supply and installation of two 3 m 100 mm diameter rock bolts to support a potentially unstable block.

Due to the high volume of traffic along Main Road (up to 20,000 combined traffic count), the traffic management plan only allowed for short duration, 4 to 5 min, road closures for any work that could an increase risk to traffic users. In particular at the Moa Bone Point site where columns of rock up to 24 m^3 were being scaled from the face using airbags.

Under traffic management control, a helicopter was used to lift the 45 rolls of rock fall netting to the edge of the cliff from the carpark. This allowed the netting to be positioned directly on to the top cable reducing the need for manual handling. With each roll weighing approximately 170 kg moving them by hand would have taken much longer and an increased risk of injury from manual handling.

The rolls of netting were then rolled down the cliff face and stitched together, secured at the base of the cliff with a 19 mm diameter galvanized wire toe

cable. The work was completed within the allocated timeframe and allowed the removal of the shipping containers providing protection to traffic along Main Road after 5 years.

Fifth International Conference on Geotechnical and Geophysical Site Characterisation, Gold Coast, 5-9 Sep 2016

Keynote speakers during conference dinner (L-R): Prof R Boulanger, Dr N Stark, Dr K Mori, Prof M Randolph, Prof P Mayne, Prof C Santamarina and Prof. A-B Huang. Absent: Prof A Gens and Dr K Been.

Hugo Acosta-Martinez
Hugo is AGS – National Chair and an Associate Director (Ground Engineering & Tunnelling) at AECOM in Adelaide. He was born and educated as a civil engineer in Colombia and carried out postgraduate studies in Spain, Japan and Australia (where he moved in 2005). His current areas of activity include railway geotechnics and major transport infrastructure projects.

THE AUSTRALIAN GEOMECHANICS Society hosted the Fifth International Conference on Geotechnical and Geophysical Site Characterisation (ISC'5) at the Gold Coast (Queensland), from 5 to 9 September 2016. This is a regular conference from ISSMGE's Technical Committee TC102 - Ground Property Characterisation from In-Situ Tests. The theme of the conference was 'In Pursuit of Best Practice'.

ISC'5 brought together the world's leading experts and practitioners in all aspects of site characterisation. The conference provided a forum for sharing their experience and knowledge with their fellow delegates. A Trade Exhibition provided opportunities to learn about the latest hardware and software in the site characterisation and extended geotechnical industry.

The conference included an inspiring Seventh James K. Mitchell Lecture delivered by Prof An-Bin Huang from Taiwan and eight other excellent keynote lectures presented by: Dr Kenji Mori (Japan), Prof Carlos Santamarina (Saudi Arabia), Dr Nina Stark (USA), Prof Paul Mayne (USA), Prof Antonio Gens (Spain), Prof Ross Boulanger (USA), Prof Mark Randolph (Australia) and Dr Ken Been (Canada). All papers from keynote presentations are included in the latest issue of Australian Geomechanics.

Keynote speakers during conference dinner (L-R): Prof R Boulanger, Dr N Stark, Dr K Mori, Prof M Randolph, Prof P Mayne, Prof C Santamarina and Prof. A-B Huang. Absent: Prof A Gens and Dr K Been.

Three one-day workshops on Cone Penetration Testing in Geotechnical Practice, Seismic Dilatometer (SDMT), and Sampling and Laboratory Testing, were held prior to the main event.

There were 24 technical sessions at which 185 papers were presented. An expert panel wrap up and discussion was held during a plenary session to close the event. This included discussions on state of practice, quantifying uncertainty and communicating risk, and future directions in technology, education and communication. A post-conference technical tour was made to Brisbane Airport's New Parallel Runway (NPR) project, one of the most complex and largest-scale ground improvement projects in Australia.

The social functions comprised the Welcome Reception (including a traditional Indigenous welcome performed during the evening), Poster Session and Conference Dinner.

The proceedings of the event consisted of two volumes with 18 theme reports and 210+ regular papers. More than 350 delegates attended from 50 countries representing all regions from ISSMGE.

What can be seen from this is that there was a very good representation of professionals from around the world, making this a truly international event. And that these professionals presented, talked, debated, listened to and learned about the state-of-the art and significant advances that are evolving in the art and science of site characterisation.

Site characterisation is a very important component of geotechnical engineering practice and the learnings from this conference were of major significance to our profession.

What cannot be seen from the statistics is the impact that the conference had on attendees. There is no doubt that ISC'5 left attendees enriched technically from sharing and learning at the conference, and also enriched personally from the networking and forging of friendships and relationships with like-minded professionals.

Of course it was not all work and no play. Like all worthwhile events when geo-professional get together there was also some "play" - the conference dinner was held with a theme "An Australian Country Pub" where GeoMusic was played according to ISC5 tradition.

The three volumes of the Proceedings of the conference will be accessible through the Australian Geomechanics website by the end of the year.

11th Australia and New Zealand Young Geotechnical Professionals Conference, Queenstown New Zealand, 25 to 28 October 2016

Above: Philippa Mills (centre) presented with the New Zealand Geotechnical Societies Youth Fellowship Award. With Charlie Price (left) and Darren Paul (right)

COMMITTEE REPORT

The 11th biennial Australia New Zealand Young Geotechnical Professionals Conference (11YGPC) was held recently in Queenstown. A good time was had by all and some would even say it went off with a bang. They were probably the ones who went over time with their presentations and were shot with a nerf gun.

The fifty delegates all aged 35 years or under assembled in spectacular Queenstown. This was the largest number of delegates to ever attend the YGPC and they were selected from over 80 abstracts. Maybe the popularity had something to do with the venue, we'd like to think it is a reflection of the level to which young geotechnical practitioners are engaged with the profession.

The objective of the conference is to provide young geotechnical professionals who otherwise have little or no experience of presenting their work to their peers, the opportunity to do so in a relaxed, low pressure forum. The delegates included a wide range of people from town and country, big firms and small, a few years experience to 10 years or so. As organisers, it was rewarding to see each delegate gain satisfaction from completing their presentations. We hope they are inspired to attend further conferences in the future.

With 50 presentations to get through in two days the nerf gun was essential, although not often used, to keep us on track. The presentations were diverse and obviously engaging; the nerf gun was also supposed to double as the "wake up tool" but was never needed, may be the odd toot from the Earnslaw sailing past the window helped with this.

The evenings included a conference dinner at the top of the Gondola and a less formal dinner the following night in Queenstown. This provided plenty of chances to mingle, with many new contacts made.

The panel of mentors, which included Darren Paul (AGS Immediate Past Chair), Charlie Price (Chair of the NZGS) and Nick Wharmby (Technical Manager, March Construction) were assigned the very difficult task of selecting the best presentations from a quality field. The winners were:

- Don Douglas Youth Fellowship Award, Nigel Ruxton of Golder Associates, Brisbane, for his work to develop a ground model and design for a challenging brownfields landfill site in south east Queensland underlain by mine workings.
- The New Zealand Geotechnical Society's Young Geotechnical Professionals Fellowship was awarded to Philippa Mills of Coffey Geotechnics, Auckland, for her work in seeking to develop an understanding of the constitutive behaviour and failure mechanisms in the highly sensitive volcanic soils of northern New Zealand.

Nigel and Philippa will have the opportunity to present their papers and to represent Australia and New Zealand respectively at the international young geotechnical professional's conference (6iYGC) in South Korea, 2017.

In addition to the main prizes, two high commendations were awarded:

- Romy Ridl, University of Canterbury for her work in looking at topographical influences on in-situ stresses and deformations in the Cromwell Gorge.
- Hamish McEwan of AECOM, Hamilton for his work in developing an understanding of the geology in a foreign country and training local geologists in site investigation techniques.

The popular award, voted on by the delegates went to Mondli Magagula of Jones and Wagener South Africa for his presentation on undertaking site characterisation around a sinkhole. His line of 'the sinkhole is about the same size as this gap in the Springboks defence' whilst showing a slide of the Wallabies playing the Springboks was easily the quote of the conference.

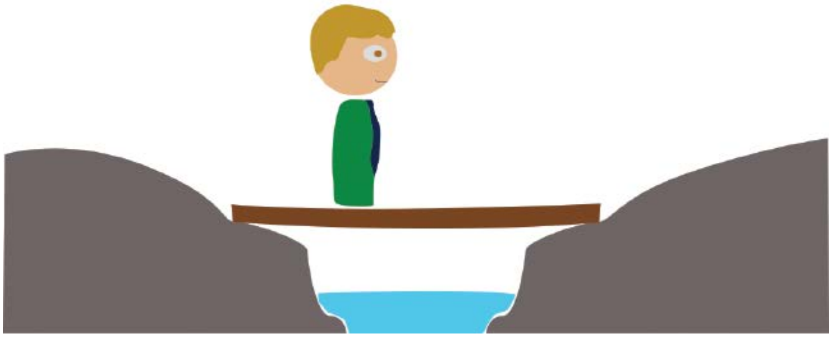
After two days of presentations, the final day was spent on a field trip to the Clyde Dam. The trip was led by Jesse Dykstra of Aecom and Greg Saul of Opus and took in a series of large landslides on the sides of the Cromwell George that were stabilised prior to the filling of Lake Dunstan in 1992-93. The field trip finished with a trip down an investigation/monitoring adit in the Clyde Slide and a tour of the Dam. The Dam seismic slip joint, which can accommodate up to 2m of movement in the underlying river fault was of considerable interest as always. Thanks to Don Macfarlane for arranging the field trip and Contact Energy for providing access and the dam tour (as well as lunch!).

Conference delegates on the field trip to the Clyde Dam

We'd like to acknowledge the conference sponsors, Opus, March Construction, T+T, Geotechnics, Aecom, Drillforce, DCN Drilling, Contract Landscapes and Southern Geophysical for their support.

The future of our profession in Australia and New Zealand is bright and we look forward to 12YGPC to be held in Australia, 2018. We encourage all young Geotechs to keep an eye out for the call for abstracts. It is well worth attending. For those who were at 11YGPC hopefully we will see you at future NZGS events.

3 Principles of the true engineer reprinted from https://www.linkedin.com/pulse/3-principles-true-engineer-maxim-millen



Maxim Millen

Maxim studied civil engineering at the University of Canterbury where he completed his undergraduate degree and a PhD. His research was focused on the design and assessment procedures for buildings including the effects of soil-foundation-structure interaction. During his studies he spent six months as a student at the ROSE School in Italy, and later returned in 2014 as a teaching assistant for the earthquake geotechnical engineering course.

THE ENGINEERING PROFESSION is advancing, advancing in the sense that engineers are dealing with more complex situations and are able to work faster due to automated design tools that can produce large design reports.

However, **I consider this the dark under belly of engineering.** It creates engineers who don't think, they just do. And it exposes us to one of the biggest risks in engineering: your analysis model misses a fundamental mechanism of the real world situation. While there are other mistakes such as calculation errors, miscommunications and missing deadlines. There is no doubt that missing a failure mechanism in an engineering design or analysis can result in catastrophe.

To reduce the chance of a fundamental error it is important to operate in a way that allows your peers and mentors to review and critique your work. In my blog (https://www.linkedin.com/pulse/convention-over-configuration-engineers-efficiency-maxim-millen) I previously talked about the importance of conventions within an engineering firm, as conventions provide a uniform structure to everyone's work and therefore makes it easier for others to understand.

This article takes a more fundamental look at how the engineering profession is advancing and the principles we should use when adopting and developing processes within engineering. The principles are:

1. Keep it Simple – avoid unnecessary complexity
2. Use thought-provoking processes – avoid design processes that you don't need to think
3. Communicate efficiently – Avoid long reports

SIMPLE

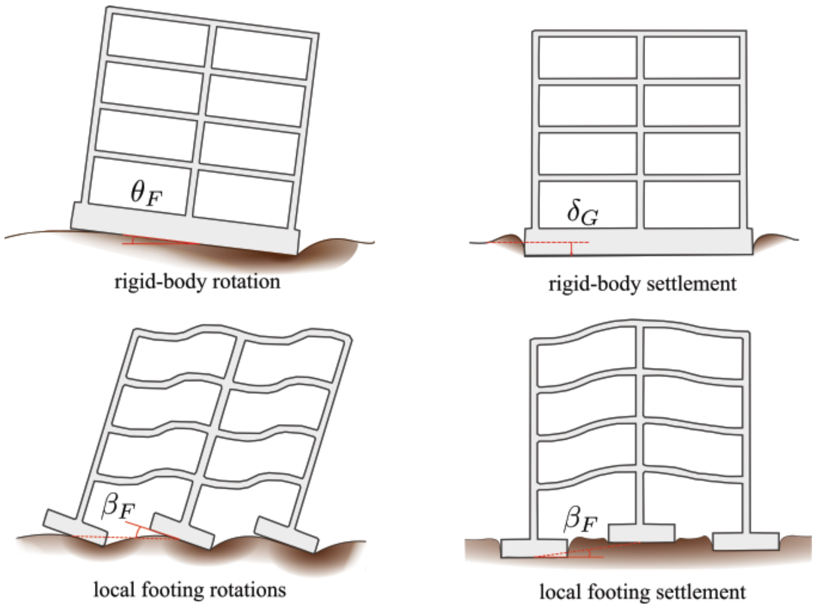
3D modeling software, optimization algorithms and time-series simulation software all have an important role to play in engineering. However, many of the techniques used in the analysis are beyond our ability to communicate and fully understand, therefore we rely on our intuition or judgment when assessing the accuracy of these techniques. This exposes us to the big risk of missing a fundamental mechanism.

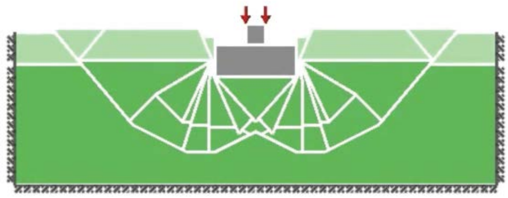
It is absolutely critical that all complex/automated design procedures are complimented with simple mechanism-based hand calculations.

This was the crux of my PhD thesis at the University of Canterbury (digitally available at the UC library) where I developed a simple hand calculation procedure to design buildings for soil-foundation-structure interaction. It was essential that the procedure was explicitly mechanism based. Even if it is crude it allows you to assess the magnitude and importance of each mechanism. In contrast, the numerical simulation of a soil-foundation-structure problem is plague by numerical instability issues and high sensitivity to damping considerations, therefore the focus of the engineer is spent on dealing with those issues rather than whether the fundamental deformation mechanisms are being considered.

THOUGHTFUL

Thoughtful or thought-provoking methods are an engineer's best-friend. They encourage you to think about your problem





while you are crunching the numbers. The opposite would be black-box automated design software, where you simply input some values and extract the output.

Unfortunately, most software seems to be a black-box because the designers want to hide all of their intellectual property. However, some software applications are quite the opposite, they provide great insight into the problem through clever visualisation or being explicit about what has been calculated.

Also, black-box processes are not constrained to software, some simple hand-calculation methods use abstracted factors that have no physical meaning and although you do all the calculations yourself, you gain no great insight into the problem.

The best example I can think of where a software provides a more thoughtful approach than a hand-calculation procedure is the calculation of bearing capacity. The typical Terzaghi formula uses vague correction factors for foundation shape and ground slope to adjust bearing capacity factors, which also struggle to derive any physical meaning, making the engineer simply lost until the final bearing capacity value is extracted. In contrast the software LimitStateGeo uses clever graphics to clearly demonstrate to the users what calculations are being run and the meaning behind them.

EFFICIENT

Efficiency is a word so tied to engineering that sometimes we forget to think about it. Of course we are looking for efficient designs that allow quick construction and minimal material. However, we should also apply efficiency to the way we convey technical information. I don't mean we should write reports faster, in fact I mean we should write them more slowly and carefully. Technical work is read many more times than it is written. Spending a bit more time to write concisely and using images to convey messages is crucial. The same can be said about writing short emails, or using an alternative to emails for in-house communication such as Slack (slack.com). We use it internally at Pensolve. It is extremely simple and removes all the added "Hi John" and "Best regards" sorts of sign off and email signatures, so you can read messages like a conversation.

SUMMARY

We as a profession are speeding up. What we chose to speed up and how we do it are important decisions. New processes are always being developed and old processes are becoming obsolete. By making sure your new processes are simple, thoughtful and efficient you are mitigating the risks around making a fundamental design or analysis error. The principles are not new, but perhaps now they are more important than ever.

NZ Geotechnical Society

2016PHOTO COMPETITION

FIRST PLACE – BEST GEOLOGY. WINNER OF $250

Michelle Willis *(Earthtech Consulting Limited, Pukekohe)* – Nature's Mighty Monuments

"The photo was taken along the edge of a walking tack near the Mangawhai beach. Gravel must have been placed on the track a while ago, and over time the rain had eroded the sand around and below the gravel, leaving these sand pillars with a piece of gravel perched on the top. I was inspired to take the photo because I had never seen anything like it before. The pillars were very well defined with vertical faces, and yet also incredibly small. I decided to submit the photo for the competition because these miniature "mighty monuments" seemed so unusual."

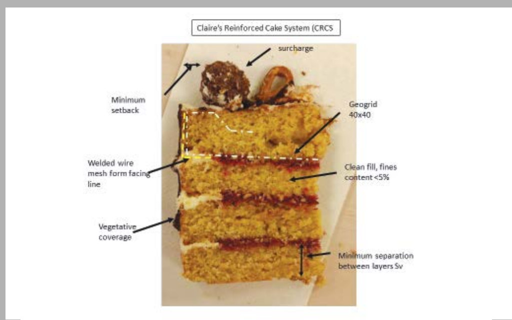
SECOND PLACE

BEST GEOTECHNICAL.
Harry Follas (Opus, Auckland) – Torhape Slip (Hauraki District)

THIRD PLACE

John Underhill (AECOM, Auckland) – *Trung Son Quarry 3A*

HONOURABLE MENTION



Andres Martinez (Cheal Consultants Ltd., Taupo) – *Claire's Reinforced Cake System (CRCS)*

Dave Ouwejan (Soil and Rock Consultants, Auckland) – *I Should Get a Desk Job*

Introduction

New Zealand is an ideal laboratory for geotechnical engineering research, with a wide variety of intensive and non-intensive land use, a large infrastructure network connecting distant parts of the country, and, of course, a very dynamic landscape all within a relatively small area. We produce world-class research in New Zealand, in a very collaborative environment that brings together industry, academia and government. In this publication we regularly highlight New Zealand research that wins international awards, and this year's ACENZ awards in this issue show the quality of the innovations we make every year.

The following papers are short descriptions of some of the most recent research that we are currently doing in New Zealand Universities and industry. Since the Canterbury earthquakes much focus has gone into capturing the lessons learned and improving our understanding of seismic geotechnical problems, so the majority of the research we showcase here relates to those advances. Given the most recent earthquakes in the South Island, it is clear that earthquake geotechnics will continue to be a strong focus of research.

One of the main things to highlight in this special feature is the participation of industry in geotechnical research. As a researcher, I cannot stress too much the value of collaboration between industry and academia. Industry provides much of the motivation for seeking solutions to engineering problems through research, as well as providing invaluable data and expertise in the field. We are lucky to work in a country with great industry participation in research, but just as every geotechnical practitioner would love one more borehole, every researcher would love one more industry collaborator.

Finally, underpinning most research in New Zealand are government funding agencies that provide us with the base load research funding to keep our labs functioning, to keep the best researchers in our universities and CRI's, and to promote industry-academic collaboration. Here's our chance to show what we've been up to and to encourage government and industry to keep investing in New Zealand research in geotechnical engineering.

Marlène Villeneuve

Marlène is a Senior Lecturer at the University of Canterbury and the coordinator of the Engineering Geology Programme. She previously worked in tunnel design and construction in Switzerland, the USA and Australia, having obtained her PhD in tunnelling at Queen's University in Canada. She currently works in rock mechanics applied to tunnelling, geothermal, landslides and seismic amplification with a particular focus on lab testing and numerical modelling.

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Research into Seismic Design and Performance of High Cut Slopes in New Zealand

Brabhaharan, P, Mason, D and Gkeli, E

INTRODUCTION AND BACKGROUND

Transportation routes are critical lifelines for the community, particularly in the event of natural hazards. New Zealand has a rugged terrain and high seismicity, which means that high cut slopes are required to form transportation routes. The performance of these slopes in earthquakes is critical to ensure a resilient transportation system. Although the focus of the research was on transport routes, the outcomes will also inform the assessment or design of steep slopes that affect the vulnerability and life safety associated with buildings and other facilities adjacent to steep slopes.

Currently there is very little guidance available for the earthquake design of high cut slopes either in New Zealand or internationally. The potential for topographical amplification of earthquake shaking, and the observation of the large landslides that have affected transportation routes in earthquake events has raised the awareness of the need for research and development of guidelines for the seismic design of high cut slopes. The New Zealand Transport Agency engaged Opus International Consultants to carry out this research and development of guidance.

RESEARCH TEAM

This research has been carried out by Messrs P Brabhaharan, Doug Mason, Eleni Gkeli and Andy Tai of Opus International Consultants, and Graham Hancox from GNS Science, and assisted by other staff from both organisations. Graham Hancox focussed on the landsliding in past earthquakes in New Zealand. The contribution of the peer reviewers, Prof George Bouckovalas, and Dr Trevor Matuschka is gratefully acknowledged. The contribution of Prof Nick Sitar is also gratefully acknowledged.

P Brabhaharan

Brabha is a Chartered Professional Engineer and Fellow of IPENZ. He is the National Technical Director for Geotechnical Engineering at Opus International Consultants, and a Technical Principal for Earthquake Engineering & Resilience. Brabha has through research and practice, developed design and assessment methods aimed at enhancing the resilience of infrastructure and communities exposed to risk from natural hazards.

Doug Mason

Doug completed bachelor degrees in geology and history and an MSc (Hons) in geology at Victoria University, and worked for GNS prior to joining Opus in 2004. Since then he has worked in NZ and the UK on a variety of geotechnical and geoenvironmental projects. His particular interests include geomorphology, rock slope stability, and earthquake and landslide hazards.

Eleni Gkeli

Eleni is an Engineering Geologist with 20 years of experience. Her main experience comes from Greece, working for the design and construction of a 680km long high standard motorway, Egnatia Odos. Eleni has been working in New Zealand since 2012, mainly for Opus International Consultants, in projects of transportation, water and building infrastructure, across the country.

RESEARCH SCOPE AND OBJECTIVES

The research included:

1. Review of the performance of high cut slopes in recent worldwide earthquakes
2. Consideration of the influences of the distinctive aspects of New Zealand's seismicity and topography,
3. Review of relevant recent research on topographical effects from New Zealand and overseas
4. Review of current design practice in New Zealand and overseas
5. Development of guidelines for the earthquake design of high cut slopes in New Zealand.

Some limited targeted numerical modelling has been carried out as part of this research to test the topographical effects and variation of seismic shaking over the height of the slope, using specific New Zealand-centric parametric analyses. The objective is to enable design of resilient and cost-effective cut slopes, taking into account the variation of peak ground acceleration over the height of the slope.

The results of the initial stage of the research, including points 1 to 4 discussed above, were presented in the 6th ICEGE held in Christchurch in November 2015 (Brabhaharan et al., 2015). The remaining work of the research is now complete and will be published by the New Zealand Transport Agency. A quick overview of the findings of the research is presented in this article.

LESSONS FROM PAST EARTHQUAKES

Detailed review of coseismic landsliding and the performance of slopes during historical New Zealand earthquakes was carried out as part of this research. This has been supplemented by a literature review to ascertain the effects of large earthquakes on slope performance in past worldwide earthquakes. The main findings were:

- The general pattern of damage observed in recent large earthquakes is of widespread slope failures characterised by shallow landslides in the surficial layers of regolith and immediately underlying weak, brittle and dilated rock mass in the upper parts of steep slopes. Large or very large landslides are comparatively rare in number and extent compared to the shallow slides, however these tend to be much larger in volume and consequently cause much greater damage.
- Widespread slope failures have occurred on slopes steeper than 40° to 50° that are underlain by young (Miocene or younger) sedimentary rocks, which have been observed to be particularly prone to these types of landslides. However, landslides in much gentler slopes in volcanic soils has been observed recently in the April 2016 Kumamoto earthquakes in Japan (Brabhaharan – pers comm).
- Steep slopes in competent bedrock are prone to more localised failures (shallow rock slides and rock falls), as well as less common, but larger, defect-controlled failures.
- The middle to upper parts of hillslopes are most susceptible to landslides, due to a combination of steep slope angles, weaker rock strength (due to the effects of weathering, dilation, fracturing etc.) and topographic amplification of ground motions in those parts of the slopes.
- Hanging wall, topographic amplification and attenuation/directivity effects result in asymmetric patterns of slope failure around the fault rupture, with larger ground motions and consequently more slope failures located at greater distances from the fault in areas that exhibit these effects. The common focal mechanism for earthquakes exhibiting these effects is thrust/reverse faulting.

CHARACTERISATION OF NEW ZEALAND TOPOGRAPHY AND SEISMICITY

The New Zealand land mass is tectonically very active due to its position within the plate boundary zone between the Pacific and Australian plates. Consequently, the topography of New Zealand is highly variable, from mountainous alpine areas, to lower relief mountains/hills, rolling hill country, rounded and incised peneplains, young volcanic terrain, recent alluvial plains and uplifted marine terraces. The differing terrains were grouped into three broad categories for areas where transportation corridors pass through hilly terrain, presented in Table 1.

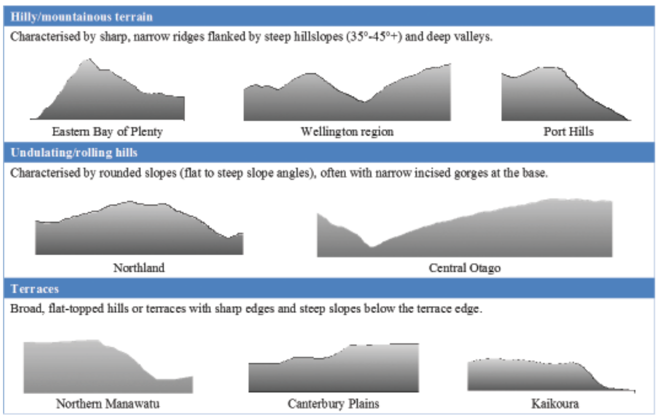


Table 1: Characterisation scheme for New Zealand topography

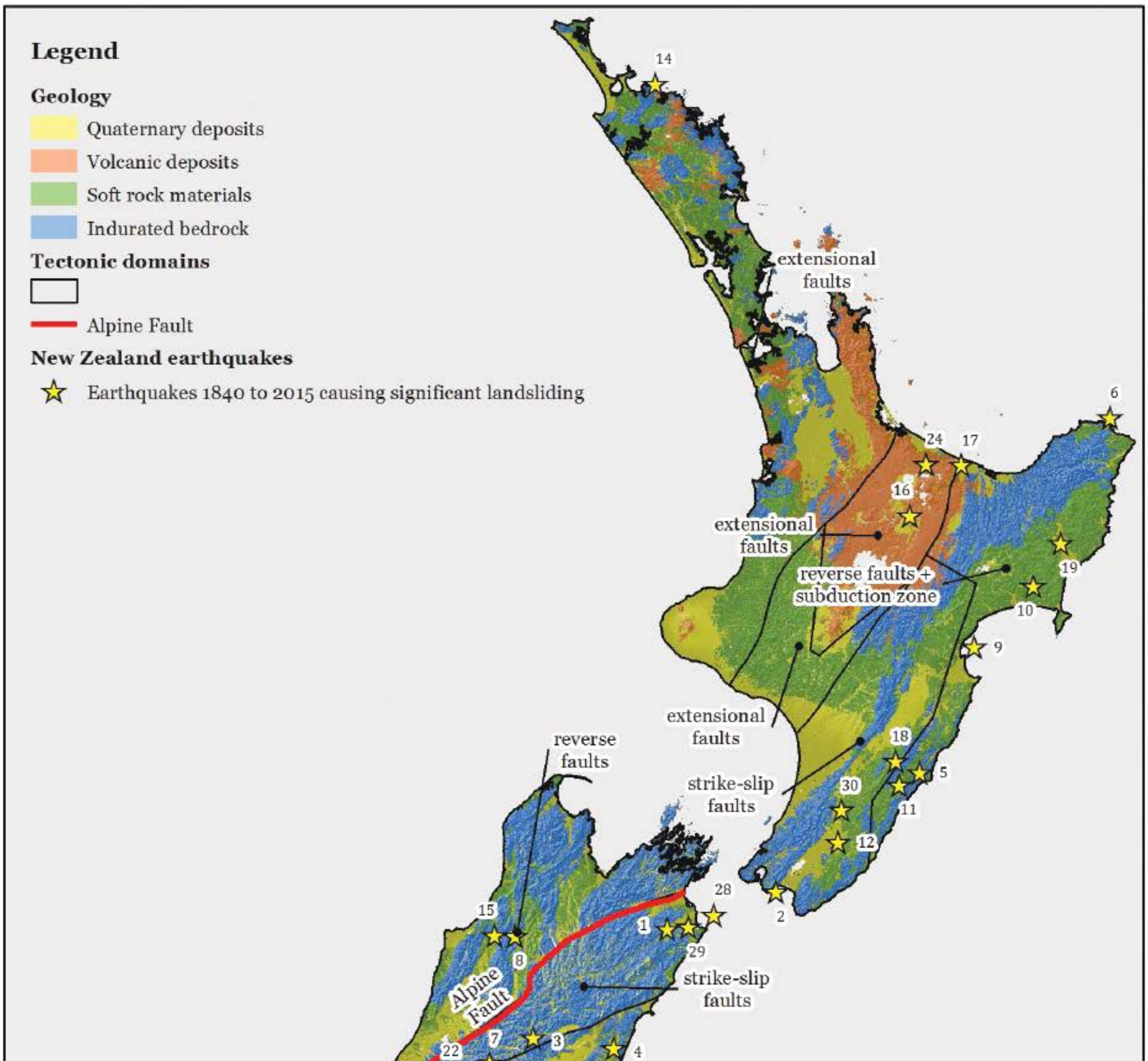
Observations from past earthquakes in New Zealand and overseas show that seismicity effects, such as topographic amplification, can have a significant effect on the distribution and concentration of earthquake-induced slope failures, particularly in areas of compressional tectonic regimes dominated by reverse or thrust faulting. The land mass of New Zealand has been characterised based on the seismotectonic type of active deformation, as shown in Figure 1. The seismotectonic regimes vary from the subduction zone and fold-thrust belt in the eastern North Island, dextral oblique-slip faults through the central part of New Zealand, reverse faults in the northwest and east of the South Island, normal faults associated with volcanic rifting on the central North Island volcanic plateau, and low seismicity areas in the northwest North Island and southeast South Island, away from the active areas.

Similarly, the review of published literature documenting the performance of slopes in different materials shows that young (Miocene or younger) sedimentary rocks are particularly prone to widespread shallow landslides in large earthquakes, with hard bedrock more prone to localised wedge failures and rock falls. Figure 1 also shows a simplified geological grouping of formations into four broad categories: indurated bedrock, soft rock materials, volcanic deposits and Quaternary soil deposits, and is based on the 1:250,000 QMAP regional geology maps.

TOPOGRAPHY EFFECTS

Topography effects is a complex phenomenon and is acknowledged in the literature that study and quantification presents challenges. The following points summarise the general conclusions extracted from literature review regarding topographic effects:

- Topography effects are influenced by shape, inclination and height of slope, as well as wave type, wavelength and angle of wave incidence. They are also influenced by the stratigraphy and geology of the slope, such as the presence of softer/weaker rock or soil layer at the top of the slope and presence of rock defects and faults.
- Amplification effects due to topography are mainly concentrated at the top of two dimensional step-like slopes and isolated cliffs or steep ridges. De-amplification is observed at the midheight and near the base of slopes and canyons.
- High amplification of the seismic motions at a zone near the crest of the slopes occurs due to a combination of the primary SV waves and reflected P, SV and Rayleigh waves. A parasitic vertical component of these waves superimposed on the incoming seismic excitation appears to be of importance.



Earthquake		Date	Magnitude
1	Marlborough	16 Oct 1848	M_W 7.8
2	Wairarapa	23 Jan 1855	M_W 8.2
3	North Canterbury	1 Sep 1888	M 7-7.3
4	Cheviot	16 Nov 1901	M_S 6.9
5	Cape Turnagain	9 Aug 1904	M_W 6.8
6	East Cape	7 Oct 1914	M_S 6.7
7	Arthurs Pass	9 Mar 1929	M_S 7.1
8	Murchison	17 Jun 1929	M_S 7.8
9	Napier	3 Feb 1931	M_S 7.8
10	Wairoa	16 Sep 1932	M_S 6.9
11	Pahiatua	5 Mar 1934	M_S 7.6
12	Masterton	24 Jun 1942	M_S 7.2
13	Lake Coleridge	27 Jun 1946	M_S 6.4
14	Peria	23 Dec 1963	M_L 4.9
16	Waiotapu	16 Dec 1983	M_W 5.1
15	Inangahua	24 May 1968	M_S 7.4
17	Edgecumbe	2 Mar 1987	M_W 6.6
18	Weber	13 May 1990	M_S 6.3
19	Ormond	10 Aug 1993	M_S 6.2
20	Fiordland	10 Aug 1993	M_W 7.0
21	Arthurs Pass	18 Jun 1994	M_W 6.7
22	Arthurs Pass	29 May 1995	M_L 5.5
23	Fiordland	22 Aug 2003	M_W 7.2
24	Rotoehu	18 Jul 2004	M_L 5.4
25	Fiordland	15 Jul 2009	M_W 7.6
26	Darfield	4 Sep 2010	M_W 7.1
27	Christchurch	22 Feb 2011	M_W 6.3
28	Cook Strait	21 Jul 2013	M_W 6.6
29	Lake Grassmere	16 Aug 2013	M_W 6.6
30	Ekatahuna	20 Jan 2014	M_L 6.2
31	Wilberforce	6 Jan 2015	M_L 6.0

Figure 1: Digital terrain model of New Zealand with simplified geology and tectonic domains (after Oyarzo-Vera et al., 2012; Stirling et al., 2012; Litchfield et al., 2013), and epicentres of historical earthquakes that have caused significant landslides (after Hancox, 2015).

- Local convexities on the slope profile can introduce important changes to the pattern of surface ground acceleration, such as amplification of the vertical component of incoming S waves.
- Topography effects fluctuate intensely above the crest with distance away from the slope, so that detecting them on the basis of field measurements alone becomes a very demanding task (Bouckovalas & Papadimitriou, 2006).
- The amplification of the seismic motion near the crest has been demonstrated in the numerical studies qualitatively, but was underestimated quantitatively. Time - domain crest:base amplification ratios from the theoretical and numerical studies do not exceed the value of 2, while values as high as 10 were observed during microtremors (Assimaki et al. 2005).

NUMERICAL ANALYSIS

Limited numerical analyses were carried out as part of the current study to provide a better insight into the variation of ground acceleration along the height of the slope, as well as the effects of cut slopes affecting part of the slope.

The topographies examined include ridge and terrace like slopes. The geometries examined are shown in Table 2. The slopes were assumed to consist of rock, so that high complexity of the model is avoided and the interpretation of the results purely concern seismic motion variation due to topography, rather than potential soil amplification effects (Cases 1 and 2 in Table 2). Limited analyses were carried out taking also into account the presence of weaker material near the slope surface, to provide insight into the degree of influence of overburden materials overlying bedrock (Cases 3 and 4 in Table 2). The effect of local convexities present on the slopes, for example created by a man-made cut at the toe of a high slope was also examined at a preliminary stage (Case 5 in Table 2).

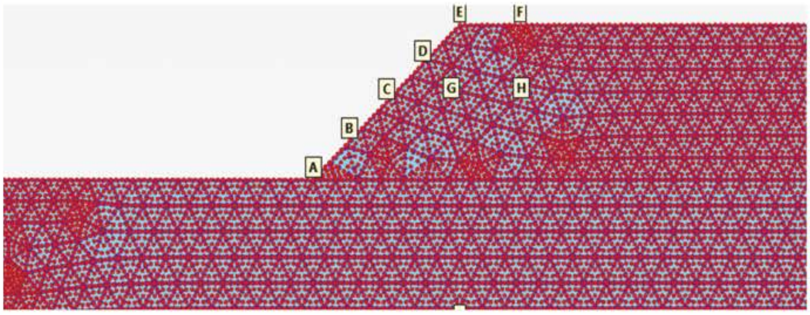
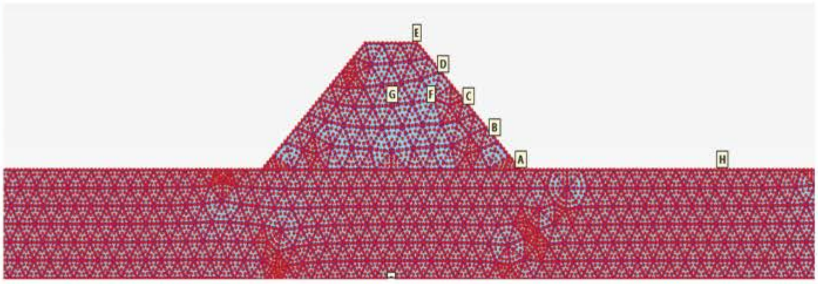
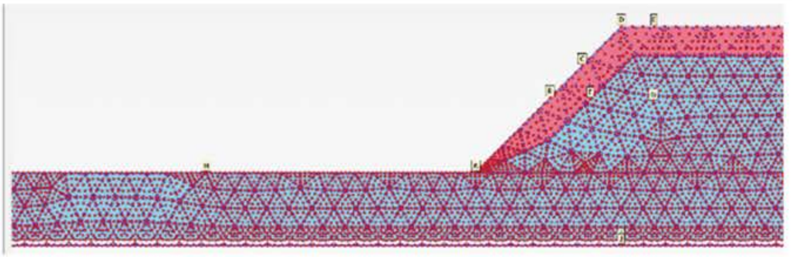
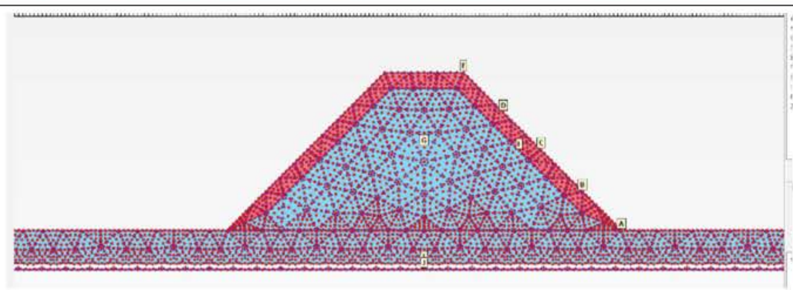
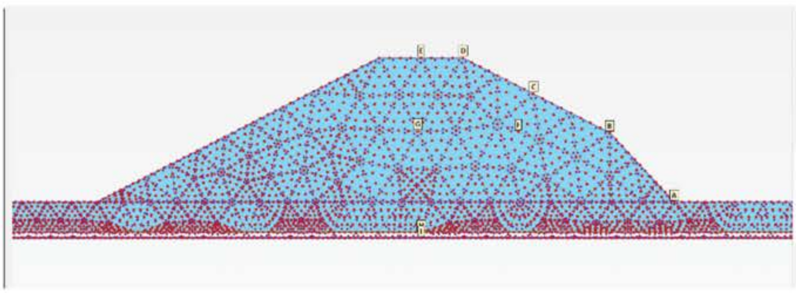
Case no	Type of Terrain	Model Geometry
1	Terrace	
2	Ridge	
3	Terrace with overburden soil layer or HW rock	
4	Ridge with overburden soil layer or HW rock	
5	Ridge with cut at the lower part of the slope	

Table 2: Summary of geometries modelled in the numerical analyses

The conclusions drawn from the analyses are in general agreement with the conclusions made from the literature review, some of them are summarised below:

- The amplification effect is found in our analyses to be mostly influenced by the frequency of the excitation and the slope height, when the slope consists of rock. It is also influenced by the presence of weaker overburden soil material or highly weathered rock overlying unweathered or slightly weathered bedrock. De-amplification of the seismic ground motions is observed at the toe of the slope and inside the slope at high frequencies.
- For the terrace topography, the amplification effect at the upper part of the slope is negligible for the small frequencies (· 2 Hz) when the slope consists of rock. However, amplification factors of the order of 1.2 and 1.4 are indicated by the numerical analyses for the 50 m high and 100 m high slopes respectively, for higher frequencies, when the slope consists of rock.
- High amplification effects are observed at the top part of the ridge for all the cases examined, with higher amplification factors occurring for the high ridges. The amplification of the top of the ridge is higher than that of the corresponding terrace-like slope.
- Considerable vertical component of ground acceleration was observed for some of the cases examined, although the harmonics used in the analysis introduced horizontal motions only to the slope. The presence of a significant vertical component has been indicated by previous researchers, as concluded in the literature review.
- Amplification of seismic acceleration was observed in the case of a ridge with a cut excavated at its toe, of slope angle 45° to 50°. The amplification factors show a tendency to increase as the irregularity becomes more pronounced.

CURRENT DESIGN PRACTICE

Design standards available for use in the seismic design of high cut slopes have been searched for and reviewed as part of the literature review. This included New Zealand, European, Canadian and United States of America standards.

NEW ZEALAND

In New Zealand, it is only the Bridge Manual third edition (New Zealand Transport Agency, 2014) that provides specific recommendations on design standards and selection of seismic design parameters for slopes associated with state highways in New Zealand. The Bridge Manual provides earthquake loading parameters, such as Peak Ground Acceleration (PGA) and the Effective Magnitude by classifying the roads depending on their level of importance.

It should be noted that the Bridge Manual requires cut slopes to be designed to a lower level of earthquake hazard, compared to other structures (bridges, retaining structures and embankments) along state highways. It is not clear why cut slopes are required to have a lower level of earthquake performance although failure of cut slopes can give rise to closure of important primary lifeline routes for six months or more (Mason and Brabhaharan, 2012), or as observed in the failure of slopes along State Highway 3 in the Manawatu Gorge, which was closed for more than 12 months.

Further, no allowance is made in the Bridge Manual for factoring seismic actions, either to allow for amplification (such as due to topographical effects) or reductions (to allow for incoherence of motions where the slopes are of significant height). Where slopes are to be designed for permitting displacement under earthquake loading, specific guidance is provided for engineered fills and retaining walls, and these same clauses are referred to for cut slopes.

The common engineering practice followed by the industry for slope stability analysis is to apply the provisions of NZS 1170.5:2004, with respect to the limit state requirements (Ultimate Limit State and Serviceability Limit State) and NZS 1170.5:2004 or the Bridge Manual for estimating the design actions. When simplified pseudo-static methods of analysis are followed, only the horizontal component of the seismic inertia forces are taken into account, without any factoring to account for topographic amplification near the crest or de-amplification near the base (Brabhaharan and Stewart, 2015). Similar assumptions are made when a displacement based design is carried out.

Rouvray et al. (2015) present a performance based design approach used for the design of the Transmission Gully motorway north of Wellington, New Zealand. This relied on the recurrence intervals specified for different serviceability and ultimate limit states in the current Bridge Manual (New Zealand Transport Agency, 2014). Toh and Swarbrick (2015) present a method of assessment of seismic loads considering topographic amplification. This method involves selection of a slope crest topographical amplification factor based on the slope angle and slope height

INTERNATIONALLY

A common element in the international design practice followed for pseudo-static slope stability analysis, is factoring of the horizontal seismic acceleration applied for the calculation of the equivalent acting force on the sliding mass. A factor of 0.5 is used in the Eurocode, of which we have not been able to clarify the background, and a factor feq is used in the Californian Code (Southern California Earthquake Centre, 2002).

Amplification of seismic waves due to topographic irregularities has generally received limited attention in the context of design codes and standards. Recommendations were found only in the French Code PS-92 (Jalil, 1992) and the Eurocode (European Committee for Standardization, 2004). Both codes have considered step-like slopes and refer to the topography effects as an amplification of the seismic motion at the upper part of the slope. The focus of the Eurocode is in providing topographical amplification for buildings built on slopes, near the crest or at some distance from the crest, rather than providing a method for assessing the effect of topographical amplification on the earthquake performance or design of the slopes themselves.

DEVELOPMENT OF DESIGN GUIDELINES

Draft guidelines for design of high cut slopes have been developed to aid design for transportation projects in New Zealand, as part of this research project. The concept of resilience is introduced to the new Design Guidelines. Resilience is the ability to recover readily and return to its original form from adversity. From an infrastructure and building perspective, this requires us to develop our built environment in a way that reduced damage and the consequent loss or reduction in its functionality, and the ability to recover quickly from such reduction. Brabhaharan et al (2006) adapt this concept for resilience for application to transportation networks as conceptually illustrated in Figure 2.

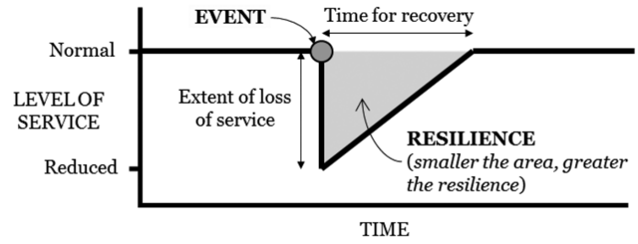


Figure 2: Resilience of route/network

Applying this to transportation routes, the cut slopes should be designed for resilience by adopting a design that has a low vulnerability to failures and closure of the route, by minimising the size and nature of failures, thus enabling the functionality of the transportation route to be restored quickly.

Classification of cut slopes in the new Design Guidelines is initially based on selecting the appropriate Importance Level of the route in order to select appropriate levels of earthquake shaking. The Importance levels (IL) proposed are those defined in AS/NZS 1170.0 supplemented by the Bridge Manual for Highways and arterial slopes. The aim of this selection has been to use the existing importance level framework in the New Zealand Standards and Bridge Manual.

A further classification of the cut slopes has been introduced to the guidelines, the Resilience Importance Category (RIC). It is recognised that the resilience expectations even for the same Importance Level of transportation route varies significantly depending on the regional context of the routes. For example IL3 routes in Christchurch or Auckland would have a lot more redundancy because of the many routes given the terrain, whereas in places like Wellington, Dunedin or Central Otago there is very little redundancy. Resilience Importance Categories have been developed to provide for incorporating the local context and resilience expectations into the design process.

Design Feature	New Proposed Guidelines	Existing Standards or Guidelines			
		NZS 1170	NZGS Module 1	Bridge Manual 3rd Edition	Eurocode
Importance Level	Uses NZS 1170 and Bridge Manual approach	Provides importance level.	Not addressed.	Importance levels based on NZS 1170.	-
Resilience	Resilience based design approach, importance earthquake motions, and performance level.	Not considered.	Not addressed.	Not considered.	Not considered.
Earthquake Motions	Consistent across all transportation structures.	Only addresses buildings.	Refers to Bridge Manual 2014 and NZS 1170.5:2004	Different earth-quake level for different components of road.	Provided.
Topographical effects	Topography Amplification Factor. Reduction of TAF along slope based literature or analysis.	Not provided for.	Not provided.	Not provided for.	Provided for buildings above slope.
Earthquake motions for pseudo-static slope design	Scaling of Ground Acceleration for deep seated failures, based on international practice.	Slopes not provided for.	No specific guidance for slopes.	No specific guid-ance.	Provides for arbitrary factor.

Table 3: Illustration of the developments introduced by the new design guidelines for cut slope design

A four-level design approach is presented, to suit the design importance level of cut slopes. A fundamental difference of the design approach given compared to that stipulated in NZS 1170 is that the design is for resilience rather than life safety alone. The different design approaches depend on the importance level of the transportation corridor, the resilience expectations for the route and the scale and complexity of the cut slopes.

The guidance also includes assessment of suitable topographical amplification factors for typical hilly terrain where highway cut slopes are commonly formed in New Zealand, and factoring of the peak ground acceleration for derivation of equivalent pseudo-static loads for use in design.

Table 3 provides a comparison of how the proposed guidelines address difference aspects of the design compared to existing standards. The proposed approach uses a novel resilience based approach to cut slope design, and addresses significant gaps in current design guidance for cut slopes.

CLOSURE

It is recognised that this is an area of recent research and development, and there is more research yet required to develop a good understanding of the topographical amplification issues. However, these guidelines will help bring up to date research and current knowledge to the design of transport infrastructure, at a time when major transportation infrastructure involving high cut slopes is currently happening or planned for the near future in New Zealand.

These draft guidelines provide an approach based on the research carried out, and may be converted into guidelines that are published for design use, and for eventual incorporation into design manuals such as the Bridge Manual, or into design standards.

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QuakeCoRE Flagship Programme: Liquefaction Impacts on Infrastructure

Misko Cubrinovski[1], Sjoerd van Ballegooy[2]

WE ARE DELIGHTED to present the ongoing research activities within QuakeCoRE, a recently inaugurated Centre of Research Excellence (http://www.quakecore.nz/). QuakeCoRE is a national network of leading New Zealand Earthquake Resilience Researchers. Clearly, geotechnical hazards play an important role in the earthquake resilience, and hence the research on seismic geotechnical hazards represents a key component of the Quake CoRE activities.

The QuakeCoRE research is organised into Technology Platforms, Research Themes and Flagship Programmes. There are six flagship programmes covering a wide range of topics across earthquake engineering and resilience science, from strong ground motions, geotechnical hazards, earthquake prone buildings, and next-generation infrastructure to societal resilience. The QuakeCoRE Flagship Programme 2 (Liquefaction Impacts on Infrastructure) focuses on soil liquefaction as one of the principal earthquake hazards affecting land and infrastructure of New Zealand. In this flagship programme, three thrust areas of research have been targeted in our initial research efforts: (i) advancement of liquefaction evaluation (beyond current state-of-practice) related to liquefaction triggering and its consequences; (ii) characterisation of cyclic behaviour and associated liquefaction vulnerability of specific New Zealand soils, and (iii) systems approach in liquefaction evaluation and its mitigation.

In the following, a brief overview of the 2016 research projects within Flagship Programme 2 are presented to give you some flavor of our initial research efforts. The QuakeCoRe activity will remain strong, and we hope to provide updates and regular input of this nature in the years to come, including some detailed research feature stories from our more significant outputs.

Misko Cubrinovski, Professor, Department of Civil and Natural Resources Engineering, University of Canterbury

Misko is a professor in Geotechnical and Earthquake Engineering at the University of Canterbury, Christchurch. His research interests and expertise are in geotechnical earthquake engineering and in particular problems associated with liquefaction, seismic response of earth structures and soil-structure interaction. Misko has had a leadership role in the research efforts following the 2010-2011 Christchurch earthquakes, especially around soil liquefaction and its impacts on land, buildings and infrastructure. Misko is the Leader of QuakeCoRE Flagship Programme 2.

1)University of Canterbury, 2)Tonkin & Taylor Ltd.

Evaluation of Liquefaction Potential of Pumiceous Deposits through Field Testing

R.P. Orense[1], L.M. Wotherspoon[1], M.J. Pender[1], S. van Ballegooy[2], M. Cubrinovski[3]

PUMICE MATERIALS, which are problematic from engineering viewpoint, are widely spread in many parts of North Island. Following the 2010-2011 Christchurch earthquakes, a clear understanding of their properties under earthquake loading is necessary. For example, the 1987 Edgecumbe Earthquake showed occurrences of localised liquefaction in sands of volcanic origin. The aim of this QuakeCoRE-funded research is to investigate the liquefaction resistance of in-situ pumice deposits through field testing, especially at sites where liquefaction was observed following the Edgecumbe earthquake. Using field-obtained data (CPT, Vs-profile), attempts will be made to explain the occurrence/non-occurrence of liquefaction at the target sites following the earthquake using available empirical chart-based approaches. The applicability of the current field-based empirical approaches in assessing the liquefaction potential of pumiceous deposits will be scrutinized vis-à-vis the observed liquefaction characteristics of pumice sands.

Pumice deposits are found in several areas of the North Island. They originated from a series of volcanic eruptions centred in the Taupo and Rotorua regions, called the "Taupo Volcanic Zone". The pumice material was initially deposited by eruptions; followed by erosion and river transport. Presently, pumice deposits exist mainly as deep sand layers in river valleys and flood plains, but are also found as coarse gravel deposits in hilly areas. Although they do not cover wide areas, their concentration in river valleys and flood plains means they tend to coincide with areas of considerable human activity and development. Thus, they are frequently encountered in engineering projects and their evaluation is a matter of considerable geotechnical interest.

Previous research at the University of Auckland showed that the penetration resistance (qc) values obtained from cone penetration tests (CPT) on pumice sand were only marginally influenced by the density of the material. The reason for this behaviour is possibly because the stresses imposed by the penetrometer are so severe that particle breakage forms a new material and that the properties of this are nearly independent of the initial state of the sand (Wesley et al. 1999). Thus, conventional relationships between the qc value and relative density, which in turn is correlated with liquefaction resistance, appear to be not valid for these soils.

Recent research showed that penetration-based approaches, such as CPT and seismic dilatometer tests, underestimated the liquefaction resistance of the pumice deposits (shown in Figure 1), confirming that any procedure where the liquefaction resistance is correlated with density will not work on pumiceous deposits (Orense et al. 2012; Orense

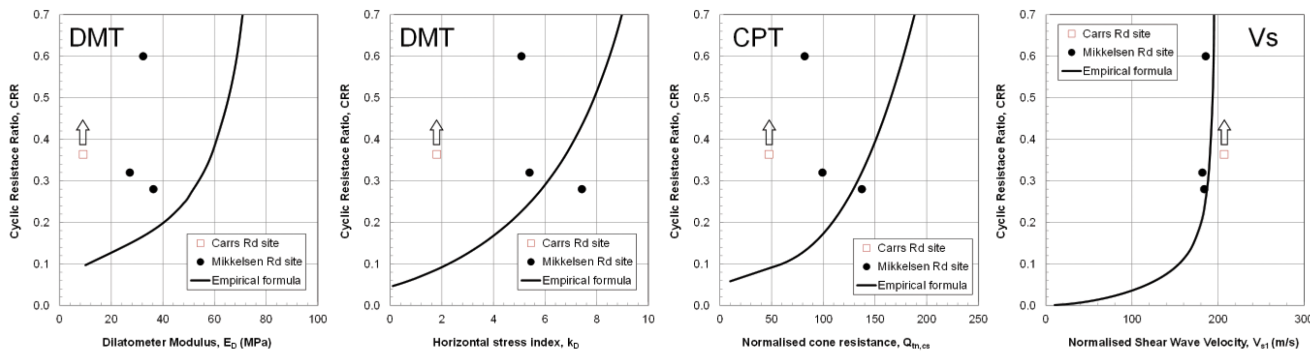

Figure 1: Comparison between laboratory-based and field-based cyclic resistance ratio (Orense et al. 2012; Orense & Pender, 2013).

Associate Professor Rolando Orense,
Department of Civil & Environmental Engineering, University of Auckland.
Rolando is the Geomechanics Group Leader at the Department of Civil & Environmental Engineering, University of Auckland. His research interest is earthquake geotechnical engineering, and he has extensive experience in doing research, teaching and consulting works related to soil liquefaction, ground response analyses and seismic soil-structure interaction.

1) University of Auckland, 2) Tonkin & Taylor Ltd., 3) University of Canterbury

Figure 2: Target sites for the study.

and Pender 2013). The same research showed that empirical method based on shear wave velocity seemed to produce good correlation with liquefaction. A possible reason is that the shear stresses during penetration were so severe that particle breakage formed new finer grained materials, the mechanical properties of which were very different from the original pumice sand. Admittedly, the above conclusions were obtained from limited number of test data and such conclusions have not been well-validated. With many consultants and practitioners constantly asking for advice on how to evaluate the liquefaction susceptibility of pumice deposits, there is indeed a need to clarify and address this issue.

RESEARCH OBJECTIVES

The key objectives of this research are as follows:

[1] Characterise various sites in the Rangitaki Plains by performing field testing at designated sites through cone penetration tests, shear wave velocity profiling and screw driving sounding tests.

[2] Assess the liquefaction potential at the said sites using available field-based empirical methods, considering estimated peak ground accelerations associated with the 1987 Edgecumbe earthquake.

[3] By validating the factor of safety against liquefaction vis-à-vis the observed performance (as evidenced by available literature and reconnaissance reports), provide recommendations on the best possible field-based method(s) to be used in evaluating liquefaction potential of pumiceous deposits.

It is envisioned that the field test results would be used in the next stage of research which would attempt to correlate the field parameters with the liquefaction resistance of pumiceous deposits derived from laboratory testing, in order to aid in the development of guidelines for evaluating liquefaction potential of pumiceous soils.

SELECTED TEST SITES

Two sites have been identified where liquefaction had been observed during the 1987 Edgecumbe Earthquake. Test site #1 is located adjacent to the Whakatane Sewage Pump station, while Test site #2 is opposite the Edgecumbe substation (see Figure 2). Manifestations of soil liquefaction, such as sand boils and ejected materials, have been reported at both sites (Pender et al. 1989).

To date, borehole sampling, cone penetration tests (CPT), seismic cone penetration tests (sCPT), and screw driving sounding (SDS) have been performed at these sites. Surface wave profiling is being planned. The results of the study will be published by the end of the year.

EXPECTED IMPACT

One of the issues that have been constantly brought to attention by the engineering community is the lack of guidance for geotechnical characterisation and evaluation of pumiceous soils. Existing empirical correlations based on hard-grained sands may not work for their characterisation and could mislead engineering assessment. In this context, the profession is facing serious issues on many large engineering projects especially in the Waikato – Bay of Plenty region in relation to liquefaction evaluation of pumiceous soils and associated significant costs for mitigating potentially non-existent liquefaction hazards.

It is envisioned that, in the short-term, the research output will assist the local profession through guidelines on how to evaluate liquefaction potential of pumiceous deposits based on field parameter(s). In the long-term, the research outputs will be combined with previous, on-going and future work to develop design guidelines for evaluating liquefaction potential of pumiceous deposits.

ACKNOWLEDGMENTS

Funding of the research project was through QuakeCoRE (Project # 16025). The assistance of Whakatane District Council, AECOM and GNS Science in determining the target test sites is gratefully acknowledged.

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Undisturbed Sampling of Pumiceous Materials in New Zealand

M.E. Stringer[1], R.P. Orense[2], M. Cubrinovski[1] & M. Pender[2]

PUMICEOUS DEPOSITS EXIST in many areas of the North Island, and pose a particular challenge to engineering projects due to the difficulties associated with characterizing these materials. Key characteristics of pumice grains are their high crushability, their low density and the presence of voids, both internally and on the surface of the grains. Each of these characteristics has important consequences for assessing the response of the material to different loading conditions. In particular, it has been shown by Wesley et al (1999) that the penetration resistance measured during a CPT sounding is insensitive to relative density in pure pumice material. This result clearly shows that extreme caution must be exercised when attempting to estimate engineering properties from correlations which have been developed for hard-grained materials. Additionally, element testing reported by Orense et al. (2012) has shown that there are some important differences between the behaviour of purely pumiceous material when compared with hard-grained materials of similar size.

While laboratory studies have shown clear issues in the characterisation of pumiceous soils, it is important to recognize that the previously described results apply to reconstituted samples of poorly graded pumice. Naturally occurring pumiceous deposits on the other hand are often well-graded, mixed with hard-grained materials, and affected by factors such as fabric, structure, ageing and stress history. As such, undisturbed sampling has a key role to play in establishing in advancing our understanding the behaviour of these materials as they exist in-situ.

Downhole undisturbed sampling of sandy material is notoriously difficult, and experience from Japan has shown that conventional samplers (i.e. push tubes and core-samplers) can significantly disturb samples such that they may no longer represent the in-situ characteristics of the soil (i.e. Yoshimi et al. 1994). With this in mind, the authors are attempting to establish the relative performance of three advanced sampling tools and conventional push-tubes in retrieving high quality specimens of pumiceous material.

The three "advanced" soil samplers being trialled in this study are the Dames and Moore (DM) fixed piston sampler, the Gel-Push Static (GP-S) fixed piston sampler and the Gel-Push Triple Tube (GP-TR) rotary sampler. These samplers have been used in a number of field trials in Christchurch since the 2010-2011 earthquakes to recover undisturbed samples of silty sands, with some promising results (i.e. Taylor et al. 2012, Stringer et al. 2015). Each of these samplers have incorporated features into their design to reduce the sidewall friction (thought to be a major source of sampling disturbance) which acts on the side of the soil specimen as it is captured within the tool; the DM sampler uses relatively short brass tubes, while the Gel-Push samplers coats the soil sample in a lubricating polymer gel.

A suitable site for trialling the samplers was located near the central business district of Whakatane. At this site, relatively thick, shallow pumice bearing layers were noted in a logged borehole. The pumice bearing materials could be separated into two representative cone penetration resistances: 2MPa and 6 MPa. Based on the experience with the field trials in Christchurch, this range in penetration resistance was deemed suitable for testing the performance of the different samplers.

The undisturbed sampling was carried out in late September by McMillan Drilling and a team of researchers from both the University of Auckland and University of Canterbury (Figures 1-3). A total of 4 boreholes (spaced 2m apart) were drilled to enable samples to be obtained with each sampler from identical depths below the ground surface. Following retrieval, the samples were allowed to drain overnight before being frozen and transported back to the university laboratories where an evaluation of sample quality will be undertaken in the near future (an

Dr. Mark Stringer, Senior Research Fellow, University of Canterbury

Mark is a Senior Research Fellow at the University of Canterbury in the Department of Civil and Natural Resources Engineering. Mark is currently researching the liquefaction resistance of New Zealand soils through element testing on undisturbed soil samples.

1) University of Canterbury, 2) University of Auckland

Figure 1: Preparing the GP-S sampler

Figure 2: Careful retrieval of soil samples

Figure 3: On-site extrusion of soil specimen. (for visual assessment only)

Figure 4: Example section of sample recovered using the GP-TR sampler (after freezing).

example section of a frozen GP-TR specimen is shown in Figure 4).

It is anticipated that this research will highlight the scenarios under which it may be possible to obtain high quality samples of pumiceous material using the samplers trialled in this study. This information will be used to optimise future sampling attempts in these soils, which in the short term will be beneficial in determining key engineering properties of specific deposits, while in the longer term, results can be combined to develop new frameworks for site characterisation based on simple field tests using tools such as the CPT.

In the coming years, the authors hope to carry out undisturbed sampling at a number of sites with pumiceous deposits as we seek to build a database of results. As part of this effort, we welcome engagement from our colleagues in industry who may be interested in performing detailed characterisation of pumiceous deposits on their specific projects.

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Effects of Partial Saturation on Liquefaction Triggering

Abdul Baki[1], Misko Cubrinovski[1], Mark Stringer[1], Sjoerd van Ballegooy[2]

THE SIMPLIFIED LIQUEFACTION evaluation methods reasonably predicted the severity and extent of liquefaction in Christchurch during the 2010-2011 Canterbury Earthquake Sequence, however there are important cases where moderate-to-severe liquefaction was predicted yet no liquefaction manifestation was observed. These high levels of conservatism have been identified as a major concern in the application of the simplified liquefaction evaluation methods. Detailed field investigations have indicated that some of these sites contain deposits at approximately 0.50 m to 1.50 m depth below the water table that are partially saturated (i.e. voids contain air bubbles or gas). These soil deposits typically contain layers of fines-containing sandy soils of low plasticity. This ongoing research aims to investigate the influence of partial saturation on liquefaction triggering including:

- Correlate liquefaction resistance with degree of saturation (S_r) for characteristic Christchurch soils including clean sands and silty sands.
- Incorporate the effects of saturation in a simplified procedures for liquefaction assessment.
- Provide basis for quantifying the effects of partial saturation in advanced seismic analysis.

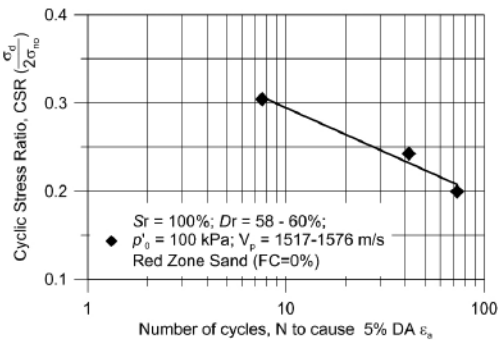
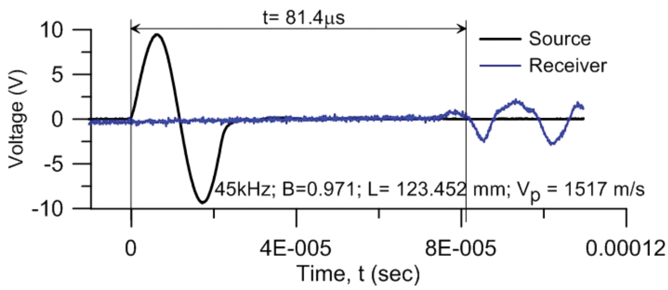


Figure 2: Liquefaction resistance curve of tested red zone sand

Three types of typical Christchurch soils have been identified for cyclic triaxial testing at three different levels of Sr (including a fully saturated soils). A laboratory methodology has been developed at the Geomechanics Laboratory, UC, to measure and correlate laboratory and in-situ S_r through P-wave velocity, V_p (Figure 1). The liquefaction resistance curve of a fully saturated (S_r = 100%) red zone sand (Fines content, FC=0%) is presented in Figure 2. For the given soil type and testing condition, this curve will be used as a reference when quantifying liquefaction resistance at partially saturated condition. The outcomes of this project will help to improve state-of-the-practice in engineering design and evaluation of NZ liquefaction hazard. This project is funded by QuakeCoRE (project #QC009/16013)



Above: a) modified triaxial platens and bender elements for Vp measurement; b) An example of Ep measurement (residential red zone sand, Sr = 100%)

Dr. M.A.L. Baki, Postdoctoral Fellow, University of Canterbury

Baki is a Postdoctoral Fellow at the University of Canterbury. He completed PhD from University of New South Wales, Australia on cyclic liquefaction behaviour of granular materials. He has a particular interest in mechanism and evaluation of soil liquefaction, site characterisation, laboratory and in-situ testing of soils. abdul.baki@canterbury.ac.nz

1 University of Canterbury, 2Tonkin & Taylor Ltd,

Geologic and geomorphic influence on the spatial extent of lateral spreading in Christchurch, New Zealand

Sarah Bastin[1], Misko Cubrinovksi[1], Sjoerd van Ballegooy[2], James Russell[2],

LATERAL SPREADING OCCURRED proximal to waterways during the 2010 to 2011 Canterbury Earthquake Sequence (CES) resulting in severe damage to overlying infrastructure. Published empirical and semi-empirical models for predicting lateral spread displacements have been shown produce displacement that varied by a factor of <0.5 to >2 from those measured in parts of Christchurch. Post-CES studies have indicated that the spatial distribution and extent of lateral spreading was strongly influenced by geomorphic and topographic features. These features are not explicitly accounted for in the current predictive models and likely account for some of the discrepancy between predicted and measured displacements. This study aims to examine the influence of geomorphic features on the spatial extent of lateral spreading and associated ground movement for a study area proximal to the Avon River.

Extensive LiDAR and satellite derived horizontal displacement datasets are available following the main CES earthquakes. These datasets provide an indication of the extent of lateral spreading and associated ground movements. However, each dataset has inherent limitations including spatial resolution, accuracy, and acquisition errors (e.g. flight line offset). It is therefore impossible to accurately derive the maximum extent of lateral spreading from a single dataset. The summation of measured crack widths, LiDAR derived ground subsidence, documented land damage, and cracking observed from post-event aerial photography provide additional information on the extent of lateral spreading.

In this study, the maximum extent of lateral spreading was derived by combining observations from the CES datasets. Relatively narrow zones of lateral spreading ranging from 0 to 300 m were identified and associated with maximum horizontal movements between 0 and 2.5 m. Zones were spatially correlated and with geomorphic maps (Figure 1). The width of these zones is shown to be strongly influenced by local geomorphic features; spreading is shown to terminate adjacent to paleo-cut banks and terrace risers irrespective of distance from the free-face. In comparison, areas underlain by point-bar deposits exhibit high horizontal displacements, while in other areas underlain by recent fluvial sediments spreading appears to gradually decay inland. Detailed geotechnical characterization of the subsurface sediments will be undertaken within each geomorphic area to identify prevalent soil types, stratification, and the thicknesses and lateral extent of critical layers. Soil types in areas on either side of the spreading extent will also be examined to determine factors influencing the extent.

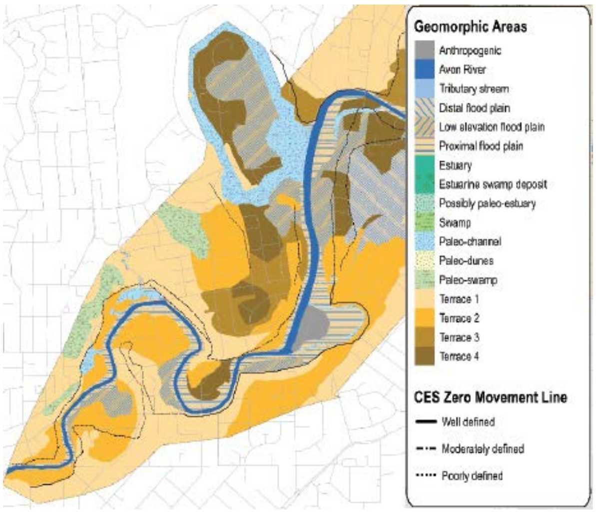


Figure 1: Geomorphic map of the study area overlain with the derived maximum extent of lateral spreading

This work aims to enhance engineering evaluation of lateral spreading, based the exceptional datasets available from the CES. The results will be incorporated into the revised geotechnical guidelines for assessing lateral spreading. This project is funded by QuakeCoRE (project #E6471/16056)

Dr. Sarah Bastin, Postdoctoral Fellow, QuakeCoRE

Sarah is a Post-doctoral Research Fellow for Flagship Programme 2. She completed her PhD at the University of Canterbury where she conducted paleo-liquefaction studies in Christchurch and examined geomorphic influences on the extent of lateral spreading. Her current work looks at verifying liquefaction hazard assessments using historical distributions of liquefaction, and continues to examine geomorphic influences on lateral spreading.

1 University of Canterbury, 2Tonkin & Taylor Ltd,

Whakatane liquefaction case history from the 1987 Edgecumbe Earthquake: examination of an extensive CPT dataset supplemented by paleo-liquefaction investigations

Sjoerd van Ballegooy[1], Sarah Bastin[2], Nick Mellsop[1], Liam Wotherspoon[3], Rolando Orense[3]

LIQUEFACTION AND ASSOCIATED LATERAL SPREADING during the 1987 M_L 6.3 Edgecumbe earthquake caused severe damage to parts of Whakatane, New Zealand. Recent studies utilizing extensive CPT investigations indicate that much of the Whakatane Central Business District is underlain by sediments with a low cyclic resistance to liquefaction. However, no evidence of liquefaction was observed in this area following the Edgecumbe earthquake. In this project, evaluation of the extensive existing CPT dataset was undertaken using estimated PGA for the Edgecumbe earthquake and modelled groundwater depths to examine the extent of liquefaction predicted from the CPT-based simplified methods. The predicted Liquefaction Severity Index (LSN) parameter at the CPTs was then compared with that documented from historical reports and resident recollections (Figure 1).

Lateral-spreading fissures associated with the 1987 Edgecumbe earthquake were exposed in trenches at sites known to have liquefied (the yellow shaded areas in Figure 1). The fissures cross-cut fluvial stratigraphy predominantly composed of fine-sand with thin lenses of medium sand, and were sourced from silty fine sand (Figure 2). No evidence of pre-historic liquefaction was observed, indicating that the sediments had not liquefied since their deposition and prior to the Edgecumbe earthquake.

Trenching at sites where severe liquefaction manifestation was predicted, yet not observed, revealed sediments comprising fine sand to silt interbedded with medium to coarse sand with pumice granules. The CPT investigation tool does not capture fine-scale inter-layering, nor does it discriminate between the coarse grained sandy soils with or without pumice granules, this may account for some of the variability between predicted and observed liquefaction. No liquefaction features were observed in

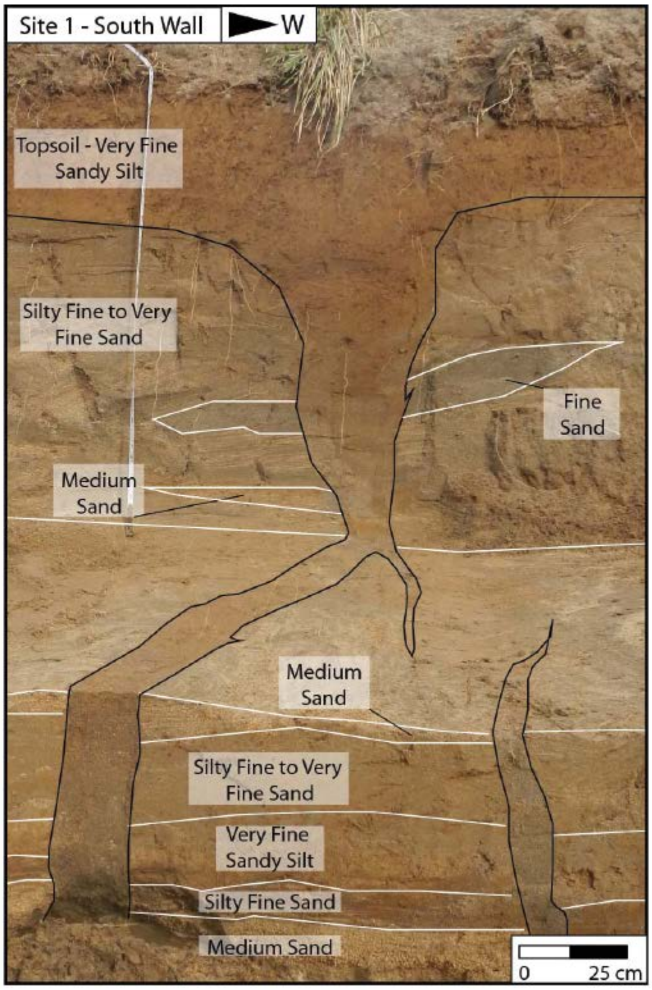


Figure 1: LSN calculated from PGA estimated for the 1987 Edgecumbe earthquake. Documented extents are indicated by black and yellow.

Sjoerd van Ballegooy, Ph.D., Technical Director, Tonkin + Taylor Ltd, New Zealand

Sjoerd is a technical director at Tonkin + Taylor. He has been involved in the geotechnical response to the liquefaction damage caused by the 2010 – 2016 Canterbury earthquakes and has been deeply involved in research to predict the consequences of liquefaction. One of the key projects led by Sjoerd is the architecture and development of the New Zealand Geotechnical Database to pool and disseminate geotechnical investigation data to the wider engineering community. Sjoerd is the Deputy Leader of QuakeCoRE Flagship Programme 2.

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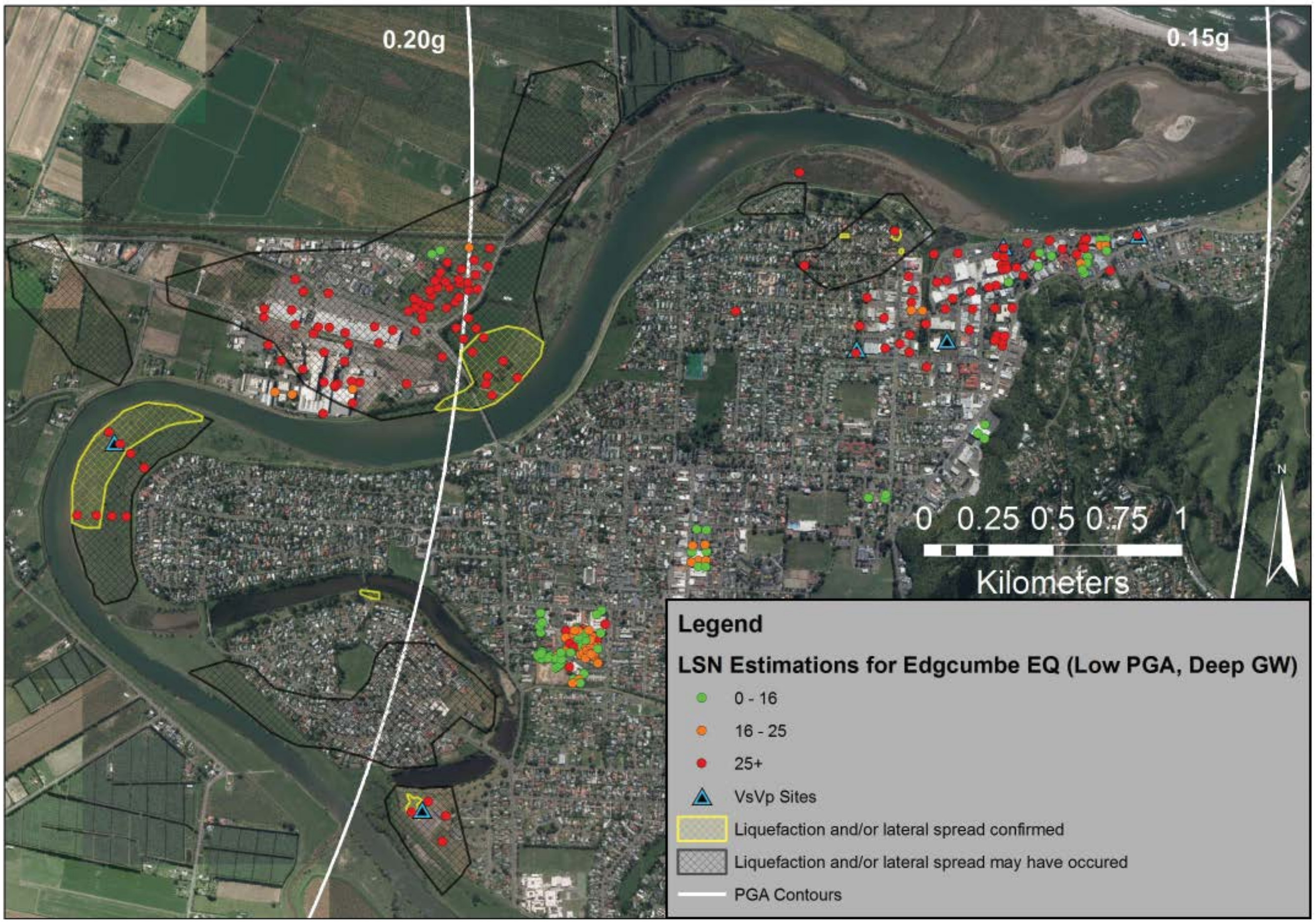

Figure 2: Interpreted field photograph of an exposed lateral spreading fissure.

the subsurface, indicating that the sediments have not liquefied since their deposition. The interlayered coarser sediment provides high hydraulic connectivity between the fine sand layers, which may enable pore-water dissipation and thus may be another reason for the variability between predicted and observed liquefaction.

This study highlights potential reasoning for the inconsistencies between predicted and observed liquefaction in Whakatane for the 1987 Edgecumbe earthquake. Future work will include cyclic simple shear testing of reconstituted bulk samples from the trenches to further examine the liquefaction resistance of these sediments.

This project is funded by QuakeCoRE (project #16060).

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Comparisons Between Deterministic and Probabilistic Liquefaction Assessment Approaches Over the Christchurch Area

V. Lacrosse[1], S. van Ballegooy[1] and B.A. Bradley[2]

LIQUEFACTION ASSESSMENTS ARE commonly undertaken by geotechnical engineers using a deterministic approach. This approach does not appropriately take into account the significant uncertainties associated with a liquefaction assessment and can potentially compound the conservatism that is introduced when selecting upper bound input parameters. Therefore, the deterministic assessment approach can be conservative and the expected performance poorly understood. This research project looks at an alternative approach for liquefaction assessments – a probabilistic assessment.

Because liquefaction consequences are considered at discrete levels of ground shaking, discrete groundwater levels and a single percentile of the triggering correlation, it is not possible to quantify the actual likelihood of a specific level of liquefaction consequence being exceeded. In order to overcome these problems, it is necessary to consider the uncertainty in the liquefaction triggering correlations, the seasonal variation in groundwater levels as well as the ground motions hazard curve (which defines the likelihood of certain levels of ground shaking). This is achieved by undertaking a full probabilistic assessment which incorporate the uncertainty for all these variables.

Figure 1 presents an example of a liquefaction assessment using the Liquefaction Severity Number (LSN) for the three CPT using the deterministic approach (with upper bound input parameters) and the probabilistic approach. The differences between the deterministic LSN value and the probabilistic LSN distribution for the three CPT are noteworthy. For CPT A, the deterministic LSN value represents the 87th percentile of the probabilistic distribution, for CPT B, the deterministic LSN value represents the 93th percentile of the probabilistic distribution whereas for CPT C, the deterministic LSN value represents the 70th percentile of the probabilistic distribution.

This research work aims to improve current engineering practice for assessing liquefaction. Using a probabilistic approach enables the engineer to better understand the uncertainty of the expected performance of a site and avoids the potential for compounded conservatism.

This project is funded by QuakeCoRE (project #16043).

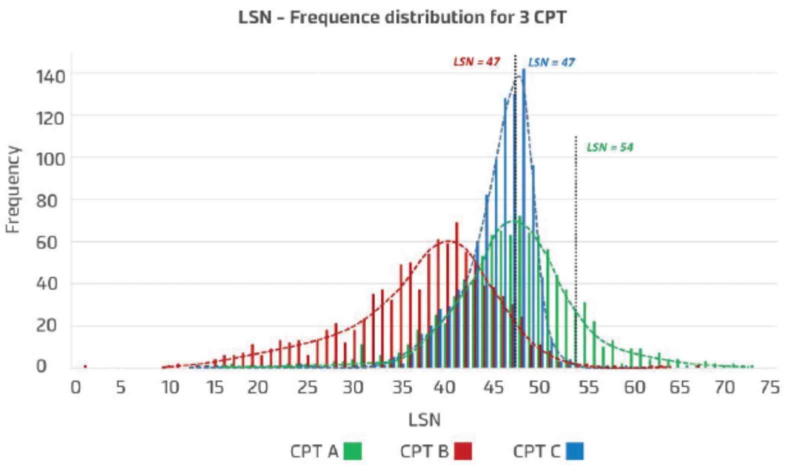


Figure 1: A graph showing the comparison between the deterministic and probabilistic liquefaction assessment approaches for three different CPT in Christchurch.

Virginie Lacrosse, MEng (Hons), Tonkin + Taylor Ltd, New Zealand

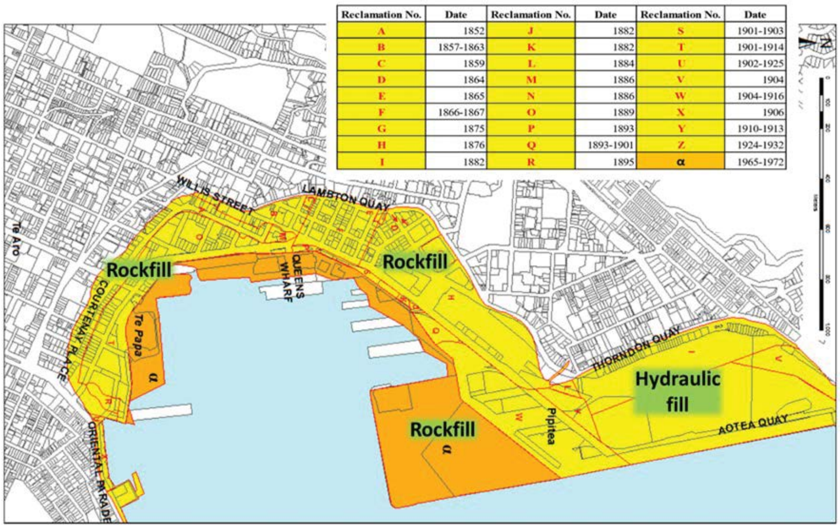
Virginie is a Natural Hazard Engineer who has been working with Tonkin + Taylor since 2011. Her key skills involve detailed liquefaction triggering assessment, modelling and interpretation, project management and hazard mapping. She works with IT and GIS teams to develop geospatial and technical tools to facilitate work being undertaken by engineers.

1Tonkin & Taylor Ltd, 2University of Canterbury

Characterisation of cyclic behaviour and liquefaction resistance of Wellington reclamations gravelly soils

Gabriele Chiaro[1], Merrick Taylor[2], Stuart Palmer[3], and Liam Wotherspoon[4]

Wellington Reclamations (Wellington Harbour Borad1936; Murashev and Palmer 1998; Wellington Waterfront Limited 2009; Semmens 2010)



Reclamation No.	Date	Reclamation No.	Date	Reclamation No.	Date
A	1852	J	1882	S	1901-1903
B	1857-1863	K	1882	T	1901-1914
C	1859	L	1884	U	1902-1925
D	1864	M	1886	V	1904
E	1865	N	1886	W	1904-1916
F	1866-1867	O	1889	X	1906
G	1875	P	1893	Y	1910-1913
H	1876	Q	1893-1901	Z	1924-1932
I	1882	R	1895	α	1965-1972

GRAVELLY SOIL (i.e. gravelly sands, sandy gravels, and uniform gravels) is generally recognized to have no or very low liquefaction potential. However, historically few case histories exist where observations of liquefied gravelly soils have been made, e.g. 1983 Borah Peak, Idaho earthquake (Youd et al. 1985), 1993 Hokkaido earthquake (Kokusho et al. 1995), 1995 Kobe earthquake (Soga, 1998) and 2013 Cook Strait earthquake (Van Dissen et al., 2015).

Since there are very few well-documented case histories of liquefied gravelly deposits in New Zealand, research in studying the liquefaction mechanism and developing proper analysing techniques for gravelly soils is necessary to characterise the hazard presented by these materials, so that engineers may effectively and economically minimize damage and loss caused by liquefaction of saturated gravelly soils. The results of this study will be valuable for characterising the impact and consequences of liquefaction on land and critical infrastructure during expected severe earthquakes, not only in the case of the Wellington reclamations but also for many critical infrastructure assets across New Zealand (dams, levees, bridge abutments/approaches, building foundations).

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Dr Gabriele Chiaro, Lecturer in Geotechnical Engineering, University of Canterbury

Gabriele's research interests include earthquake geotechnical engineering and related problems; constitutive modelling for geomaterials; development of advanced laboratory and field testing devices; geo-hazard reconnaissance and mitigation; reuse and recycling of industrial granular wastes as sustainable geomaterials.

For more detail regarding Dr Gabriele Chiaro visit his personal website: https://sites.google.com/site/chiarogabriele/

[1]University of Canterbury; [2]Arup, Auckland; [3]Tonkin and Taylor, Wellington; [4]University of Auckland

Dynamic Characteristics of Auckland Central Business District Reclaimed Zones

Kuanjin Lee, ENGEO Ltd, Liam Wotherspoon, University of Auckland

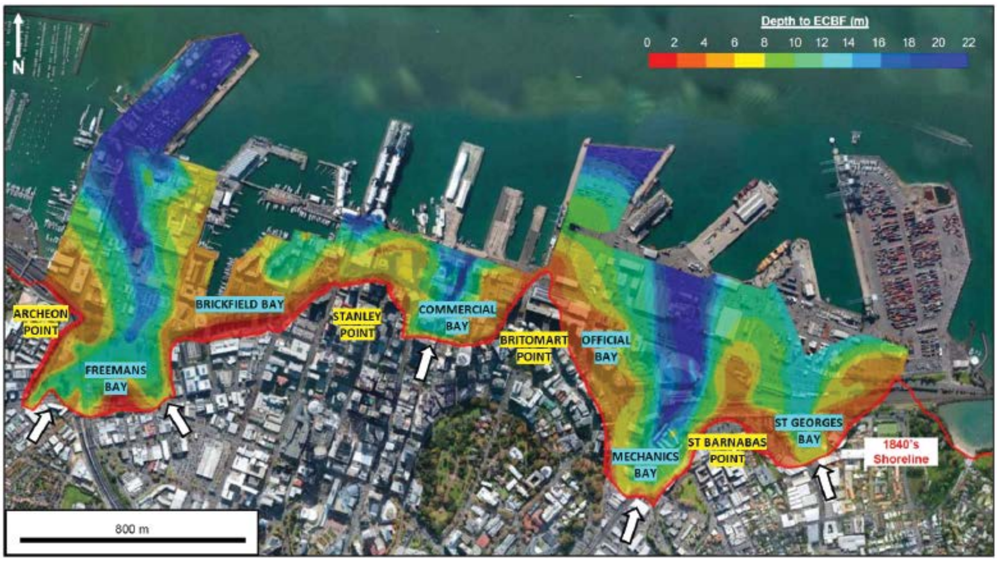
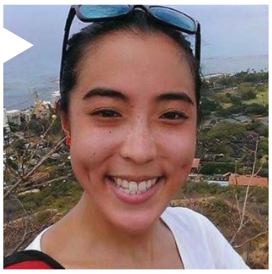


Figure 1: Auckland's reclamation zones are north of the original 1840's shoreline (red). Map shows historic bays (light blue), headlands (yellow), paleo-river/stream channels (white arrows showing direction of flow). The surface represents the depth to unweathered ECBF based on subsurface site investigation data and geomorphic cues. Zones without contours did not have enough investigation data to develop surfaces.

INTRODUCTION

The central Auckland City waterfront has been progressively modified since 1840, through a series of reclamation fills. Located on the northern edge of an isthmus, the reclamations served to increase the area of available land, catering for an increasing population. The aim of this study was to define the dynamic characteristics of the reclaimed land in the Auckland City Business District (CBD) using a combination of geotechnical, geological and geophysical data. The objectives were to (1) understand the history of reclamation and geology from historical ground investigations to provide constraints for the surface wave testing; (2) define shear wave velocity (Vs) of the deposits in the reclamation zones and; (3) define a range of site classification metrics across the region.



Kuanjin Lee

Jin holds a conjoint Bachelor of Arts and Science at the University of Auckland majoring in Geography, Geology and Environmental Science. She has since completed her Honours and Masters in Engineering Geology. In 2015, Jin came first in the NZGS Student Poster Competition. She is an Engineering Geologist at ENGEO Ltd in Auckland.

Liam Wotherspoon

Liam Wotherspoon is a Senior Lecturer in the Department of Civil and Environmental Engineering at the University of Auckland. He sits on the leadership teams of QuakeCoRE and the Resilience to Nature's Challenges research programmes, and is a Management Committee member for the New Zealand Society for Earthquake Engineering.

RECLAMATION HISTORY

Reclamations began in the bays closest to the shoreline using local East Coast Bays Formation (ECBF) material from nearby headlands, with areas such as Commercial Bay, Brickfield Bay, Official Bay and Mechanics Bay (shown in Figure 1) using material excavated from Stanley Point and Britomart Point. From the early 1900's, having depleted local sources, there was a shift to use hydraulic fill sourced from the Waitemata Harbour, including Wynyard Quarter and areas of Britomart Transport Centre. In the late 1980's and early 2000's, mudcrete was used to extend reclamations in the Viaduct Harbour and eastern areas of Wynyard Quarter. Across the reclaimed zones these fills overlie Tauranga Group Alluvium (TGA), which thickens in paleo-channels above the ECBF sandstone and siltstone bedrock.

METHODS

Site Investigation data and geomorphic characteristics were used to develop representative soil profiles in each reclamation zone and develop surfaces for the top of the unweathered ECBF and the base of the reclaimed deposits. Historical subsurface data across the region from a number of sources was used in this process. In addition, over 100 horizontal-to-vertical spectral ratio (HVSR) measurements were made to provide an estimate of the site period of the reclaimed soil profiles above the ECBF, and identify contrasts between adjacent reclamation zones. At a selected number of sites, Multi-channel Analysis of Surface Wave (MASW) testing was performed to provide an estimate of the Vs of the deposits. The layering of the profiles developed in this process were calibrated using collated subsurface stratigraphy.

RESULTS

1. Collated Subsurface Investigations

Subsurface investigations collated across the region revealed that the soils vary greatly across and within each of the reclamation zones. Sites proximal to the 1840's shoreline generally contain shallow (0 – 4 m) fill and TGA deposits. However, collated subsurface investigation data and geomorphic cues show ECBF incised paleo-channels ~ >20 m depth across the reclaimed zones, as shown by the blue regions in Figure 1. In areas such as Britomart (previously Commercial Bay), the stratigraphy is highly layered and has sharp changes in both composition and stiffness relative to depth.

2. Site Period and Shear Wave Velocity Estimates

HVSR and Vs profile derived site period estimates above the unweathered ECBF provided an estimate of the fundamental period of the soil profile above rock. In general, the longest periods were identified in areas furthest offshore from the 1840's shoreline and within paleo-channels. In general, the site periods showed a good correlation to the depth to rock contours, as site period increases with increasing depth to rock. Additionally, there were no clear differences in the relationship between site period and depth across the different reclaimed zones using different reclamation materials.

MASW testing showed that the upper TGA deposits have lower Vs than most of the reclaimed materials, and only slightly higher velocities than the hydraulic fill. Given this is the dominant deposit, this seems to have the largest influence on the site period across this region.

3. Site Subsoil Class

The focus of this study was the use of site period to classify site subsoil class according to NZS1170.5:2004, to delineate between site subsoil class C and D (using a site period of 0.6 seconds above the ECBF). The majority of the reclaimed region is defined as site subsoil class C, with site subsoil class D sites located proximal to paleo-channels in the area containing thickened deposits of alluvial material and in reclaimed areas that extended further out into the harbour. A more extensive site investigation programme is needed to identify potential site subsoil class E sites (with greater than 10 m of material with a Vs <150 m/s) and further assessment to identify areas susceptible to liquefaction.

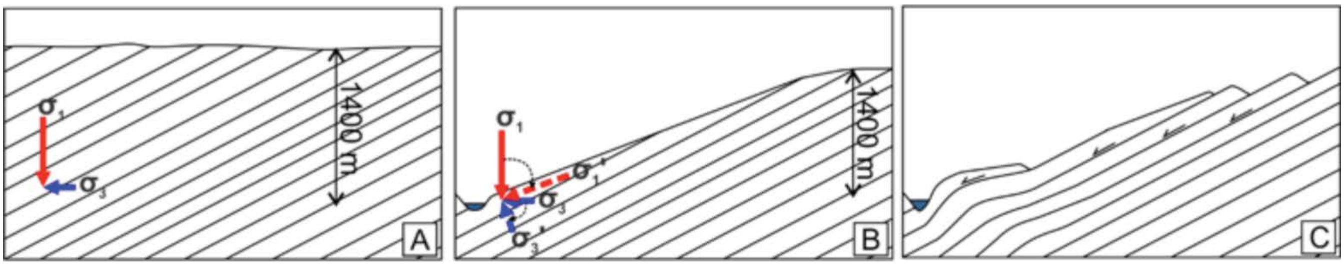
CONCLUSIONS

- Across the highly variable reclaimed deposits in central Auckland site subsoil class C is the most prevalent based on site period, with site subsoil class D sites located proximal to paleo-channels and in reclaimed areas that extended further out into the harbour.
- Comparisons across all reclaimed zones suggest that the underlying young, unconsolidated TGA deposits have more influence on the site period than the various materials used for reclamations.
- The HVSR method was shown to be a valuable technique for providing a rapid assessment of the depth to the ECBF deposits in the region.

ACKNOWLEDGEMENTS

Gratitude and thanks go out to the following for their support and aid in field investigations: André Bellvé, David Elliott, Jamie Lai, David Bevan and Shannon Hawkins.
For their patience and sharing of expertise: Simon Nelis and Jennifer Eccles. Additionally, thanks to Tonkin and Taylor and OPUS for allowing access to their investigation database and the New Zealand Geotechnical Database

Toe buckling deformation in Central Otago Schists



1. INTRODUCTION

Toe bucking deformation is not a widely familiar mechanism within the field of rock mechanics. Case studies, however, indicate that this type of deformation has been identified globally in a wide range of environments as a result of anthropogenic infrastructure such as road cuttings and open pit mines (Hu and Cruden 1993; Stead and Eberhardt 1997) and natural large scale landslides associated with deep seated gravitational slope deformations (DSGSD) (Agliardi et al. 2001). In some instances, toe buckling has triggered catastrophic landslides, such as during the 1999 Chi Chi earthquake in Taiwan (Wang et al. 2003). It is therefore important for hazard management and civil engineering design applications to identify and understand the major parameters governing this type of deformation mechanism.

Research in the form of a PhD project is currently in progress, focusing on toe buckling deformation in a New Zealand context. The study is based on three large scale landslides in Cromwell Gorge, Central Otago, which exhibit toe buckling deformation beneath their basal failure zone. Kinematic instability of these slopes are considered to be related to toe buckling deformation. It is anticipated that mass movement was triggered in response to breakout at the toe buckled hinge zone. This theory is being verified through numerical back analysis of the evolution of Cromwell Gorge.

Figure 1: a) and b) Incision of the valley induces a progressive rotation of principal stress axes (σ_1, σ_3); c) once accumulated stress concentrated at the base of the slope exceeds the strength of the anisotropic rock, toe buckling deformation develops in the form of oversteepened foliations.

2. WHAT IS TOE BUCKLING DEFORMATION?

Toe buckling deformation is associated with the development of slopes either through anthropogenic excavation or via natural processes such as uplift and erosion, where removal of the overburden induces the localised rotation of principal stress axes, as demonstrated in Figure 1. Differential stresses concentrated at the base of the slope can lead to the outward expression of strain in the form of toe buckling deformation, where stresses exceed the strength of an anisotropic rock mass. Variations of toe buckling deformation comprise curvilinear flexural buckling and three hinge buckling (see Cavers, 1981).

3. LOCATION AND BACKGROUND

Toe buckling deformation has been identified beneath the slide bases of several large scale landslides in both the Kawarau and the Cromwell Gorges located in Central Otago, New Zealand (Beetham et al. 1991b). This research focuses on the landslides within the Cromwell Gorge due to the detailed geotechnical database available from the Clyde Hydropower Project. Between 1975 and 1992, the Cromwell Gorge was subjected to extensive geotechnical

Romy Ridl

Romy is a PhD student at the University of Canterbury. Prior to returning to university, Romy worked as an engineering geologist for six years on a wide range of international geotechnical projects based in Australia, Madagascar, Malawi, Mozambique, South Africa and Zambia. She gained geotechnical experience on a variety of projects including investigations for railways, roads, pipelines, bridging structures, critical facilities, industrial, commercial and residential developments.
romy.ridl@pg.canterbury.ac.nz

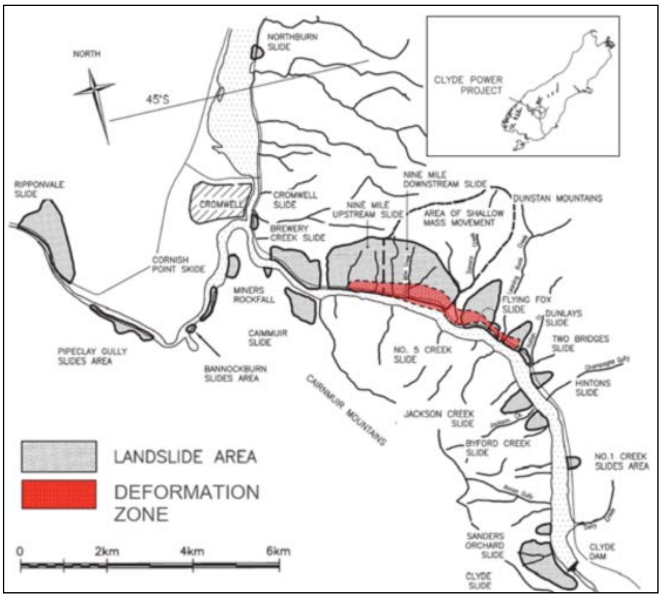
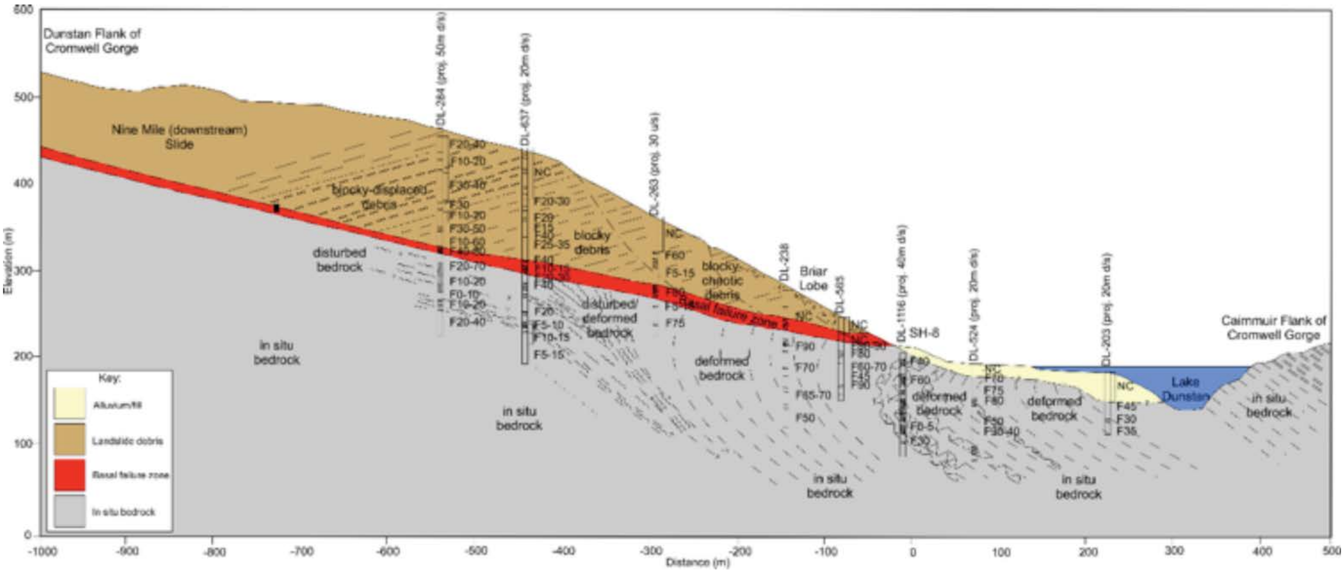


Figure 2: Locality map of Cromwell Gorge depicting approximate locations of major landslides within the Lake Dunstan catchment (modifed from Macfarlane 2009). The zone of observed toe buckling deformation is highlighted in red.



works whereby seventeen major landslides were identified and subsequently stabilised in the Lake Dunstan Catchment (Macfarlane 2009) (see Figure 2).

The results of these extensive geotechnical investigations indicated that in general, the Cromwell Gorge comprises Haast Schist (textural grade IV) bedrock, which is inclined at an average angle of 20° to 40° towards the southwest. Anomalous, oversteepened to overturned foliations were identified beneath the slide bases of three landslides, namely Nine Mile, No. 5 Creek and Dunlays Slides (Beetham et al. 1991b). This zone of deformation was identified along the Dunstan Flank of the gorge as highlighted in Figure 2. Bedrock in this area is deformed with schistosity exhibiting flexural buckling steepening upwards toward the slide base at angles from 50° to 90° to completely overturned (Figure 3).

Figure 3: Inferred geological cross section of Nine Mile (Downstream) Slide illustrating deformed schist directly underlying the basal failure zone comprising oversteepened to overturned foliations (modified from Beetham et al. 1991a).

4. NUMERICAL MODELLING

Toe buckling deformation identified along the Dunstan Flank is considered to be a result of time-dependent gravitational creep deformation in response to progressive incision of the Cromwell Gorge. Finite Element Method (FEM) numerical models were used to model the anisotropic nature of the schist rock mass by incorporating an elasto-plastic Mohr-Coulomb failure criterion with an explicit joint network. A sequential unloading method (Figure 4) was adopted to simulate valley evolution commencing from a relatively low relief surface, corresponding to the Otago Peneplain, progressively incising through uplift and erosion to present day topography (1400 m deep valley). A summary of the FEM input parameters and model considerations are provided in Table 1.

Boundary conditions	Sequential unloading based on typical slope morphologies and suggested exhumation rates (see Figure 4).
Field stresses	In situ crustal stresses incorporating gravitational and tectonic stresses. Far field tectonic stresses were derived from comparisons between FEM simulations and in situ field stress measurements.
Material properties	Mohr-Coulomb elasto-plastic rock strength parameters derived from extensive laboratory test data conducted during PhD research (UCS, triaxial, indirect tensile, point load and shear box).
Method of analysis	Sensitivity analysis consecutively varying one parameter per model. 83 FEM models processed to date.

Table 1: Summary of input parameters and model considerations.

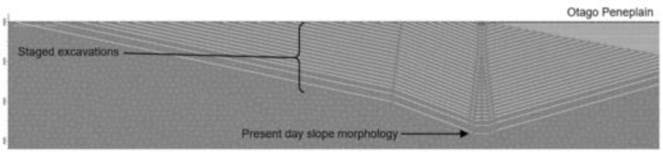


Figure 4: Sequential unloading method adopted in FEM model where staged excavations correspond to suggested exhumation rates for the Cromwell Gorge commencing from a low relief surface (Otago Peneplain) progressively incising to the present day topography (~1400 m deep valley).

5. PRELIMINARY RESULTS

Preliminary results suggest that the major parameters which governed the development of toe buckling deformation in the Cromwell Gorge are the combination of topographically induced gravitational stresses, high far-field active horizontal tectonic stresses, foliation stiffness and orientation. Flexural "z-shaped' folds (toe buckling) develop in a shear band along a linear zone of abrupt principal stress orientation change. This shear band, shown in Figure 5, develops at the intersection of topographically induced gravitational stresses localized proximal to the slope (red stress trajectories) and the far field horizontal tectonic stresses which dominate along the base of the slope (black stress trajectories).

The anisotropic nature of the schist plays an integral part in the development of toe buckling. Numerical simulations reveal that 'z-shaped' folds (toe buckling) do not develop where foliations are orientated sub horizontally to gently inclined (10-20°) or sub vertically inclined (80°). In addition, a certain magnitude of foliation competency (stiffness) is required to facilitate buckling. At low assigned shear and normal stiffness, the material merely yields in translation along preferential planes of weakness (foliation shears). Whereas, at higher stiffnesses, foliations facilitate plastic deformation by withstanding complete dislocation, and deform in a curvilinear manner (i.e. 'z-shaped' buckle folds).

6. CONCLUSIONS

Back analysis of toe bucking deformation in the Cromwell Gorge using FEM numerical simulations of valley evolution indicates that oversteepened foliations at the base of the Dunstan Flank are a product of prolonged stresses acting on an anisotropic rock mass. Preliminary

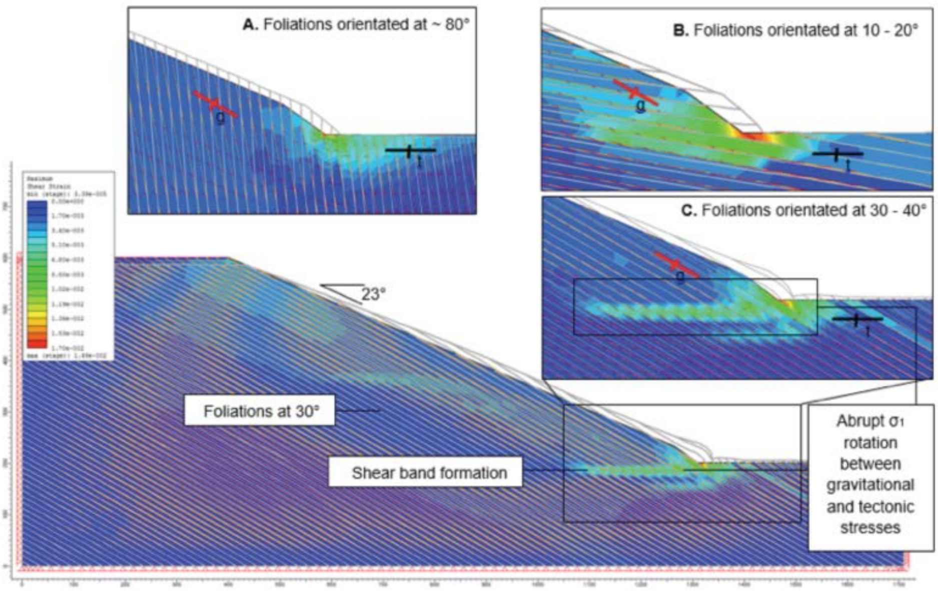


Figure 5: Foliations deform via flexural buckling developed along bands of maximum shear strain due to localised rotations of principal stress axes between topographically induced gravitational (red stress trajectories) and far field tectonic principal stress (black stress trajectories) orientations. Note the height of the slope was scaled from 1400 m to 400 m for model simplification.

results reveal that dominant contributing parameters to this style of deformation are topographically induced gravitational stresses, high horizontal tectonic stresses and the anisotropic nature of schist (foliation orientation and stiffness).

Moving forward with this research, further geotechnical laboratory tests will be carried out on Rakaia Terrane Schists (a subdivision of Haast Schist) as preliminary research highlighted limited availability of published geotechnical data. Additional numerical simulations incorporating new laboratory data as well as creep constitutive models will be processed. Furthermore, structural mapping and the implications of potential imbricate nappe-like structures associated with the juxtaposition of the Caples Terrane over the Rakaia Terrane will be considered. Any comments or points of consideration for this research will be welcomed. Please contact Romy Ridl on romy.ridl@pg.canterbury.ac.nz

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Fifteenth Geomechanics Lecture Geotechnical Issues in Displacement Based Earthquake Design of Highway Bridges and Walls: Part 2 - Erratum

John Wood

PARAMETER	WOOD, 1973 AND 1991	VELETSOS AND YOUNAN, 1997
Method	Theory of Elasticity using finite element analysis.	Approximate analytical Theory of Elasticity: Vertical normal stresses = 0 Horizontal variation of vertical displacement = 0
Wall contact	Smooth and perfectly bonded.	Limited to perfectly bonded by approximate theory.
Poisson's ratio Soil	$\nu = 0.3$	$\nu = 0.333$
Relative flexibility of wall and soil for: Fixed base flexure in stem.	$S = \dfrac{E_s H^3}{E_w I_w}$ Where, E_s and E_w = Young's modulus for soil and wall respectively; I_w = second moment of area for wall.	$d_w = \dfrac{G H^3}{E_w I_w}$ Where G = shear modulus of soil
Relative flexibility of wall and soil for: Rotating base rigid stem walls.	$R = \dfrac{K_{mr}}{K_{mf} + \dfrac{R_\theta}{E_s H^2}}$ Where K_{mr}, K_{mf} are rigid and forced wall dimensionless moment coefficients respectively (approximately 0.6 and 0.4). R_θ is the rotational stiffness of the base.	$d_\theta = \dfrac{G H^2}{R_\theta}$

Table 1: Assumptions for Cantilever Wall Analysis

Fifteenth Geomechanics Lecture Geotechnical Issues in Displacement Based Earthquake Design of Highway Bridges and Walls: Part 3 – John Wood

John Wood is a consulting civil engineer specialising in bridge design, structural investigation, soil-structure interaction and earthquake engineering. Before setting up his consulting engineering practice in 1986, he wasHead of the Ministry of Works, Central Laboratories. His recent work includes peer reviews of seismic strengthening proposals and seismic risk assessment for hydro power stations. He has carried out bridge strengthening design and peer review for the New Zealand Transport Agency, and research into the earthquake performance of underground structures, reinforced earth retaining walls and bridge abutments. John is a Life Member and past President of the New Zealand Society for Earthquake Engineering. He holds post-graduate degrees in structural and civil engineering from both the University of Canterbury and California Institute of Technology

LECTURE SUMMARY

There is a growing emphasis on Displacement Based Earthquake Design (DBD) analysis for buildings, retaining walls and bridge structures. DBD is specified as the preferred design method for highway structures in a draft revision of Section 5 of the Bridge Manual (NZ Transport Agency) expected to be adopted in 2016.

For bridges and major retaining wall structures, the damping and deformations within their foundations and backfilling have a major impact on their displacement response. In the past, the geotechnical input for the design of structures has focused on investigating and defining the soil strength parameters. To implement DBD methods there is now a need to investigate and assess soil stiffness as well as strength and to focus more on soil-structure interaction analysis.

The lecture highlighted the influence of soil stiffness and damping on the earthquake response of retaining walls and bridges and discussed the effects of the uncertainty in these parameters. DBD design procedures were illustrated by examples from the presenter's research background on soil-structure interaction.

The lecture was presented in the following three parts:

1. Rigid and flexible including outward sliding retaining walls
2. Stiff retaining walls and bridge abutments
3. Bridge DBD and pile foundations

This paper presents material discussed in Part 3 of the lecture. Material presented in Parts 1 and 2 was published in the June 2015 and June 2016 issues of NZ Geomechanics News respectively.

1. THE MAITAI RIVER BRIDGE

Pile testing work carried out during the construction of Maitai River Bridge on State Highway 6 is described in this Part 3 of the lecture material. The bridge is also used as an example to outline the principles of DBD and to demonstrate the influence of the foundations and site subsoil conditions on the displacement response of typical highway bridges.

The Maitai River Bridge was constructed in 1988 and is located at the mouth of the Maitai River, approximately 0.6 km north of the central business area of Nelson City. The bridge has five precast prestressed concrete I beam spans with a cast in-situ reinforced concrete deck. The superstructure is supported on four piers with single 1.4 m diameter reinforced concrete columns and hammerhead type column caps. Each pier has a 1.8 m diameter cylinder foundation that consists of a reinforced concrete core cast in a 12 mm thick steel liner. The cylinders were placed by excavation and top driving of the steel shell. Typical details of the structure are shown in Figures 1 to 4.

The prestressed I beams are seated on lead-rubber bearings on both the piers and abutments that provide energy dissipation and act as base isolators to limit the lateral loads on the substructure during earthquake loading.

Shear keys on the hammerhead column caps are located each side of twin transverse beams formed by diaphragms at the end of the spans. For longitudinal earthquake response there is a clearance of 75 mm between the diaphragm beams and the shear keys. In the transverse direction there is also a clear gap of 75 mm between the keys and the bottom flange of the beams. There are deck joints on the centre-line of the piers and no linkage bolts between the diaphragms either side of the joints so the spans can displace independently and hammering contact between the diaphragm beams may

Figure 1: Maitai River Bridge. Looking to south.

occur during strong longitudinal response.

The 1.4 m diameter columns are reinforced with 34 D32 longitudinal bars with a specified minimum yield stress of 275 MPa. Confinement over a 1.5 m length at the base of the column is provided by D20 hoops at 125 mm spacing. All four columns have a height of 7.62 m measured from the top of the cylinders to the top of the hammer-head pier caps. Although the bridge is base isolated, the pier columns were designed to have a relatively high resistance to lateral load and details at the base of the columns provide for moderate amounts of ductility in the unlikely event of a plastic hinge developing.

The main longitudinal reinforcement in the 1.8 m diameter cylinder foundations consists of 36 D32 bars (275 MPa specified minimum yield).

Four investigational bore holes were drilled to provide foundation design data. The upper sections of the foundation cylinders are embedded in unweathered to moderately weathered fine gravels in a matrix of yellowish brown silty fine sand with a trace of clay. In the upper layers the soil is medium dense with the coarser gravel layers towards the bottom of the cylinders becoming dense to very dense. The investigational work showed that the foundation soil was relatively uniform over the extent of the bridge site. A summary of the Standard Penetration Test (SPT) results is shown in Figure 5.

The bridge site is within the tidal influence of Nelson Harbour and the tops of the foundation cylinders are between 3.2 to 3.7 m below mean sea level. The maximum tidal range is about 4.0 m and thus the cylinder tops are

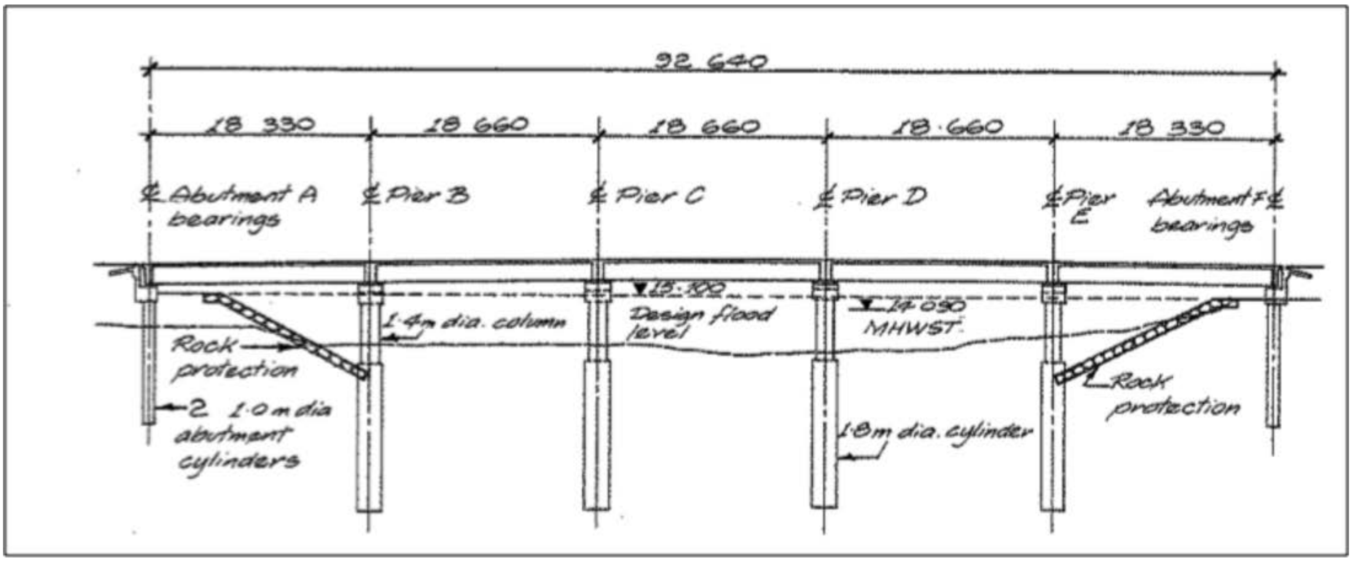

Figure 2: Elevation of Maitai River Bridge.

below ground water level during the complete tidal range.

Static lateral load tests were carried out during construction of the bridge to determine the stiffness of three of the completed piers and their cylinder foundations. The maximum load applied at the tops of the piers was 325 kN which is about 41% of the load estimated to develop the ultimate flexural strengths of the columns based on estimates of probable steel yield and concrete compressive strengths.

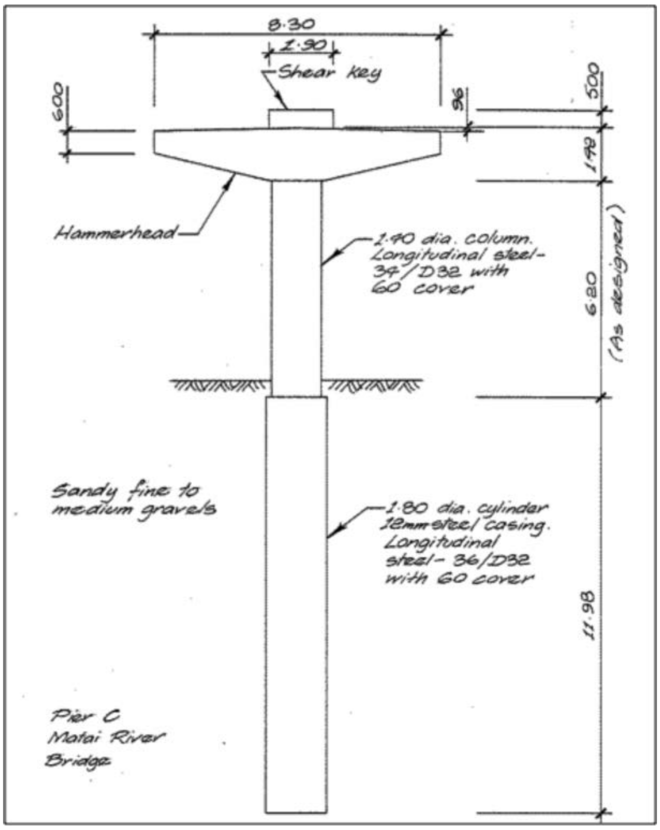

Figure 3: Typical section of the piers and foundations.

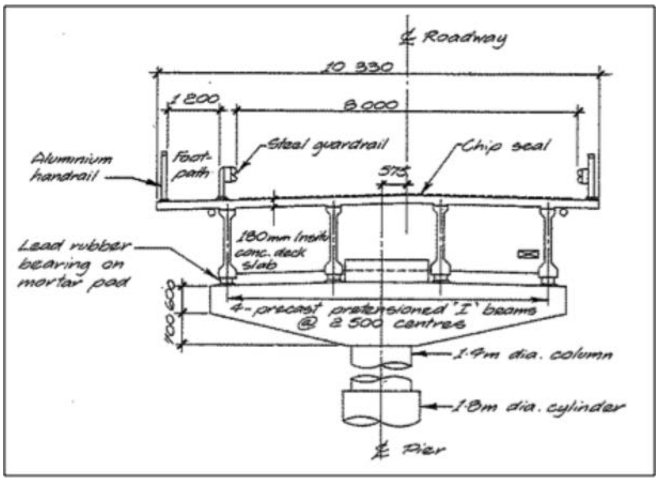

Figure 4: Typical section of the superstructure

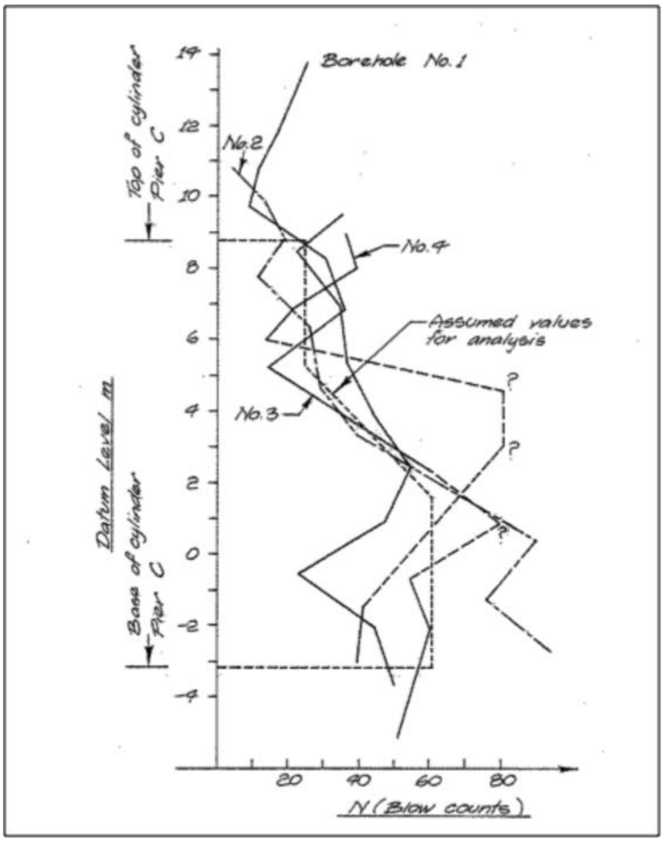

Figure 4: SPT N-values at pier pile locations.

2. MAITAI RIVER BRIDGE PILE TESTING

2.1 Loading Method

Horizontal static load testing was carried out on the piers after construction of the piers was complete and before the beams were placed on the piers. A tension loading system comprising of prestressing cables operating between the tops of adjacent piers was used. The loading system was designed to apply static horizontal loads at either the top of the pier hammerhead caps or at the base of the pier columns; however, because of the difficulties imposed by tidal waters and uneven bed levels only the pier top loading configuration was used.

Details of the pier top loading system are shown in Figures 5 and 6. The loads were applied at the top of the hammerhead of Pier C with two 25 t, 350 mm stroke jacks mounted in pairs of T shaped steel reaction frames that "hooked" on to the hammerhead and reacted against the vertical faces. Each jack and pair of reaction frames was positioned symmetrically about the column centre-line and tensioned a pair of 12.9 mm diameter Superstrand prestressing cables that had a safe load capacity of 250 kN. In order to load Pier C in a complete cycle with load reversal, cables were run to both Piers B and E and additional pairs of reaction frames were "hooked" on to both vertical faces of their hammerheads. The jacks were shifted between the two sets of loading frames each half cycle to alternately tension the cables to Piers B and E.

The cables were anchored to Piers B and E by passing them through 20 mm diameter plastic tubes cast into the hammerheads near their tops and using standard prestressing strand wedge grips.

Figure 5: Cable loading system between piers.

Figure 6: Loading frames on top of Pier C.

The slack in the cables was initially pulled out by hand and then by the jacks using strand wedge anchors to hold the strand temporarily against the jacking beam. The initial tension load required to remove sufficient sag in the cable spans to enable the jacks (350 mm stroke) to reach maximum load without stopping and packing was about 3.0 kN for each pair of cables.

2.2 Instrumentation

Simple instrumentation that could be installed immediately prior to the load testing was used. The loads applied by each jack were monitored by strain gauge type load cells connected to direct reading strain bridges. The accuracy of the load measuring system was of the order of 2%.

Horizontal deflections at the tops of the piers were monitored with dumpy levels positioned at distances of about 6 m from each pier and set-up to read graduated scales fastened to the hammerheads at their top surface level.

The rotation at the top of Pier C was measured with a theodolite set-up on the hammerhead to read a scale fixed to the face of Abutment A at a distance of 37.6 m from the instrument. The rotation of the top of Pier B was measured in a similar fashion using a dumpy level and a sighting distance of 19.0 m to the scale on Abutment A. The rotation and displacement scales were read to the nearest 0.5 mm.

Deflections and rotations at the tops of the piles at Piers B and C were measured using mechanical dial gauges supported from alloy reference frames held by scaffold tube posts driven into the ground on either side of the bridge centreline. The dial gauges enabled displacements of 0.01 mm to be resolved. However, because of temperature effects in the gauge support frames and creep in the loaded foundation soil it is unlikely that the displacements were recorded to better than ±0.05 mm accuracy. The accuracy of rotation measurement by the dial gauge method was of the order of $\pm 0.07 \times 10^{-3}$ radians.

2.3 Loading Sequence

Five complete cycles of loading were applied to Pier C by alternately tensioning the cables to Piers B and E. In the first two cycles, the loads in each direction were limited to about 60% and 90% of the maximum peak test load. The maximum peak load of 325 kN was applied in each direction of the three remaining cycles. This peak value represented approximately 75% of the theoretical load to produce first yield in the column reinforcement and was about 13% of the total dead load carried by the columns in the completed structure.

The first two of the three cycles to peak load contained six load increments and five unload increments in each of the two directions. A brief pause was made at each increment to read the instrumentation and each of these cycles took about 40 minutes to complete. The final cycle to peak load was curtailed with fewer increments because the dial gauges at the column bases had to be removed before the inflow of tidal water.

2.4 Pier Top Deflections

Deflections measured at the top of Pier C are shown in Figures 7 and 8. The plotted deflections are as recorded at the measurement point on the top surface level of the pier hammerhead.

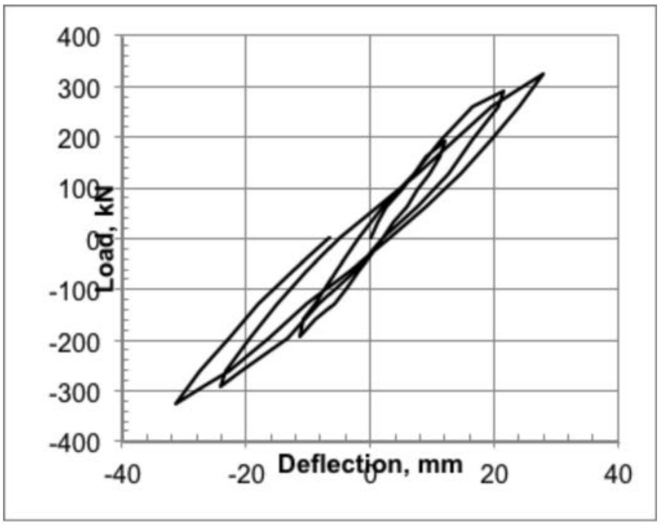

Figure 7: Pier C top deflection. Cycles 1 to 3.

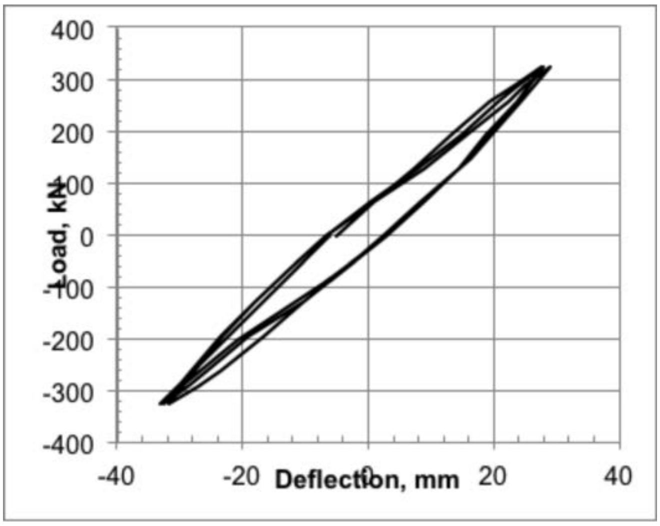

Figure 8: Pier C top deflection. Cycles 3 to 5.

A clear break in the linearity of the second loading cycle curve occurred at a load of approximately 260 kN. This change in stiffness corresponds to the load producing cracking of the concrete in the columns. On the assumption that the concrete strength in the columns at the time of testing (175 days after column pouring) was a factor of 1.5 greater than the 28 day compression test results (36 MPa), the ratio between the modulus of rupture and the compression strength at the time of test was estimated to be 0.13.

After the initial three cycles of loading, the hysteresis loops for Pier C became relatively constant in shape and the loading parts of the curves were relatively linear. A small degradation in stiffness occurred with increasing cycle number but the change is not very significant. The top deflection of Pier C in the fifth cycle, based on the average of the peak in each direction, was 31 mm. The average permanent set on release of the load in this cycle was 4 mm. The maximum top deflections of Piers B, and E in the fifth cycle were 32, and 15 mm respectively and the permanent sets in these deflections were 6, and 5 mm respectively.

2.5 Pile Top Deflections

Deflections recorded at the top of Pier C pile are shown in Figure 9. In the final three cycles of loading the Pier C pile hysteresis loops had almost constant shape with no significant degradation of stiffness. However, there was a significant permanent drift towards Pier E.

The deflection of the Pier C pile in the fifth cycle, based on the average of the peak in each direction, was 5.5 mm. The average permanent set on release of the load in this cycle was 2.2 mm.

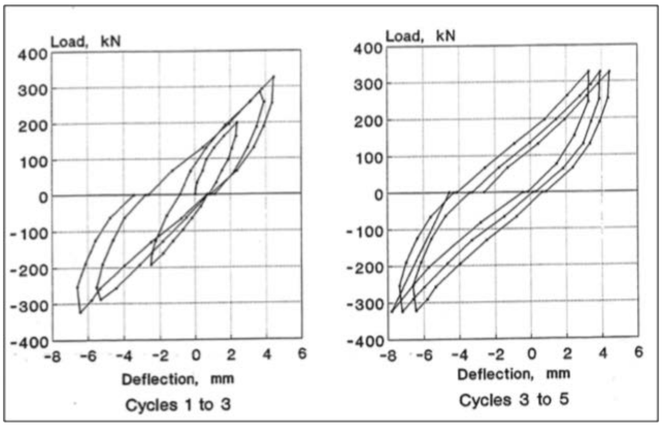

Figure 9: Pier C pile top deflection.

2.6 Pile Top Rotations

Rotations recorded at the top of Pier C pile are shown in Figures 10. The rotation of the Pier C pile in the fifth cycle, based on the average of the peak in each direction, was 1 x 10^{-3} radians.

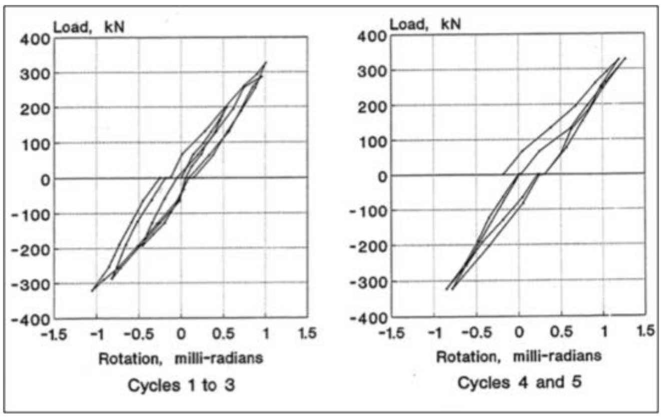

Figure 10: Pier C pile top rotations.

2.7 Theoretical Analyses

To provide a comparison with the test results, two analytical methods were used to compute the foundation and pier top deflections.

2.7.1 Frame Model Using Winkler Springs

In the first method a two-dimensional linear elastic frame model was used to predict the deflections of the Pier C column and pile resulting from the peak horizontal test load applied at the top of the piers. Details of the computer model are shown in Figure 11.

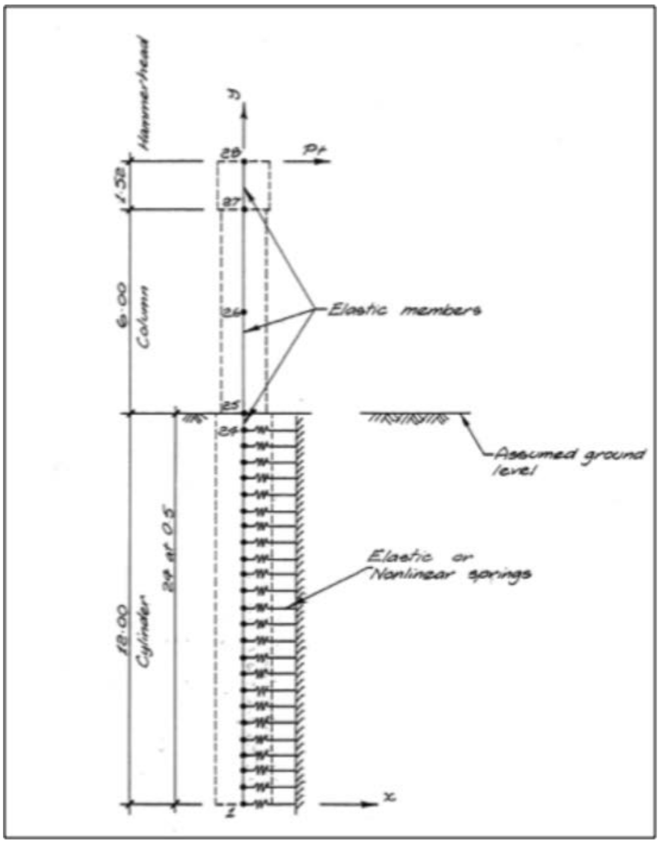

Figure 11: Computer model used to analyse Pier C.

The soil was modelled using Winkler springs. For the simple analysis described in the present paper linear springs were used but in other analyses described in Wood and Phillips, 1989 non-linear springs were also investigated.

Elastic moduli for concrete in the column and pile were based on measured 28 day strengths with an adjustment for the increase in strength at the time of testing at the time of testing. A cracked stiffness of the column for the column was estimated using stiffness ratios given in a chart presented in Priestley et al, 2007. The cracked stiffness is dependent on the longitudinal reinforcement and axial load ratio for the column. The pile was assumed to be uncracked and a transformed section was calculated based on the full thickness of the 12 mm thick steel shell.

SPT results obtained during the site investigation are plotted in Figure 4 together with an idealized N-value curve. This idealized curve is an approximate average of the test results and was used as the basis for determining the Winkler spring stiffness values. Firstly the N-values were increased by a factor of 1.2 to allow for the difference in driving energy between the typical New Zealand and USA penetration testing equipment. The charts of Schultze and Melzer, as published in Scott, 1981, and shown in Figure 12 were used to compute the relative density of the soil at a range of depths corresponding to the SPT N-values. The relative density was found to be approximately constant at 80% over the top 8 m of the soil profile below the top of the pile. The rate of increase of the initial tangent modulus of subgrade reaction with depth was estimated from the soil relative density using a chart from Lam and Martin, 1986 reproduced in Figure 13.

P-y curves calculated using the method described in Lam and Martin, 1986 were used to estimate the effect of non-linear behaviour and to estimate equivalent elastic springs. Curves shown in Figure 14 for the upper soil layers were calculated using a reduction factor of 1.5 on the initial tangent stiffness. This reduction factor was found to provide a good estimate of the secant stiffness in the top 2 m depth of soil.

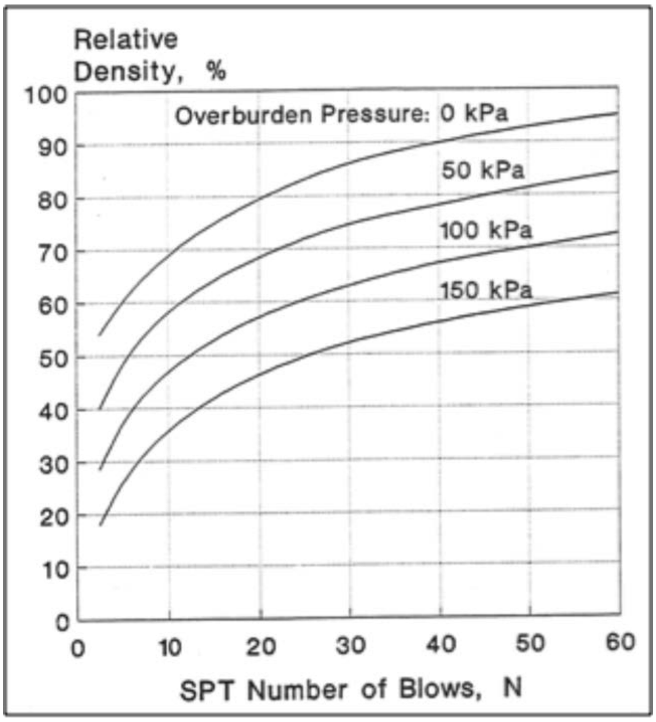

Figure 12: Schultze and Melzer curves from Scott, 1981.

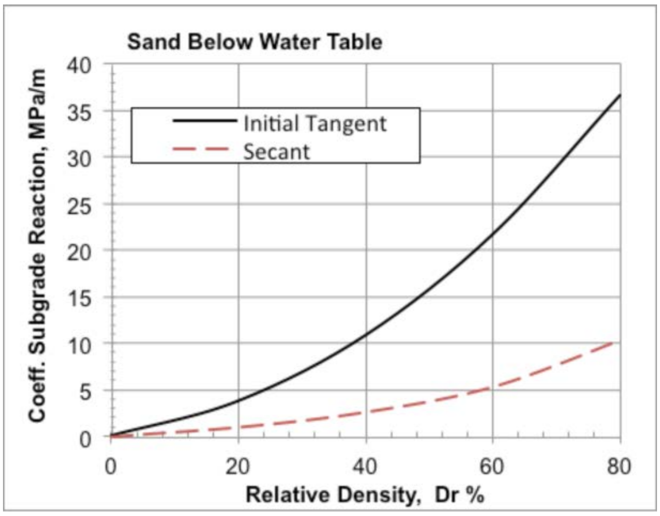

Figure 13: Coefficient of subgrade reaction from Lam and Martin, 1986.

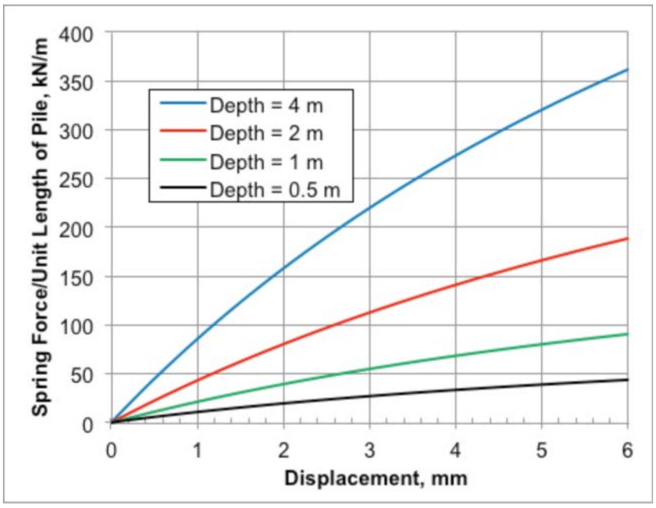

Figure 14: P-y curves for initial tangent modulus reduced by 1.5.

Input Parameter	Value	Comment
28 day strength concrete in column	36 MPa	Increased by 1.29 for strength at time of test (175 days)
28 day strength concrete in pile	43 MPa	Increased by 1.29 for strength at time of test (175 days)
Gross area of column	1.539 m^2	1.4 m diameter
Longitudinal steel ratio	0.0178	Area steel / gross column area
Axial force ratio	0.012	(Weight cap + column) / (Column area x 28 day strength)
Cracked stiffness reduction factor	0.38	Priestley el al, 2007, Fig 4.12
Column cracked I value	0.072 m^4	
Elastic modulus concrete in column	34.1 GPa	At time of test. Priestley et al, 2007 Eq (4.43)
Elastic modulus concrete in pile	37.2 GPa	At time of test. Priestley et al, 2007 Eq (4.43)
Pile transformed I value	0.666 m^4	12 mm thick steel shell
Height of test load above pile top	7.515 m	
Maximum test load	325 kN	
Soil relative density	80%	Ave. over top 6 m. From Schultze and Melzer curves.
Soil initial tangent modulus	35 MN/m^3	Sand below water table. From Martin and Lam, 1986
Reduction factor for secant modulus	1.5	Initial modulus reduced by this factor for spring stiffnesses

Table 1: Input Parameters for Frame Model Analysis

A summary of the main input parameters for the elastic frame model of Pier C is given in Table 1.

2.7.2 Elastic Continuum Analysis

The second analysis method was based on elastic continuum theory as presented by Pender, 1993.

For an elastic pile embedded in an elastic soil and loaded at the pile head the displacement u and the rotation θ at the ground line are given by:

$$u = f_{uH}H + f_{uM}M \qquad (2.1)$$

$$\theta = f_{\theta H}H + f_{\theta M}M \qquad (2.2)$$

Where: H is the applied horizontal load, M is the applied moment, and f_{uH}, f_{uM}, $f_{\theta H}$, $f_{\theta M}$ are flexibility coefficients.

Closed form expressions are available for the flexibility coefficients in terms of Young's modulus of the pile and soil, Poisson's ratio of the soil and the pile diameter. For short piles, the length is also a parameter. For cohesionless soils it is often assumed that the Young's modulus increases linearly with depth and is given by:

$$E_s = mz \qquad (2.3)$$

Where: m is the rate of increase in Young's modulus with depth, and z is the depth below the ground line.

For a linear variation of m with depth, the flexibility coefficients are given by:

$$f_{UH} = 3.2\ K^{-333}/(mD^2) \qquad (2.4)$$

$$f_{UM} = f_{\theta H} = 5.0\ K^{-0.556}/(mD^3) \qquad (2.5)$$

$$f_{\theta M} = 13.6\ K^{-0.778}/(mD^4) \qquad (2.6)$$

Where: $K = E_p/mD$

D is the pile diameter and E_p the equivalent Young's modulus for the pile.

A value for m was estimated using the average of five equations given in Pender, 1993 for large strain E_s values as a function of the SPT N-value. These equations were presented by five different research groups and are plotted in Figure 15. (Equation numbers in the legend are the number used by Pender.) The average E_s at a depth of 5 m was estimated to be 48 MPa indicating an m value of approximately 10 MPa/m.

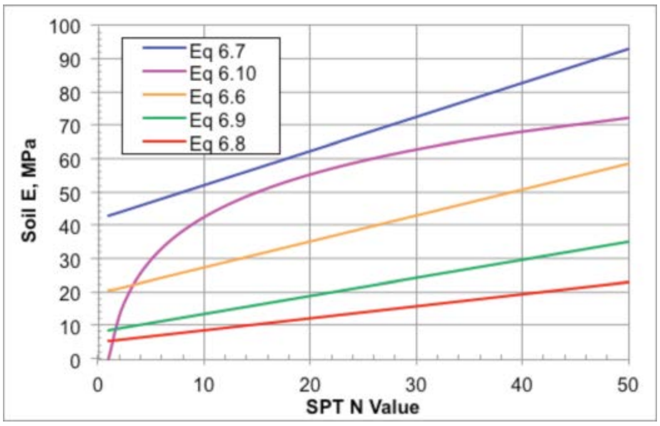

Figure 15: Plots of equations for large strain E_s values. From Pender, 1993.

This value was checked using the following relationship given by Seed et al, 1985 for the small strain shear wave velocity of cohesionless soils:

$$v_s \text{ (in m/s)} = 86\, N_j^{0.17} z^{0.2} \qquad (2.7)$$

Where: N_j is the SPT N-value as measured in Japanese practice and z is the depth below ground surface.

Because of the small power for N_j in Equation 2.7 the difference in SPT N-values from different practice can be neglected.

The small strain shear modulus G_{max} is obtained from:

$$G_{max} = \rho\, v_s^2 \qquad (2.8)$$

Where: ρ is the soil density.

A large strain E_s value can be estimated from G_{max} using:

$$E_s = 0.4\,(1+\nu)G_{max} \qquad (2.9)$$

Where: ν is the Poisson's ratio for the soil.

Equation 2.9 is based on the assumption that the large strain E_s value is obtained by reducing the small strain value for the elastic constants by a factor of 0.2. This reduction factor is uncertain but is of the correct order. Equations 2.7 to 2.9 gave E_s = 50 MPa at a depth of 5 m confirming that m = 10 MPa/m is of the correct order for the medium to dense sand and gravel below the water table.

2.8 Comparison of Analytical and Experimental Results

A comparison between peak measured displacements and rotations and the analytical predictions for Pier C is given in Table 2. The test displacements and rotations are given for one-half of the total peak to peak values in the fifth cycle.

Two sets of results are tabulated for each of the two analysis methods. For the frame model reducing the spring stiffness values by a factor of 2.0 was investigated. In the elastic continuum model the effect of reducing m from 10 to 9 was checked.

The frame and elastic continuum predictions based on the best estimate stiffness parameters for the soil (initial tangent modulus reduced by 1.5 for the frame model springs and m = 10 MPa/m for the elastic continuum model) are in generally good agreement with the measured results. Both analysis methods overestimated the pier top rotations by about 20%. Most of this discrepancy can be attributed to assuming a constant moment of inertia over the full height of the pier. In fact, only the lower half of the column would have been cracked and a more accurate variable stiffness model of the column would give a reduced pier top rotation.

Reducing the elastic continuum m value from 10 to 9 MPa/m gives a slightly improved prediction for the pile top displacement. Reducing the Winkler spring stiffness values by a factor of 2.0 increases the displacements at the top of the pile by approximately 60% demonstrating that although the pile steel encased section is very stiff, the pile top displacement (and rotation) are moderately sensitive to the soil stiffness assumptions.

2.9 Damping

Although the test loading was applied slowly (each cycle took approximately 40 minutes to complete) it is possible to make an estimate of the hysteric damping of the pier/pile and the pile systems from the areas within the Pier C hysteresis loops which are approximately elliptical in shape (Figures 8 and 9).

The equivalent viscous damping ratio for forced harmonic vibration is given by (Priestley et al, 2007, Equation 3.10):

$$\zeta = A/(2\,\pi\, u_m\, F_m) \qquad (2.10)$$

Where: A is the area within the force-displacement loop, u_m is the maximum displacement and F_m is the maximum force.

Parameter	Test Result	Frame Model		Elastic Continuum Model	
		Elastic Spring Stiffness Reduction Factor = 1.5	Elastic Spring Stiffness Reduction Factor = 3.0	m = 10 MPa/m	m = 9 MPa/m
Pier top displacement, mm	31	32	38	31	31
Pier top rotation, milli-rad	4.0	4.8	5.2	4.8	4.8
Pile top displacement, mm	5.5	5.1	8.2	4.9	5.2
Pile top rotation, milli-rad	1.0	1.1	1.5	1.0	1.1
Column disp (flexure comp), mm	18	19	19	19	19

Table 2: Comparison of Test and Analytical Results

Equation 2.10 gives equivalent viscous damping for the pier/pile (pier top displacement) and the separate pile (pile top displacement) systems of 7% and 15% respectively.

Because of the influence of creep deflection in the slow static testing, the damping under dynamic loads will be less than indicated by the above estimates. However, under dynamic loads, loss of energy also occurs because of wave propagation associated with soil-structure interaction and this will tend to increase the overall damping and compensate for static creep effects.

The maximum moment applied at the base of the columns during the tests was 2,242 kN m. The flexural capacity of the column section, based on increasing the specified steel reinforcement yield stress of 275 MPa by a factor of 1.1 to give a probable yield strength of 303 MPa, was 5,990 kN m. (To calculate this value the axial load on the column was taken as the full dead load from a completed span and no strength reduction factor was applied.) The maximum test moment was therefore 41% of the flexural capacity. Greater non-linear behaviour would be expected in both the soil and column at loads corresponding to the flexural capacity of the column. This would result in higher damping than indicated by the test results.

3. DISPLACEMENT BASED DESIGN OF BRIDGES

The displacement based design procedure (DBD) has been developed-over the past 20 years (Priestley et al, 1996, and 2007) with the aim of mitigating some of deficiencies in conventional force-based design (FBD). The main difference from force-based design is that DBD characterises the structure to be designed by a single-degree-of-freedom (SDOF) representation of performance at peak displacement response, rather than by its initial elastic characteristics.

The design procedure determines the strength required at designated plastic hinge locations to achieve the design aims in terms of defined displacement objectives. It must then be combined with capacity design procedures to ensure that plastic hinges occur only where intended, and that non-ductile modes of inelastic deformation do not develop. This capacity design approach is followed in FBD but in DBD the requirements are generally less onerous than those for FBD.

3.1 Basic DBD Design Theory

The design method is illustrated with reference to Figure 16, which considers a SDOF representation of a bridge, although the basic fundamentals apply to all structural types.

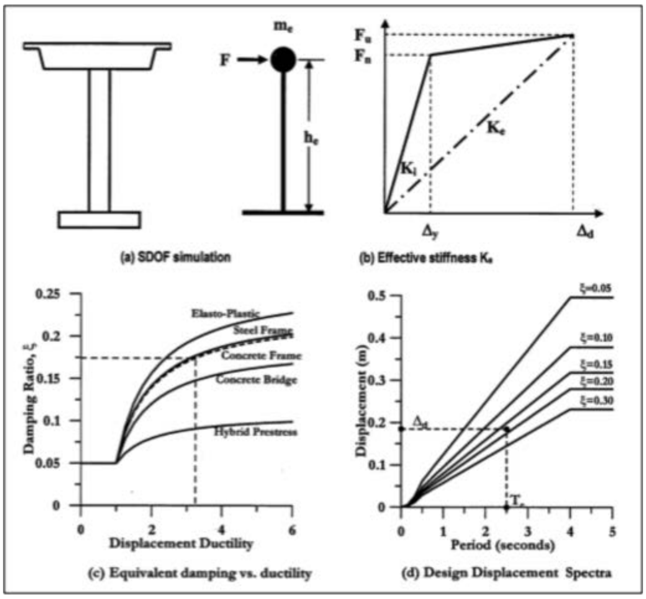

Figure 16: Main steps in DBD analysis. From Priestley, 2012.

The bi-linear envelope of the lateral force-displacement response of the SDOF representation is shown in Figure 16b. An initial elastic stiffness K_i is followed by a post yield stiffness of rK_i. In contrast to FBD which characterises a structure in terms of elastic, pre-yield properties (initial stiffness K_i and elastic damping), DBD characterises the structure by secant stiffness K_e at maximum displacement D_d (Figure 16b), and a level of equivalent viscous damping representative of the combined elastic damping and the hysteretic energy absorbed during inelastic response.

In a typical DBD design procedure a trial displacement at maximum response is chosen based initially on an acceptable drift ratio or ductility level (D_d/D_y) for the structure being considered. The system damping is then determined from charts such as those shown in Figure 16c, or other published equations for structural systems or components, which give the equivalent viscous damping as a function of the ductility demand on the system or component. The effective period T_e at maximum displacement response, measured at the effective height h_e (Figure 16a) can be read from a set of design displacement spectra for different levels of damping, as shown in Figure 16d. Design displacement spectra can be calculated from conventional acceleration response spectra (for example those given in NZS 1170.5). The effective stiffness K_e of the equivalent SDOF system at maximum displacement can be found by inverting the normal equation for the period of a SDOF oscillator to provide:

$$K_e = 4p^2 m_e / T_e^2 \quad (3.1)$$

Where: m_e is the effective mass of the structure participating in the fundamental mode of vibration.

From Figure 16b, the design lateral force and base shear force is:

$F = V_b = K_e D_d$ (3.2)

A design base moment can then obtained from the lateral force F and the effective height h_e. The section is designed to meet his moment demand and with ductile detailing based on the assumed initial maximum plastic displacement.

The design procedure for many bridges where the mass is concentrated at the superstructure level is very straightforward with much of the effort related to determination of the substructure characteristics and determination of the design displacement. Careful consideration is however necessary in for the distribution of the design base shear force V_B to the different discretized mass locations of more complex structures such as bridges with different height piers.

DBD has the merit of characterising the effects of ductility on seismic demand in a way that is independent of the system hysteretic characteristics and avoids ductility reduction factors used in FBD. The damping/ductility relationships separately generated for different hysteretic rules are available in published reports and design guidelines and there are simple rules for estimating the influence of different levels of damping on the design displacement response spectra

3.2 Design Example

A design example for the transverse response of a single bridge pier based on the dimensions and details of the Maitai River Bridge is summarised in Table 3. Because the spans are simply supported, a tributary mass approach using a single pier gives an acceptable approximation to the maximum displacement response near the centre of the bridge. To simplify the example, the displacements in the superstructure bearings at the tops of the piers have been neglected. The lead-rubber bearings used in the construction are quite flexible and provide significant damping so the calculations presented here will not closely represent the response of the as-constructed bridge. Including the bearings complicates that analysis because in this case the pier mass cannot be combined with the superstructure mass and two-mass system needs to be considered instead of the single mass used in the example.

The equation numbers listed in the comment/formula column of Table 3 refer to equations in Priestley et al 2007.

Flexibility of the foundation pile based on the test results is included in the Table 3 analysis. The sensitivity of the results to variations in these stiffness factors was investigated.

The main steps and assumption used in the example analysis are summarised below.

Structural Inputs

The structural inputs are based on the Maitai River bridge details. The span weight was taken as the weight of the superstructure plus the weight of the pier cap and one-third of the weight of the column. The effective height is the height of the centre of gravity of the total span weight with the pier cap and column weight assumed to be at the underside of the superstructure.

Foundation Stiffness Inputs

The values for pile head translation and rotation were based on the pile test results.

Earthquake Inputs

Geotechnical information for the site indicated that in terms of the NZS 1170.5 definitions the site subsoil category should be class C.

Displacement response spectra were computed from the NZS1170.5 acceleration spectra using Equation (2.2) in Priestley et al, 2007. The relationship between spectral displacement Δ(T) and spectral acceleration $S_a(T)$ can be expressed as:

$$\Delta(T) = T^2 S_a(T)/(4 p^2) \qquad (3.3)$$

Where: T is the period of vibration

Displacement spectra derived from NZS1170.5 for Soil C and for 5% and 15.5% overall damping (value calculated for the example) are shown in Figure 17.

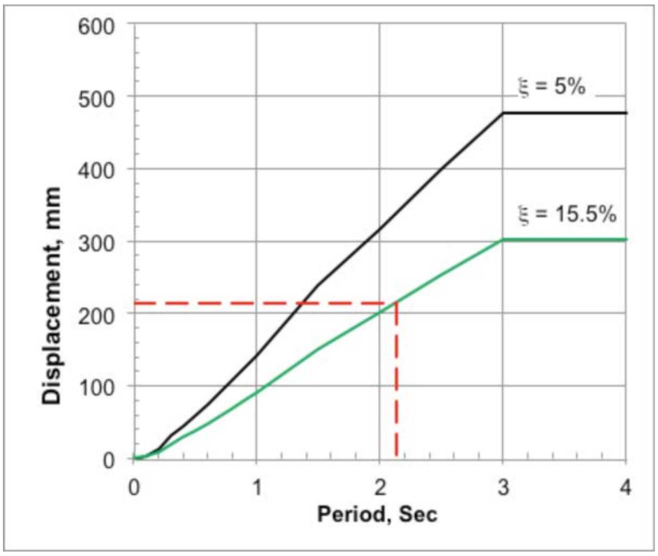

Figure 17: Displacement spectra for Soil Category C.

Yield Displacement

The yield displacement of the column was calculated from the section yield curvature which is independent of the column strength and is related to the column diameter and steel reinforcement yield strain.

Item	Symbol	Value	Units	Comment / Formula
Structural Inputs				
Span weight of superstructure and pier	W_s	2,380	kN	Includes pier cap + 1/3 column
Effective height of superstructure	H_e	8.55	m	Centre of dynamic mass above pile top
Column diameter	D_c	1.4	m	
Design ductility limit for pier	μ_d	3.5		Reasonable limit for concrete piers
Diameter longitudinal bars	d_b	32	mm	As designed
Specified concrete 28 day strength	f_c	35	MPa	As designed
Specified steel yield stress	f_y	275	MPa	As designed
Foundation Stiffness Inputs				
Horizontal stiffness at ground level	K_h	60	MN/m	From pile test results
Rotational stiffness at ground level	K_θ	2,440	MN m/rad	From pile test results
Earthquake Inputs				
Site Subsoil Category		C		
Zone Factor	Z	0.27		Nelson
Return Period Factor	R_u	1.8		2500 year return period
Spectrum corner period	T_c	3.0	s	See figure below
Corner period spectral shape factor	$\Delta_h(3)$	985	mm	See figure below
Corner period displacement	$\Delta(3)$	478	mm	$\Delta(3) = R_u{*}Z{*}\Delta_h(3)$
Yield Displacement				
Probable yield strength reinforcement	f_{ye}	303	MPa	$f_{ye} = 1.1{*}f_y$
Young's modulus reinforcement	E_s	200,000	MPa	
Yield strain	ε_y	0.00151		$\varepsilon_y = f_{ye} / E_s$
Yield curvature	ϕ_{yy}	2.43E-03	1/m	$\phi_{yy} = 2.25{*}\varepsilon_y / D_c$: Eq (4.57a)
Strain penetration length	L_{sp}	0.213	m	$L_{sp} = 0.022{*}f_{ye}{*}d_b /1000$: Eq (4.30)
Coefficient for column fixity	C_1	0.333		For cantilever: see note below Eq (10.1)
Yield displacement for pier	Δ_y	56.2	mm	$\Delta_y = C_1{*}\phi_{yy}{*}(H_e + L_{sp})^2{*}1000$: Eq (10.1)
Disp from foundation at height of mass	Δ_f	18.0	mm	$\Delta_f = M_b{*}H_c /K_\theta + V_b /K_h$
				Iteration required: Δ_f depends on M_b and V_b
Design Base Shear				
Design max inelastic displacement	Δ_d	215	mm	$\Delta_d = \Delta_f + \mu_d{*}\Delta_y$
Equivalent viscous damping ratio pier	ξ_e	0.151		$\xi_e = 0.05+0.444{*}(\mu_d - 1)/(\mu_d{*}\pi)$: Eq (3.17a)
Damping ratio for pile foundation	ξ_f	0.20		From test: adjusted to pier flexural capacity
Combined damping	ξ_c	0.155		$\xi_c = ((\Delta_d-\Delta_f){*}\xi_e+\Delta_f{*}\xi_f)/\Delta_d$: Eq(10.16)
Damping modifier	M_ξ	0.63		$M_\xi = (0.07/(0.02+\xi_c))^{0.5}$: Eq (2.8)
Corner disp of reduced spectrum	Δ_{cm}	302	mm	$\Delta_{cm} = \Delta(3){*}M_\xi$
Effective structure period	T_e	2.13	s	$T_e = T_c{*}\Delta_d /\Delta_{cm}$
Effective secant stiffness	K_e	2,110	kN/m	$K_e = 4{*}\pi^2{*}W_s/(9.81{*}T_e^2)$: Eq (3.1)
Design base shear	V_b	453	kN	$V_b = K_e{*}\Delta_d /1000$: Eq (3.2)
Required design base mom capacity	M_b	3,675	kN m	$M_b = V_b{*}H_e$: does not include P - Δ effects
As-Designed Flexural Capacity				
Unreduced moment capacity: 34 bars	M_c	5,980	kN m	For f_y = 303 MPa: f_c = 1.3 x 35 = 46 MPa

Table 3: Bridge Pier Analysis Example

The total yield displacement at the centre of mass is the sum of the column displacement and the contribution from flexibility of the foundation pile. The contribution from the pile is dependent on the column base moment and shear but at this stage of the analysis these parameters have yet to been determined. Trial and error is required to calculate the foundation displacement component. On an Excel spread sheet this can be carried out automatically using the formula iteration function.

Design Inelastic Displacement and Damping

The design level inelastic displacement was calculated from the sum of the foundation yield displacement and the total displacement in the column which is calculated from the column yield displacement and the assumed ductility capacity.

The equivalent viscous damping for the column was calculated from the assumed ductility capacity using the empirical equation given in Priestley et al, 2007. The damping in the pile/soil foundation system was increased from the value of 15% measured in the test to 20% to make allowance for the greater inelastic response expected in the soil under the applied moment and shear at the strength capacity of the column. (The moment from the test load was approximately 41% of the strength capacity of the column.)

The damping from the pier and foundation were combined using a weighted average based on displacements in the respective components. A damping modifier for the 5% damped code spectrum was calculated from the equation given in Priestley et al, 2007 (an expression first presented in Eurocode EC8, 1998).

Base Shear and Moment

The effective secant response period was calculated form the corner period of the damping reduced spectrum (15.5% damping) assuming the spectrum to ne linear over the period range from 0 to 3 seconds. This calculation is shown graphically in Figure 17.

The secant stiffness for the design level inelastic response was calculated using the SDOF expression for the relationship between period and stiffness.

The secant stiffness and the design level inelastic displacement give the required design lateral force (equal to the base shear for a SDOF structure). The required moment capacity at the base of the pier was calculated from this design lateral force and the effective height of the mass.

Column Section Details

Conventional strength or moment curvature analyses need to be carried out to determine the quantity of longitudinal reinforcement required in the column to provide the required capacity. As part of this design process the effects of P-Δ moments and the shear strength of the column need to be checked.

The column section in the plastic hinge region needs to be detailed with hoop confinement to provide the level of ductility assumed in the analysis. Procedures for calculating the confinement reinforcement required are given in Priestley el al, 2007. The calculations require establishing limiting strains for the steel and concrete in the plastic hinge region which are functions of the confining stress from the hoops, the volumetric ratio of the hoop steel and the strength of the confined concrete. Limit state and plastic curvatures can then be calculated from these strain limits, the section dimensions, neutral axis depth and the yield curvature (see Table 3). A plastic hinge length calculation enables the plastic displacement at the mass location to be estimated from the plastic curvature. Although the ductility calculations are straightforward, because the objective of the paper is to highlight the geotechnical aspects of DBD they are not presented here.

The ductility capacity of the pier can be calculated from the yield displacements (foundation and column) and the plastic displacement capacity of the column. The stiffness of the foundation affects the total yield displacement and since the structure ductility factor is directly related to this displacement, the foundation stiffness in turn influences the ductility capacity of the pier. Since the magnitude of the displacement spectrum is dependent on the site subsoil category the site local geology also affects the ductility capacity. Increasing the displacement demand on the pier (Soil D has a spectrum with displacements approximately 60% greater than Soil C) results in greater levels of design force and base moment and although the yield displacement in the column is unaffected by these increased demands, the yield displacement component from the foundation increases. For given set of column section properties, an increase in the yield displacement therefore reduces the pier structure ductility capacity.

3.3 Parameter Study for Foundation Stiffness and Subsoil Category

The example analysis summarized in Table 3 was repeated for site subsoil Category D and for each of the two subsoil categories the required column base moment demand was computed for cases where the stiffness of the pile (lateral displacement and rotation) was increased and decreased by a factor of 2.0 (stiffness ratios of 0.5, 1.0 and 2.0). The influence of the foundation damping assumption was investigated by increasing the damping ratio (DR) from 0.2 to 0.3 for the case when the pile stiffness was assumed to be as derived from the test (stiffness ratio = 1.0).

The results of the parameter investigation are shown in Figure 18 which shows the column base moment demand for each Soil Category (C and D) and for the foundation stiffness and damping ratios investigated. The stiffness ratio shown in the figure is the ratio of the stiffness used in the analysis / the stiffness derived from the test.

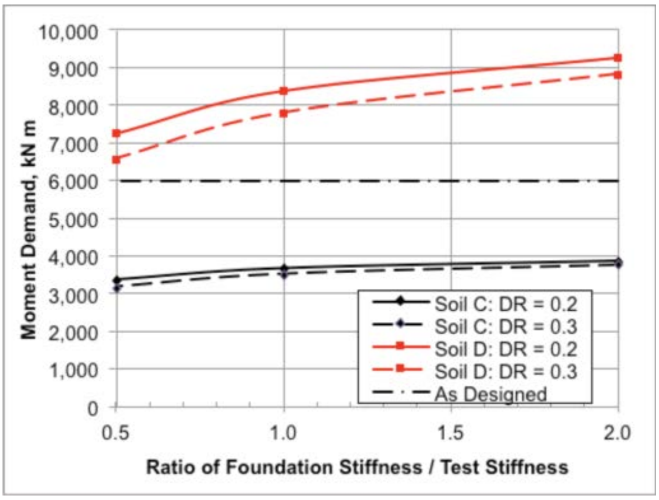

Figure 18: Moment demand related to stiffness and damping assumptions

The "as-designed" line in Figure 17 is the column base moment capacity calculated using the reinforcing in the as-constructed column and probable strengths (steel characteristic strength increased by 1.1 and concrete 28 day strength increased by 1.3).

The parameter study indicated that the base moment demand is not very sensitive to the foundation stiffness and damping ratio assumptions, particularly for the Soil C case. For the Soil D case, changing the foundation stiffness by a factor of 2.0 changes the moment demand by approximately 12%. Increasing the DR from 0.2 to 0.3 reduces the moment demand by between 5 to 10% for Soil D with the greatest decrease when the stiffness ratio is 0.5.

As expected, the subsoil category has a very large influence on the moment demand with the change from soil C to D approximately doubling the demand. It is clearly important to establish the soil category by adequate investigation and to consider interpolating between the two categories in some cases.

For this example, the as-designed columns have a moderate level of flexural reinforcement (1.8% of gross section area) and would have more than adequate moment capacity on Soil Category C but would have insufficient capacity for Soil Category D.

The total displacement and displacement components at the centre-of-mass as a function of the foundation stiffness ratio are plotted in Figure 19. The foundations contribute from 5% to 14% of the total displacement for Soil C and from 10% to 27% for Soil D. The relative magnitude of these contributions depends on the design ductility factor for the pier structure and in this case the factor of 3.5 results in quite large plastic deformations in the pier reducing the percentage contribution from the foundation. For lower structural ductility factors the foundation would have a more significant impact on the total displacements and also on the pier base moments.

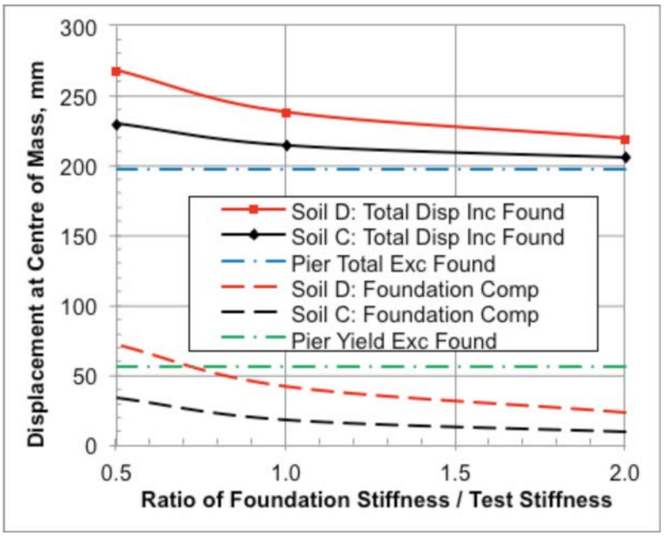

Figure 19: Total displacements and components at centre of mass

The ductility capacity of the as-designed column was checked for the range of foundation stiffness ratios investigated (0.5, 1.0 and 2.0) for each of the Soil Categories C and D. As mentioned above, the foundation stiffness and soil category influence the total yield displacement at the centre-of-mass and in this way affect the ductility capacity of the pier. Ductility capacities computed for the range of stiffness ratios investigated are shown in Figure 20. The ductility capacity is moderately sensitive to the foundation stiffness ratio and changes approximately 12% and 16% for Soil C and D respectively for a foundation stiffness ratio change of from 1.0 to 0.5 (or from 1.0 to 2.0).

The assumed ductility capacity of 3.5 in the example summarised in Table 3 would have been achieved by the as-designed section on Soil Category C. For the test pile stiffness (and less stiff foundations) more confinement reinforcing would be required than used in the design to give a ductility capacity of 3.5 for a bridge on a Soil Category D site.

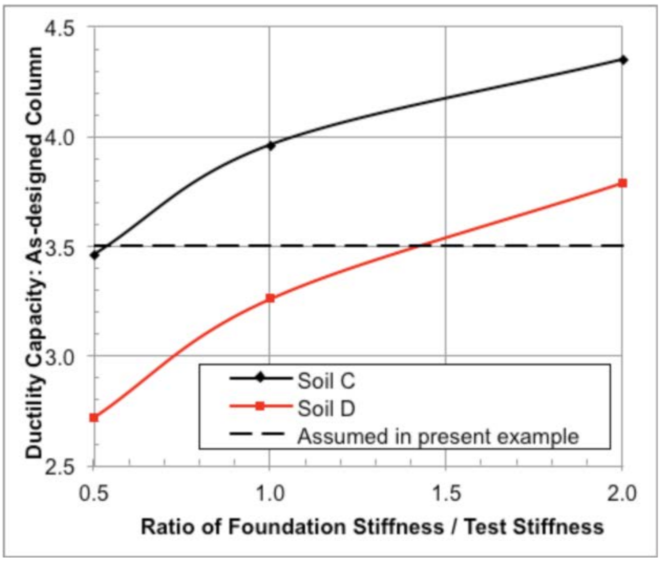

Figure 20: Ductility capacity as a function of foundation stiffness and Soil Category

In the plastic hinge zone the as-designed column had confinement reinforcing with a characteristic yield strength of 275 MPa consisting of 20 mm diameter hoops spaced at 125 mm. Reducing the hoop spacing to 100 mm would give a ductility capacity of 3.9 for a stiffness ratio of 1.0 on Soil Category D.

4 CONCLUSIONS

4.1 Pile Testing

A relatively simple static horizontal load pile test procedure followed by detailed back-analysis of the results provided valuable information on the pile foundation stiffness inputs that are important in the DBD design of highway bridges. This type of testing should be included as part of the investigation on major bridge construction projects as there is very limited published test information on the displacement performance of large diameter piles.

There was reasonably good agreement between the test and analytical predictions. However, the assumptions made regarding the soil stiffness parameters are not widely adopted in design where there may often be a tendency to underestimate the stiffness of large diameter piles in cohesionless soils. Undertaking sensitivity analyses and using several different analytical methods is an important part of any design procedure.

4.2 Displacement Based Design

Recently developed DBD procedures enable the stiffness and damping characteristics of bridge substructures to be incorporated into the design analysis is a rational and logical manner. The impact of variations in the soil foundation properties on the displacement response of a bridge can be readily established using this new approach.

A DBD example on a typical New Zealand highway bridge showed that the bridge pier flexural design was not strongly influenced by the assumptions made regarding the pile stiffness. However, the foundation stiffness will have a stronger influence on the design for low ductility factors (2.5 or less).

In the example presented the pier ductility capacity was found to be moderately sensitive to the pile stiffness assumptions.

The site Soil Category has a strong influence on the magnitude of the design displacement spectrum and in turn this has a major impact on the flexural demand and ductility detailing requirements for bridge piers. Adequate site investigation to reliably establish the local site geology is therefore important.

5. ACKNOWLEDGMENTS

The pile testing work on the Maitai River Bridge was part of a larger project involving pile testing on four bridges. The project was financially supported by the Road Research Unit of the National Roads Board. Murray Phillips of Mills & Wood and Graeme Beattie and Bob Stevenson of the Central Laboratories of Ministry of Works and Development provided valuable assistance with the site testing work.

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Reinforced soil structures used in mitigating dynamic rockfall and debris flow impact applications – New Zealand Examples

Eric Ewe, Geofabrics New Zealand Ltd (formerly Maccaferri NZ Ltd)

***THIS ARTICLE INTRODUCES* reinforced soil structures, their ductile characteristics, research and full-scale trials carried out on the rockfall impact ability. Two recent projects in Christchurch involving Green Terramesh reinforced soil bunds are presented. They are designed with the aim to withstand dynamic impacts caused by high kinetic energy rockfall impacts and debris flow impacts.**

INTRODUCTION

Reinforced soil can be defined as a soil mass containing multiple layers of horizontally laid planar reinforcement. The use of reinforced soil is increasing on a worldwide scale. Engineers are attracted by the greater reliability and control arising from the uniform properties of modern reinforcement, and the ability of reinforcement elements to strengthen local soil and offset the effects of defects.

The three major components that make up a reinforced soil structure are:

- Soil fill - compacted structural fill material from front facing to rear of reinforcement
- Reinforcement element - such as polymer geogrids, double twist steel mesh
- Facing element - to protect the reinforced soil mass against natural degradation

Each component is essential in ensuring the structure performs according to the design requirements for both ultimate and serviceability limit state.

The soil mass combined with reinforcement elements exhibits vastly enhanced strength and deformation properties and provides greater ability to resist forces. This is due primarily to the increased ductility of the reinforced soil mass. In materials, ductility refers to the ability of a material to deform under tensile stresses; unreinforced soil generally has little to no ability to withstand tensile stresses. Closely spaced reinforcement added to soil provides additional internal strength and increases stiffness properties due to surface interaction between the reinforcement element and soil.

The majority of the reinforced soil applications worldwide involve walls and slopes supporting infrastructure. Most design code of practices (such as AS4678:2002; FHWA and BS8006:2010) provide detailed design requirements for internal and external stability in accordance to the importance level of the structure and loading type.

This article focusses on a more recent and less common use of reinforced soil where it is used in structures built to resist dynamic impact loads imposed by falling rocks or flowing debris. Unlike for other reinforced soil structures, there are currently no widely accepted design codes of practice for these types of structures; to date, these types of structures are designed on the basis of physical testing, numerical modelling and back-analysis of the performance of existing structures.

Two examples of these types of structures built in Christchurch are presented and discussed.

DESIGN OF REINFORCED SOIL FOR RESISTING DYNAMIC IMPACTS

Reinforced soil structures (or bunds) are able to absorb high energy impacts while maintaining structural integrity. Considerable research and full scale trials have been carried out on reinforced soil bunds in an effort to develop and refine design parameters.

The design of the reinforced soil bund is primarily based on the penetration depth of an impacting rock based on its kinetic energy and size; the penetration depth largely determines the required thickness of the bund. The relationship between penetration depth and boulder sizes with their kinetic energy has been derived in a simplified chart developed by Calvetti & Di Prisco

Eric Ewe

Eric is a civil engineer with more than 19 years of experience; including 9 years working in Malaysia and 10 years in New Zealand at Geofabrics New Zealand (formerly Maccaferri NZ). His experience includes designing reinforced soil slopes and walls, slope rehabilitation works, slope stability analysis, embankment over soft ground, pavement stabilisation and rockfall protection structures. He is on the panel for the "MBIE Rockfall Protection Structures Design Considerations Guideline".

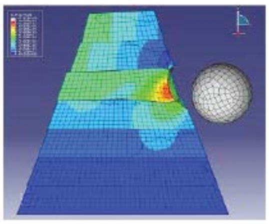

Figure 1: FEM modelling of the impact of a cubic, rigid body on a Green Terramesh reinforced soil embankment

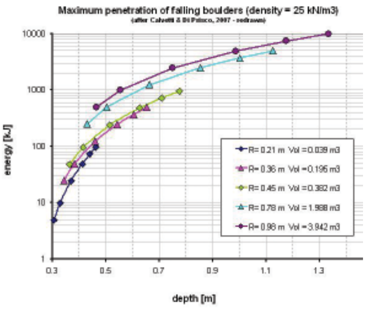

Figure 2: Derived maximum penetration of an impacting block in relation to impact energy (Galvetti and Di Prisco, 2007)

Photo 1: Real case 'piercing impact' on Green Terramesh reinforced soil bund

(2007), see Figures 1 and 2. This chart has been developed on the basis of FEM analysis, full scale trials on reinforced soil bunds and back analysis of actual events. The design approach and considerations have been covered by many technical papers and publications.

The research findings and investigation of actual rockfall events have shown that the key mechanism involved in stopping a high energy rock impact is dissipation of energy during formation of a crater in the upslope side of the bund. It is reported that 80-85% of the energy dissipation is through this plastic deformation while the remaining energy dissipates through frictional loss. Photo 1 indicates a piercing impact of an estimated 7500kJ rock impact energy on a Green Terramesh reinfoced soil bund; the edge of the bund was not affected by this impact indicating high energy absorption and ductility characteristics. There are exising bunds that have been designed to received impact energy of >15,000 kJ.

PROJECT 1- SUMNER ROAD, WAKEFIELD AVENUE ROCKFALL PROTECTION BUND

During the 2010/11 Canterbury earthquake sequence, the Sumner area experienced rockfall, cliff collapse and landslides. Wakefield Avenue in Sumner is part of an important transportation link from Lyttelton (the port of Christchurch) to Sumner and Christchurch. Christchurch City Council, as part of the **Summer-Lyttelton Corridor** project, has identified four major sections along the route where it is necessary to mitigate the slope instability risks to an acceptable level for road users. Wakefield Avenue is one of the sections.

The primary objective of the Wakefield Avenue project is to mitigate rockfall risk posed by three rock source areas scattered some hundreds of meters away on the slope above a total affected length of 1.2 km of Wakefield Avenue. Fallen blocks of up to 2.0 m nominal diameter have been observed at site. Trajectory analysis and post-earthquake observations indicate that falling rocks have impacted the road and that there is a risk that they could reach the residential zone across the road.

Initial attempts to mitigate the risk by scaling and removing the loose rocks on the slope were judged not to be feasible after subsequent earthquakes resulted in additional rockfall and caused further damage to the cliffs and rock bluff source areas. It was not considered practical to undertake scaling works over the large source area, nor was there any certainty that scaling would successfully treat the problem as it was unknown how far the damage extended into the rock mass. Mesh draping over the entire source area to control the rockfall was deemed to be cost prohibitive due to the large surface area for meshing. Rockfall analyses were performed by the engineers (Jacobs New Zealand Limited) on various sections along Wakefield Avenue. The final decision was made to construct rockfall protection structures atthree locations to intercept the falling rocks. This paper primarily discusses the longest bund, 410m long. The other bunds are 132m and 26m in length.

The two alternatives considered were an ETAG 27 certified catch fence and reinforced soil bunds. In this instance, the reinforced soil bund option was selected on the basis of the following reasons:

- Lower whole life cost
- Ability to withstand multiple impacts
- Easier maintenance and repair after each rockfall event without the need of replacement
- No setback needed to allow for horizontal displacement of structure
- Blends well with the environment
- Design life more than 50 years

The rockfall anlysis data was collated into three major categories based on the boulder sizes, bounce height and kinetic energy based on the design 95th percentile boulder size. The input data for the bund geometry determination is summarized in Table 1 below:

	Bund 1(a)	Bund 1(b)	Bund 2
Boulder unit weight	25 kN/m^3	25 kN/m^3	25kN/m^3
Nominal boulder diameter	2.0 m	2.0 m	2.0 m
Bounce height at the proposed embankment location, h_b (bottom)	1.4 m	3.8 m	1.5 m
Bounce height at the centre of the block, hd (Centre of mass)	2.4 m	4.8 m	2.5m
Expected kinetic energy	1,200 kJ	900 kJ	2,600 kJ

Table 1: GTM bund input data provided from trajectory analysis

The final results are Green Terramesh bunds with three different heights ranging from 3.6 m to 6.0 m and face inclinations of 70° or 80°. Along the different sections of the bund, choices of surface finish will consist of rock fill face and vegetated or planted face.

The adopted 'facing unit' for the reinforced soil bund is a Green Terramesh (GTM) unit. Figure 3 shows the isometric view of the unit. For lower height, narrower structures the soil reinforcement is comprised of the 2 m long double twist PVC Galfan (Zn + 5% Al) coated mesh 'tail' that extends along the base length of the unit; this forms a continuous layer between the upslope and downslope facing units. For taller, wider structures, additional geogrid reinforcement is included in between unit heights and to connect between the 2m-long tails from the opposing faces. The mesh is continuous from the base to the upper lid and the units are connected with stainless steel rings during construction.

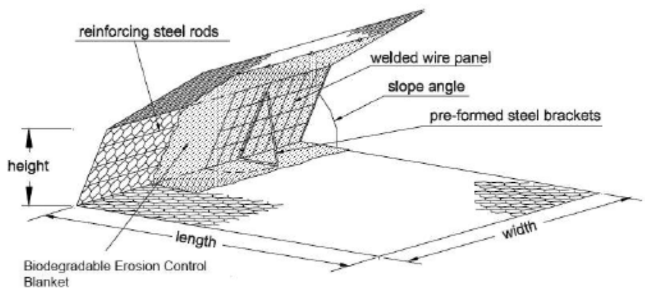

Figure 3: Green Terramesh Unit

One of the key benefits of the Green Terramesh unit is the ability to undertake local repairs after rock impacts without affecting the reinforced soil bund structural integrity. Repairs can be undertaken for penetration depths of <20 % of the bund thickness or usually < 500-700 mm of upslope deformation. This is the serviceability limit state of the bund. A similar double twist mesh layer can be laced over the impacted area so that the structure will be ready for subsequent impacts.

At the time of writing, the construction of the GTM bund by HEB Construction is still in progress (Photos 2 to 4) with the completion expected at the end of November 2016.

Photo 2: GTM bund 1 in Wakefield Avenue (before vegetation)

Photo 3: Cross section view of the Green Terramesh bund in Wakefield Avenue before filling

Photo 4: Green Terramesh bund 2 in Wakefield Avenue (rockfill face)

PROJECT 2- MAFFEYS ROAD MASS MOVEMENT PROTECTION BUND

Following the 2010/11 Canterbury earthquake sequence, the loess slope below Maffeys Road was identified by GNS Science as one of the areas with a high risk of mass movement that could affect properties and infrastructure down slope. The potential failure mechanism was identified as shallow sliding of overlying loess on top of the basalt bedrock surface. Instead of undertaking slope stabilization works, it was decided that a protection barrier should be constructed to partially deflect, intercept and contain the loess slope in the event of failure. AECOM were retained as the designers and worked closely with Geofabrics. Limited space available between the slope toe and properties meant that a reinforced soil protection bund had to be used to reduce the footprint and increase the containment volume.

In addition to the footprint consideration, the design of the reinforced soil bund had to fulfil a number of technical requirements:

- The protection bund has to be able to withstand, without toppling failure, the dynamic impact of debris flow impact at 5 m/s
- The protection bund must be able to be partially repaired or patched should any of the section be impacted
- The protection bund must have proven records of use
- It has to be durable (typically >50 years)
- There has to be very minimal maintenance required

The Green Terramesh reinforced soil bund was adopted because it meets all of these design requirements.

One of the design requirements is that the protection bund must resist the dynamic impact load from the mass soil movement. This mass soil movement was treated as a debris flow consisting of impact waves (resulting in dynamic forces) and consolidation of the material on the upslope side (resulting in static forces). Based on a velocity of 5 m/s with estimated saturated soil weight of 20 kN/m3, the dynamic pressure was calculated to be approximately 100 kPa through the use of an empirical formula provided by Hong Kong GEO "DN1/2012- Suggestions on Design Approaches for Flexible Debris-resisting Barriers". This dynamic pressure is further reduced since it will impact on a 70 degree inclined face upslope.

In this case, wave height has been assumed and slope

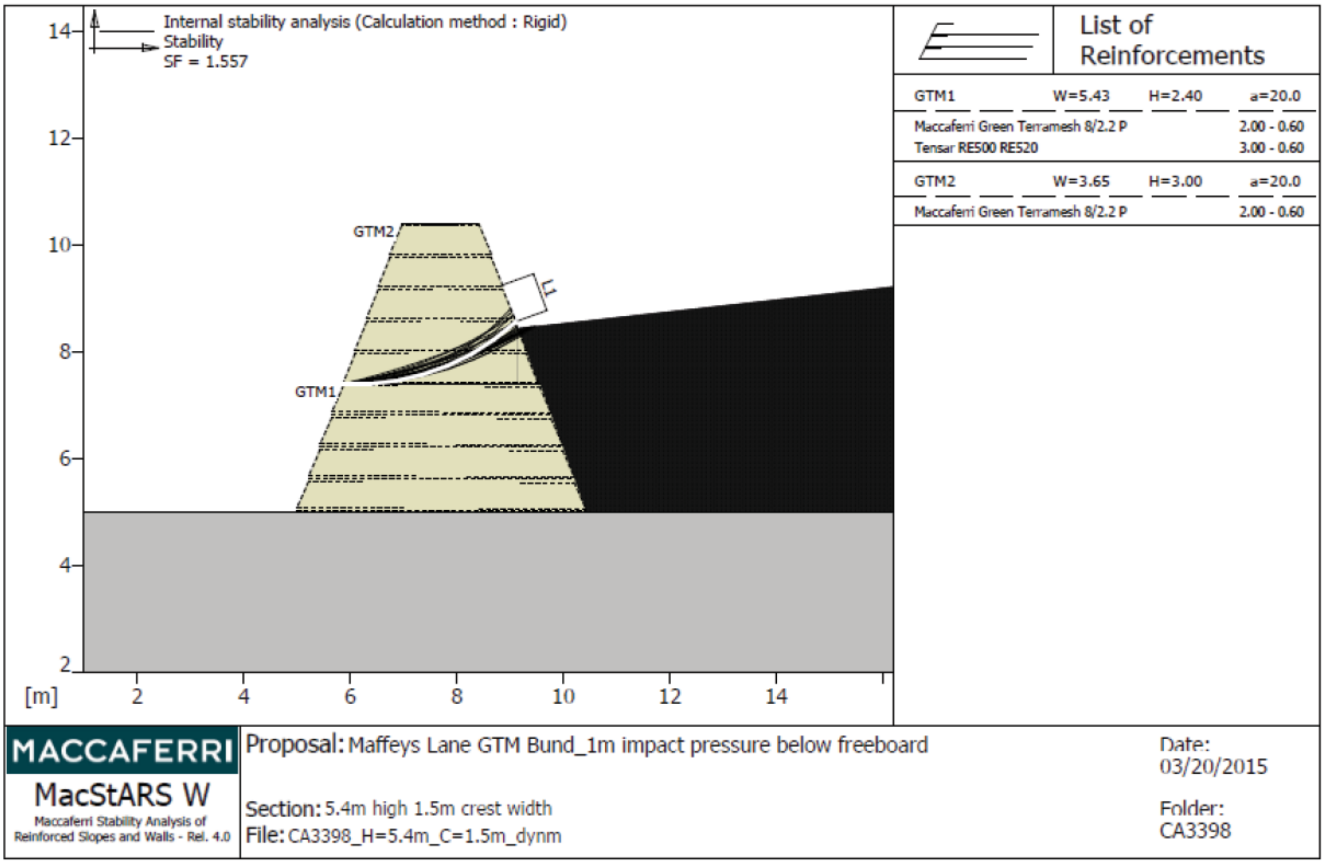

Figure 4: Macstars stability analysis snapshot of 5.4m high Green Terramesh bund

Photo 5: Area view of Green Terramesh bund in Maffeys Road (photo courtesy of Fulton Hogan)

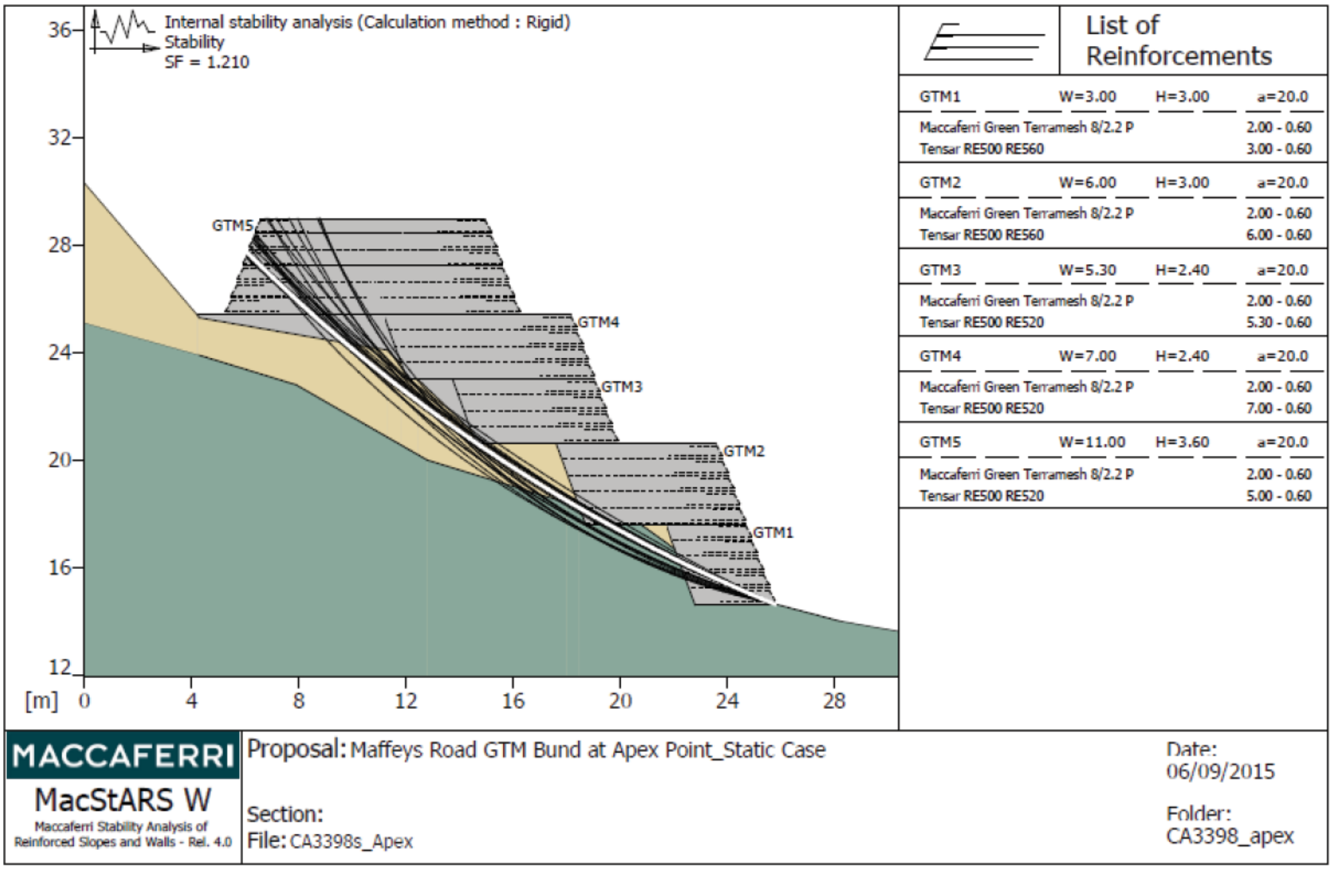

Figure 5: Macstars stability analysis snapshot of the apex section

Photo 6: Front view of GTM bund in Maffeys Road

Photo 7: Side view of Green Terramesh bund in Maffeys Road

stability software (MacStars) has been used to simulate several phases of the dynamic and static. Figure 4 shows a snapshot of the internal stability analysis result with accumulated static debris load and dynamic pressure acting at the upslope side of the bund.

The alignment of the Green Terramesh bund (Photo 5) incorporates an apex nearer to the source area to divert and deflect the soil mass to either side of the apex. This approach was selected by AECOM to separate the soil mass and thereby significantly reduce the required bund height. Constructing the apex near the main source area means that the landslide mass is intercepted after a much shorter travel distance, reducing the dynamic impact pressure. The long curve shape layout of the bund left and right of the apex is aligned at an angle to the impact direction, which further reduces the dynamic impact pressure. The long bund layout helps to contain the debris volume allowing for clearing as well as minimising the height of the bund.

The apex structure in the centre is a reinforced soil structure with total height of 14.4 m. Seismic and static internal and global stability analysis (Figure 5) have been performed to ensure the structure is self-stable under its own weight. Internal/compound stability factor of safety under the reduced seismic horizontal acceleration of 0.26g for ductile structure was found to be >1.1. Static stability analysis factor of safety was in excess of 1.7.

The Green Terramesh bund construction took the contractor (Fulton Hogan) approximately 1 year and was completed in early August 2016.

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The approval of Christchurch City Council, Jacobs and AECOM to publish this article is gratefully acknowledged. And I particularly thank Rori Green for her editorial efforts.

Bodies of Knowledge and Skills (BOKS) Projects

IPENZ

THE BODIES OF KNOWLEDGE and Skills (BOKS) project is a joint IPENZ, MBIE, SESOC and NZGS initiative to develop bodies of knowledge and skills initially for two fields of engineering – structural and geotechnical. It is envisaged the BOKS will form the basis for future assessments for Chartered Professional Engineers to provide a greater level of consistency and quality. Furthermore having a detailed BOKS endorsed by the relevant technical society provides a foundation from which ongoing CPD and other career development initiatives can be developed and delivered.

Graham Dilks
General Manager – Engineering Leadership

Brett Williams
General Manager – Professional Standards

Geoff Farquhar
Governing Board Member

Evolution of the CPEng(Geotechnical) Body of Knowledge and Skills

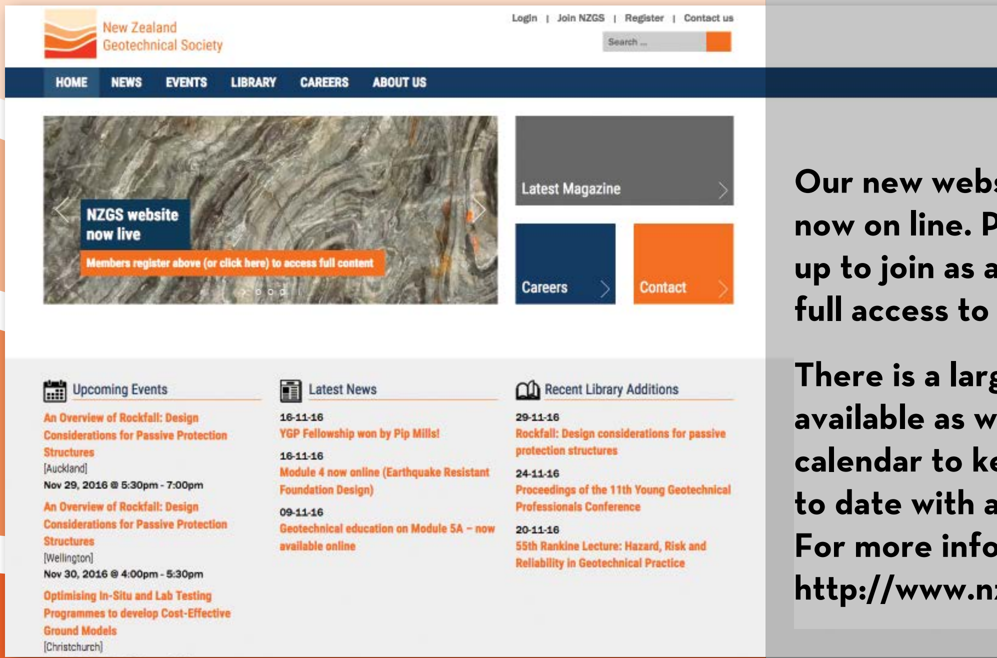
THE RATIONALE BEHIND the Chartered Professional Engineer (CPEng) Body of Knowledge and Skills (BOKS) was to define the core knowledge and skills that a recently qualified CPEng working as geotechnical engineer should be able to do in order to investigate, design and supervise the construction of geotechnical works in New Zealand. What has become clear over the past few years is that the geotechnical engineering profession in New Zealand (and other related construction professions) has had a poorly defined body of learning. It therefore falls upon the profession to ensure that the education and post-graduation professional training is matched to the knowledge and skills required in the profession. Concerns have also been raised about the quality standard of the CPEng qualification, particularly with regard to specialist fields of engineering like geotechnics. It was in this context that the Quake Centre and NZGS, supported by IPENZ and MBIE, set about to create a better foundation on which to discuss the experience and ongoing formal training needs of the geotechnical engineering profession. BOKS for engineering geology and structural engineering are also being developed.

From this an indicative set of tasks was loosely defined. The BOKS was then built up from the knowledge and skills that would be required to undertake these tasks. It is important to note that the tasks were not intended to define what an engineer must have done but rather they provide a reference for a likely range of experiences that would cover the required skills and knowledge. It must also be stated that the BOKS does not define competency. It is an essential part of the profession that an engineer seeking CPEng is assessed through the Registration Authority process, where the engineer must demonstrate competence in their Practice Area. It is envisaged that the BOKS will become a key reference document for this assessment.

It is hoped that the BOKS will provide a valuable document for the improvement of quality of the profession in the way that meets the expectations of clients, Building Consent Authorities, government, the wider engineering industry and the general public.

Greg Preston
Manager, Quake Centre

NZGS New Website



Our new website is now on line. Please sign up to join as a member for full access to the site.

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Chartered Professional Engineer (Geotechnical) – Body of Knowledge and Skills – 18 October 2016

1. INTRODUCTION

This document defines the core knowledge and skills that a Chartered Professional Engineer (Geotechnical) (CPEng(Geotechnical)) is expected to have in order to competently investigate, design and supervise the construction of geotechnical works in New Zealand. This Body of Knowledge and Skills (BOKS) is intended to complement and inform the Chartered Professional Engineer assessment process.

The purpose of the BOKS is to:

- Define the prerequisite skills and knowledge that are required of a CPEng(Geotechnical)
- Provide a framework for Continuing Professional Development (CPD) and postgraduate training.

The BOKS is not intended to be a competence assessment framework. However, it is expected that the BOKS will inform the competence assessment process used by the Registration Authority to assess a CPEng(Geotechnical).

The title 'CPEng(Geotechnical)' is not proposed by NZGS or the Registration Authority. It is simply used as a convenient descriptor in this document.

2. BACKGROUND

The Chartered Professional Engineers Registration Authority expects all Chartered Professional Engineers to:

- Either have a Washington Accord-accredited qualification (a four-year Bachelor of Engineering, Honours degree) or be able to demonstrate equivalent knowledge
- Demonstrate that they can work from first principles
- Demonstrate that they can solve complex engineering problems that require the application of engineering knowledge.

The Registration Authority gives these expectations in a competence standard(), which requires every Chartered Professional Engineer to demonstrate competence in their Practice Area. The Practice Area in which an engineer is assessed is aligned with one or two broad fields of engineering practice, which are published on the Register to assist the public when looking to engage an engineer. One of those Practice Fields is 'Geotechnical'.

While an engineer's Practice Area might be quite narrowly defined, engineers wishing to align their practice with the specialist field of geotechnical engineering, and be recognised as a CPEng(Geotechnical), are expected to demonstrate a breadth of geotechnical knowledge and skills (refer Section 3) which they are able to apply in a range of situations (refer Section 4).

The competence standard has 12 elements.

1. Comprehend and apply knowledge of accepted principles underpinning widely applied good practice for professional engineering
2. Comprehend and apply knowledge of accepted principles underpinning good practice for professional engineering that is specific to New Zealand
3. Define, investigate and analyse complex engineering problems in accordance with good practice for professional engineering
4. Design or develop solutions for complex engineering problems in accordance with good practice for professional engineering
5. Be responsible for making decisions on part or all of one or more complex engineering activities
6. Manage part or all of one or more complex engineering activities in accordance with good engineering management practice
7. Identify, assess and manage engineering risk
8. Conduct his or her professional engineering activities to an ethical standard at least equivalent to the code of ethical conduct
9. Recognise the reasonably foreseeable social, cultural and environmental effects of professional engineering activities generally
10. Communicate clearly with other engineers and others that he or she is likely to deal with in the course of his or her professional engineering activities
11. Maintain the currency of his or her engineering knowledge and skills
12. Exercise sound professional engineering judgement.

Complex engineering activities means engineering activities or projects that have some or all of the following characteristics:

(a) involve the use of diverse resources (resources includes people, money, equipment, materials, and technologies)
(b) require resolution of significant problems arising from interactions between wide-ranging or conflicting technical, engineering, and other issues
(c) have significant consequences in a range of contexts

(d) involve the use of new materials, techniques, or processes or the use of existing materials, techniques, or processes in innovative ways

Complex engineering problems means engineering problems that have some or all of the following characteristics:

(a) involve wide-ranging or conflicting technical, engineering, and other issues
(b) have no obvious solution and require originality in analysis
(c) involve infrequently encountered issues
(d) are outside problems encompassed by standards and codes of practice for professional engineering
(e) involve diverse groups of stakeholders with widely varying needs
(f) have significant consequences in a range of contexts
(g) cannot be resolved without in-depth engineering knowledge

Elements 1, 2, 3 and 4 focus on the technical knowledge and skills of a Chartered Professional Engineer. Thus those four 'technical' elements can be considered to distinguish the knowledge and skills specifically required of a CPEng(Geotechnical) from those of other Chartered Professional Engineers. Elements 3 and 4 deal with complex engineering problems while elements 5 and 6 deal with complex engineering activities.

3. KNOWLEDGE AND SKILLS

In looking to establish a finite set of knowledge and skills for a CPEng(Geotechnical), NZGS has defined the knowledge and skills required in order to deliver engineering outcomes for the different phases of a typical engineering project. A broad mapping back to the four 'technical' elements of the CPEng competence standard is then provided.

The knowledge and skills of a Chartered Professional Engineer (Geotechnical) are applied to the typical phases for investigation, design and construction of a project, namely:

(a) Options and alternatives identification and evaluation
(b) Site or project route selection
(c) Assessment of the geotechnical issues that need to be addressed in a project
(d) Development of programmes of geotechnical investigation focussed on addressing these issues
(e) Performance of geotechnical field and laboratory studies
(f) Analysis of geotechnical data and the performance of engineering computations
(g) Performance and engineering evaluation of construction, post-construction and site monitoring
(h) Preparation and engineering evaluation of geotechnical reports, design, and health and safety documentation
(i) Monitoring the construction of the projects listed above
(j) Inspection of the construction of geotechnical elements of the projects listed above
(k) Awareness and use of key technical documentation, guidance and standards
(l) Understanding of key building and health and safety regulations

It is recognised that there is overlap with the engineering geology profession and the PEngGeol registration administered by IPENZ and with the structural engineering BOKS (for example aspects of foundations and retaining walls).

KNOWLEDGE AND SKILLS		
Project phase	A Chartered Professional Engineer (Geotechnical) should be able to:	Element of CPEng competence standard
a) Options and alternatives identification and evaluation	i. Understand and describe the need for the project ii. Identify constraints and potential significant geotechnical issues iii. Identify range of potential solutions iv. Evaluate potential options and alternatives considering their relative feasibility, benefits and limitations	3
b) Site and route selection c) Assessment of geotechnical issues	i. Perform literature searches and site history analyses (including geology/geomorphology maps, hazard maps, aerial photographs, council files etc.) related to surface and subsurface conditions ii. Undertake a walk over survey, demonstrate a good understanding of geology and geomorphology, and how these provide evidence of the geotechnical issues that need to be considered in the design process. iii. Develop a preliminary ground model and possible hazards and documentation of the results iv. Screen sites based on this evidence	3
d) Development of programmes of geotechnical investigations	i. Communicate with other design consultants to determine the geotechnical input and the scope of the information needs ii. Formulate or evaluate the engineering aspects of ground investigation and laboratory testing programmes with appropriate consideration of the benefits and limitations of each investigation and laboratory test method. The investigations should include a variety of techniques, for example boreholes, test pits, in-situ testing such as shear vane, CPTs, SPTs and shear wave testing, sample collection and a range of laboratory tests such as classification tests (Atterberg, PSD), strength tests (UCS, triaxial, CBR) and compaction/stiffness (MDD, oedometer)	
	iii. Specify the scope and engagement of site investigation contractors, in consideration of the client's budget iv. Evaluate ground investigation and laboratory testing proposals v. Direct and/or modify ground investigation programmes, as required, upon evaluation of the conditions encountered with respect to the preliminary ground model	3
e) Performance of geotechnical field and laboratory studies	i. Classify and evaluate subsurface conditions so as to further develop the ground model ii. Understand the purposes for and direct (and/or perform) routine field and laboratory tests for many of the following: a. soil and rock description in accordance with the NZGS guideline b. soil classification c. soil and rock strength d. bearing capacity e. expansion properties f. consolidation characteristics g. compaction characteristics and/or material acceptability for use in fill h. special properties such as soil collapse potential, erosion a. potential and acid sulphate conditions i. pavement sub-base qualities iii. Installation and monitoring of field instrumentation (e.g. groundwater, slope movements, settlement).	3

f) Analysis of geotechnical data and performance of engineering computations	Assess or calculate: i. Soil and rock strength ii. Bearing capacity, pile capacity (shallow and deep foundations) and allowable bearing pressures iii. Settlement and/or ground movement under static and seismic loads and over the design life, including expansion and consolidation properties iv. Slope stability and displacement under static and seismic actions v. Geotechnical aspects of retaining systems under static and seismic loads vi. Soil collapse and/or erosion potential vii. Control of groundwater viii Earthworks including site preparation, cut/fill, compaction characteristics and material acceptability for use as fill ix. Pavement subgrade qualities and pavement design x. Understand and have competency in the use of most commonly used geotechnical analytical software (e.g. for retaining wall, foundation, pile and slope stability analysis and liquefaction assessment); and xi. Understand the limitations and assumptions behind this software.	2, 3, 4
g) Earthquake geotechnical engineering	Assess or calculate: i. Ground response to seismic action ii. Liquefaction susceptibility and vulnerability including assessment of secondary effect of settlement and lateral spread potential iii. Soil dynamic properties iv. Site subsoil class characterisation in terms of NZS1170.5 v. Seismic design parameters for geotechnical design vi. Engagement with structural engineers, for example soil-structure interaction vii. Understand the broad principles behind probabilistic seismic hazard analysis.	2, 3, 4
h) Performance or engineering evaluation of construction, post-construction and site monitoring	Confirm encountered ground and groundwater conditions and structure response consistent with design assumptions, but not limited to: i. Perform or supervise geotechnical testing and observe site construction such as foundations, earthworks, retaining walls and excavation ii. Analyse, design and evaluate instrumentation programmes to evaluate or monitor various phenomena in the field, such as settlement, deformations, slope creep, porewater pressures, groundwater variations and the development of trigger criteria and response actions iii. Evaluate geotechnical performance during construction iv. Evaluate engineering aspects of ground related distress associated with for example slope, foundation, and/or retaining wall distress or failure	4

i) Preparation and engineering evaluation of geotechnical reports	i. Prepare appropriate plans, borelogs, in-situ and laboratory test results; ii. Document laboratory and field testing results and observations; iii. Prepare written factual and interpretive reports which present ground model and findings and present and interpret these reports to the clients; iv. Interpret and verify factual and interpretative geotechnical reports prepared by others; v. Quantify and document geotechnical uncertainties on a systematic basis and incorporate of these into the design and risk assessment process; vi. Demonstrate judgement as to the key risks and mitigation strategies and an awareness of current risk guidance and standards.	3, 4
j) Preparation of design and health and safety documentation	i. **Prepare geotechnical design documentation, design** features reports and relevant construction specifications; ii. Design and document construction sequence; iii. **Provide the geotechnical aspects for "safety in design" reports prepared by others.**	4
k) Supervise construction	i. Oversee construction sequencing, managing the risk of instability throughout the construction sequence; ii. Review/update soil/rock exposures against the ground model and design assumptions; iii. Design and review temporary support system proposals; iv. Be familiar with geotechnical construction plant and machinery and their strengths and limitations; v. Supervise the construction to confirm it complies with the drawings and specifications and expected quality standards; vi. Design and issue any variations to the design as required to mitigate nonconforming work; vii. Keep records of all observations, contract variations and site instructions as they pertain to geotechnical matters; viii. Understand the management issues associated with contaminants in soils; ix. Understand the management issues with sediment run-off.	4

l) Awareness and use of key technical documentation, guidance and standards	Demonstrate general knowledge of the Building Act, Building Code, its core cited design actions and materials standards and other important guidelines and standards such as: i. AS/NZS1170 Structural Design Actions ii. NZS3604 Timber Framed Buildings iii. NZS4402 Methods of testing soils for civil engineering purposes iv. NZS4431 Code of Practice for Earth Fill for Residential Development v. AS2159 – 2009 Pile Design and Construction vi. AS4678 – 2002 Earth Retaining Structures vii. NZGS/MBIE Earthquake Geotechnical Engineering guidance modules viii. NZGS Field Description of Soil and Rock guideline ix. Awareness of international key standards for rock sample testing and in-situ testing etc x. MBIE guidance documents and practice advisories xi. NZTA Highways Structures Design Guide and Bridge Manual xii. IPENZ Practice notes and guidelines xiii. NZS 3910 Conditions of contract for building and civil engineering construction xiv. NZSEE "Red Book" The Seismic Assessment of Existing Buildings xv. Construction Industry Council – Design Documentation Guidelines xvi. Design Features Report templates xvii. AGS "Guideline for Landslide Susceptibility, Hazard and Risk Zoning for Land Use Planning xviii. ISO 31001 Risk management — Risk assessment techniques xix. Ministry of Education Structural and Geotechnical Guidelines for School Design and Education Infrastructure Design Guidance and other documents	2
m) Understanding of building and health and safety regulations	Demonstrate a good understanding of: i. Building Act, Regulations and Building Code ii. Demonstrate a good understanding of the Health and Safety at Work Act (2015) iii. Demonstrate a good understanding of the Chartered Professional Engineers of New Zealand Act (2002)	2

4. EXAMPLES OF COMPLEX GEOTECHNICAL ENGINEERING PROBLEMS AND ACTIVITIES

The CPEng competence standard requires an engineer to demonstrate an ability to analyse and develop solutions to complex engineering problems. The engineer uses his knowledge and skills to do these tasks. NZGS has identified a number of complex geotechnical engineering problems and activities for a CPEng(Geotechnical).

Engineers seeking specialist recognition as a CPEng(Geotechnical) are expected to demonstrate that they are capable of carrying out most of the following complex geotechnical engineering problems and activities:

a. Infrastructure route selection, land development, and/or sub-divisions
b. The assessment of the stability of natural, fill and cut slopes in soil and rock, in the order of 10m in height under static and seismic loadings with a medium to high risk to life (and/or property) if they fail
c. The geotechnical design of foundations and soil structure interaction analysis for IL 2 buildings (as defined by AS/NZS 1170.0) in the order of 3 storeys (about 10m) high, or bridges of comparable importance, founded on a range of ground conditions and a range of foundation types
d. Retaining structures in the order of 5m in height
e. Excavations in the order of 6m in depth (for example two levels of basement)
f. Assessment of situations with high risk to life or property where special precautions or expertise are or maybe required, for example:
    - During a response to an emergency events such as an earthquake or landslide
    - Significant potential of undermining or overwhelming of a nearby building or utility,
    - Obvious signs of distress of slope (natural, cut or fill) or retaining wall
    - Obvious signs of contaminated soils
    - Obvious signs of geothermal issues

The difficulty of the ground conditions shall also be considered when assessing complexity. For example, a simple structure on very difficult ground conditions such as thick peat may be considered complex geotechnical engineering problem.

- Supplementary complex engineering problems and activities include:
- Low Potential Impact Classification (PIC) dams
- Route selection and design of tunnels
- Route selection and geotechnical design for pipelines
- Specialist ground improvement techniques such as underpinning or grouting
- Rockfall protection structures
- Erosion control structures
- Offshore structures

5. COMMENTS ON DEMONSTRATING COMPETENCY

The complex engineering problems and activities in Section 4 represent the range of projects to which knowledge and skills would be applied by Geotechnical Engineers working within small, large or specialised organisations. Applicants would be expected to have at least five years of practical experience following graduation to cover the range of complex problems and activities, and to have been supervised by and have their work reviewed by a more experienced CPEng(Geotechnical).

Applicants should also be able to demonstrate that they understand the boundaries of their knowledge and skills, and will seek assistance when asked to work outside their competency or level of expertise, as per elements 10 and 12 of the competence standard. For example, a retaining wall design task may have a Structural Engineer design the structural elements while a Geotechnical Engineer will define appropriate earth loadings and bearing capacities, and assess overall stability.

It is acknowledged that applicants will have a range of both experience and competency for each complex problem and activity. They are likely to have worked as part of a team, in which case they will need to demonstrate they have taken responsibility for most of the problem and activity and/or when the problems and activities are spread over more than one project.

Demonstration of competency is likely to be through a combination of:

- presenting a portfolio of design calculations, drawings and reports;
- outlining the steps and judgment calls in the design process, and calculations for specific elements; and
- presenting case studies of project issues encountered and resolved, for example site inspections etc.

It is recognised that there are specialist fields or activities in geotechnical engineering that are not covered by the above BOKS. These include:

i. Tunnel engineering
ii. Mine design and mine slope engineering
iii. Offshore structure foundation design
iv. Water retaining structures and dams

These specialist activities will require much of the same knowledge and skills as listed in the table in Section 3 and may require working from first principles which are applicable across the broader aspects of geotechnical engineering. It may therefore be possible to meet the geotechnical CPEng(Geotechnical) BOKS requirements above even though the range of activities is quite different to that listed in Section 4.

PEngGeol BOKS

IN PARALLEL WITH the CPEng Geotechnical Engineering BOKS, we are preparing a Body of Knowledge and Skills for Professional Engineering Geologists (PEngGeol). The PEngGeol BOKS is a few months behind the CPEng (Geotech) BOKS to ensure that we make the best use of the outcomes of the CPEng (Geotech) BOKS development process.

We are following a very similar process for developing the PEngGeol BOKS to the one outlined by Kevin Anderson in this issue for the CPEng (Geotech) BOKS. We currently have a draft that is being reviewed by a working group representing a wide range of senior practitioners for all regions of New Zealand. We are also working with representatives from IPENZ to ensure that the BOKS is in line with the PEngGeol element descriptors. Once we have completed this phase, we will be ready for the consultation phase with the wider NZGS membership.

Please keep your eyes out for the PEngGeol BOKS consultation phase in a few months.

Marlène Villeneuve
Chair of the PEngGeol BOKS development working group

Obituary for Professor Stavros Bandis (1951 – 2016)

ON 11 JANUARY 2016 *the international engineering community lost Professor Stavros Bandis, a recognised expert in Rock Mechanics. Stavros passed away while at work, in his office at the Aristotle University of Thessaloniki in Greece, from a sudden heart failure.*

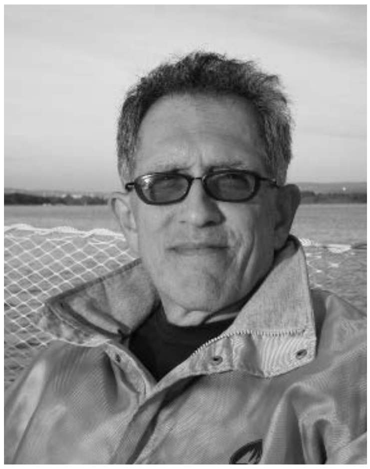
Stavros Bandis completed his Ph.D. at the University of Leeds in 1980 with Professor Dearman. His thesis 'Experimental studies of scale effects on shear strength, and Deformation of Rock Joints', was awarded the 5th Rocha Medal by the ISRM in 1983, a highly deserved award for those in rock mechanics who valued an alternative to Mohr-Coulomb for describing rock joints, as Dr. Nick Barton notes. Through this work, Stavros Bandis set the scene for the development of the Barton-Bandis joint behaviour criterion.

Stavros Bandis continued his post-doctoral research in the Norwegian Geotechnical Institute (NGI) between the years 1981 and 1985. For his post –doctorate research he received a scholarship by the Royal Norwegian Council for Scientific and Industrial Research (NTNF). His work at NGI included co-developing with Dr Nick Barton constitutive laws for modelling the engineering behaviour of rock discontinuities, known as the Barton-Bandis model, which has been adopted internationally. The recommended scale-effects for JRC and JCS, which are block-size dependent, were derived from his and Nick Barton's physical model and jointed rock experiments. Bandis alone was responsible for the hyperbolic normal closure and stiffness behaviour.

Since then, he was recognised as a world leader on the characterisation of discontinuities, discontinuous rock masses and in the field of Numerical Modelling of Discrete Materials in Geotechnical Engineering. The "Barton – Bandis model of engineering properties of joints" is widely used in international software for modelling rock mass discontinuous behaviour (UDEC-BB and FLAC by Itasca Inc, Swedge, UNWEDGE etc. by RocScience Inc. etc.).

Stavros had a long academic career as a Professor in the Department of Engineering Geology of Aristotle University of Thessaloniki. He started his academic career as a Lecturer in 1986 and was promoted to Professor in 1996 in the same department. His contribution in the University included the reorganisation and continuous development of the laboratory of Engineering Geology and the introduction and establishment of Engineering and Environmental Geology and Rock Mechanics courses for Civil Engineers. His contribution in the development of the Postgraduate Program of Protection, Preservation and Restoration of Cultural Monuments in Greece is recognised by the Aristotle University. He also served as Head of the Department of Geotechnical Engineering in the same University.

His friend and colleague John Sharp has written for him: "Stavros Bandis had an uncanny wish to achieve reality when characterizing and then modelling rock mass and rock excavation behaviour with UDEC-BB or 3DEC. His modelling work was outstanding and has probably not been matched due to his constant attention to detail. His loss as a world leader in his field with such an enormous insight and depth of knowledge is unaccountable. He



spoke to everyone as an equal, with interest and humour, always making a substantial and yet understated impression. A selection of his many contributions will be included in our joint book-project, which has been progressing for the last four years, during chapter-by-chapter sessions, mostly undertaken in a deserted village high in the Greek mountains, cut off from the internet."

Stavros was an internationally recognised expert in his field, and was often a panelist and keynote lecturer at international conferences and a member of several scientific advisory boards. He combined in an exceptional way the roles of academic researcher and geotechnical engineer of practice, providing his expertise and advice in a number of civil engineering projects internationally.

Above all, Stavros was a great Professor, Teacher and an inspiration for many students. Sadly Stavros is no longer with us but his legacy will remain among the scientific community, his team, students, colleagues, friends and family.

By Eleni Gkeli
Eleni.Gkeli@opus.co.nz

NZGS Awards Summary

THE NZ GEOTECHNICAL SOCIETY awards a number of prizes and scholarships on a variety of different timeframes. Full details of each of the awards available, along with details of how to apply or how to nominate someone, their frequency of repetition and a list of previous winners is on the website at www.nzgs.org. Please contact Sally Hargraves on the NZGS committee if you have any queries regarding any of the awards listed.

A list of the more regular awards and associated closing dates is given below.

NZGS 2017 AWARDS

NZGS Scholarship
applications close 31 October 2017. The scholarship is currently for up to $20,000 to enable a member of the Society to undertake postgraduate research in NZ that would advance the objectives of the Society.

Student Poster Award
registrations will close 20 October 2017; Completed posters are due by 10 November 2017.

YGP Conference 2018
abstracts due December 2017. Awards are available to NZGS members, under the age of 35 years, who submit an abstract for each YGP Conference (NZ/Aus). Judging is via the abstracts submitted and the award is to help fund attendance at the Conference at which the paper is to be presented.

UPDATE TO NOVEMBER 2016
We have received a total of eight applications for the NZGS Scholarship – which is increased this year up to a potential total of $20,000. Applications were from a mix of geologists and engineers, from all around the country and across a mix of PhD and Masters research topics, mostly full time, but a couple from people completing Masters whilst still working. It's great to see such a high level of interest in the scholarship this year. Successful applicants will be notified in December / January with confirmation in the June edition of Geomechanics News.

As at the end of October 2016, we had received six entries for the annual NZGS student poster competition, which will be judged and the winners notified in early to mid-December. All posters will be put up on display at a local branch event (location yet to be determined), prior to the prize giving. The winning posters will also be published in the June edition of Geomechanics News.

COMING UP IN 2017 - 2018
YGP Fellowship 2017 – this will be awarded to the author of the best paper by a NZ representative at each local (Aus/NZ) YGP Conference.

INTERNATIONAL SOCIETY AWARDS
The NZGS committee are continually seeking nominations for the following awards which are awarded either annually or biannually. Please contact the committee if you would like to nominate someone for any of these awards.

IAEG AWARDS
The Hans-Cloos Medal is the senior award presented by IAEG, given to an engineering geologist of outstanding merit in commemoration of the "founder of geomechanics". The recipient should therefore be 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.

The Richard Wolters' prize specifically recognises meritorious scientific achievement by a younger member of the engineering geology profession (less than 35 years old on January 1st of the relevant year) and is awarded to honour Dr. Wolters' many contributions to international understanding and co-operation.

ISRM AWARDS
The Rocha Medal has been awarded annually by the ISRM, since 1982, for an outstanding doctoral thesis in rock mechanics or rock engineering. Struck in the memory of ISRM Past President Manuel Rocha, it is intended to stimulate and reward young researchers. Since 2010, in addition to the Rocha Medal awarded to the winner, one or two runner-up certificates have also been awarded.

As with all the awards, the latest news can be found on the NZGS website at www.nzgs.org.

Sally Hargraves
sally@tfel.co.nz

International Association for Engineering Geology Environment

IAEG STRATEGIC PLANNING

Planning statements

The main objective of the extraordinary London Executive Committee meeting in May was discussion and formulation of a strategic plan for the IAEG. The concept, approach and outline of the strategic plan were subsequently ratified by Council at the Cape Town meeting in August.

A copy of the draft IAEG Vision, Mission and Objectives will be available on the IAEG and NZGS websites. These statements form Phase 1 of the overall strategic plan. The IAEG Executive is seeking comments on these draft statements from National Groups before finalisation in 2017. Please email feedback to Mark Eggers.

Phase 2 of the Strategic Plan will be establishing goals and actions under each major objective. Together with the various functions and work of the Executive Committee, these will form the detail of the Plan, which will be presented to November 2017 Council meeting during the 11th Asian Regional IAEG Conference in Nepal.

MANAGEMENT COMMITTEES

As part of the new strategic plan a number of new management committees are being established to assist with administrating the Association and promoting a number of the non-technical strategic goals. These committees will sit beside the Commissions, which address the technical elements of the Association. Members of the new committees will be appointed from Executive, Council and general membership.

BULLETIN

The Bulletin remains in a strong position with more papers being submitted each year and a rising Impact Factor. Martin Culshaw is Chief Editor 2012-2018. Due to the workload, two new Joint Editors-in-Chief will be appointed to begin in 2019. Martin is seeking new Editorial Board members. If you are interested contact him at **martin.culshaw2@ ntlworld.com.** Only around 1% of all papers come from Australia and New Zealand so, we still need to improve our publication record in the Bulletin.

WEBSITE

The IAEG web site (**http://www. iaeg.info/**) is a key part of the IAEG Communication Strategy, which consists of video lectures, web letter and messaging on social networks such as Linkedin. We are creating a specific database of presentations and lectures on different topics related to engineering geology to spread the knowledge in this area and try to involve more young people. Commission presidents/secretaries can upload and publish directly the documents of their commissions. There are now members only pages for IAEG members.

MEDALS AND AWARDS

Winners of the following awards for 2016 were announced at the Cape Town meetings:

Hans Cloos Medal – Prof Resat Ulusay, Turkey. Lecture to be presented at the 11th Asian Regional IAEG Conference in Nepal

Marcel Arnould Medal – Prof Giorgio Lollino, Italy

Richard Wolters Prize – Dr Fan Xuanmei, China.

No Richard Wolters Prize nominations were received from Australia or New Zealand. This lack of participation by Australasia will likely be addressed for the next round of the RWP at the 2018 IAEG Congress in San Francisco by incorporating a national competition associated with the Chapter-led YGP nights in 2017.

Mark Eggers

Mark is is a Principal and Director at Pells Sullivan Meynink where he consults on large civil and mining projects across Australia, New Zealand and SE Asia. Mark has a keen interest in education and research through close associations with University of New South Wales and University of Canterbury. He also co-teaches field courses in engineering geology for the Australian Geomechanics Society.

OTHER ITEMS

A new electronic newsletter will commence sometime in late 2016 for members which will be called the IAEG Connector.

An IAEG member survey will take place shortly to gain a better understanding of the make-up of the membership in terms of qualifications, degree, work place, and areas of expertise.

A FedIGS meeting was held on April 14 2016. JTC progress was a major item of discussion including JTC3 on education. A proposal to set up a new JTC on mining was scuttled by the ISRM; as such a new IAEG Commission on mining is being planned.

International Society of Soil Mechanics and Geotechnical Engineering

NEW ISSMGE WEBSITE

The ISSMGE Board has commissioned the design of a new website, which is now live. The new website has a modern layout, more valuable content with enhanced functionality on a range of devices. The navigation bar better communicates the many activities of our organization, including:

- "Online Library", under "Publications", includes a collection of 3,450 papers for download and a number of conference proceedings, including ICSMGE.
- "Media" contains a collection of 28 webinars and online honour lectures.
- There are sub-websites for each of the TCs, the Young Members Presidential Group (YMPG), and the Corporate Associates Presidential Group (CAPG). These websites will now be operated directly by the Committees and Groups themselves, and news from these committees and groups will be automatically promoted through the main page of the ISSMGE website.
- Under "News", you will find announcements made by various active groups in the ISSMGE.

Members are encouraged you to submit any feedback about the current website, as well as ideas about future initiatives.

WEBINARS

The ISSMGE continues to offer successful and topical bi-monthly webinars on a wide range of geotechnical engineering topics by international experts in their fields. Past webinars are an excellent long-term resource for the profession and are available for viewing at any time from the website. Members are encouraged to suggest future webinar presenters and/or topics by emailing me.

19TH ICSMGE, SEOUL 2017

For the 19th International Conference on Soil Mechanics and Geotechnical engineering, in Seoul, Korea in September 2017, 9 papers are being prepared by NZGS members and 45 by AGS members. The AGS is grateful to the NZGS for transferring 9 of its unallocated papers to the AGS. Prof. Mick Pender is managing the NZGS's submissions and I am managing the AGS's. Papers need to be uploaded to the 19ICSMGE website by January 15, 2017.

SYDNEY BID FOR 2021 ICSMGE (20ICSMGE)

The bid by the AGS to host the 20th ICSMGE in Sydney in 2021 is progressing with regular teleconferences. Our bid is far more visible in 2016, with exhibits (poster + AGS promotional materials) at many international conferences. Several Member Societies have confirmed or reaffirmed their support for the AGS bid and, at this stage, no other Member Society has indicated an intention to bid. Nominations must be submitted to the ISSMGE Secretariat by April 10, 2017.

ISSMGE BULLETINS

Much of the information about the ISSMGE's activities are contained in the ISSMGE Bulletin. To download copies, please go to the website.

ISSMGE FOUNDATION

The next deadline for receipt of applications for awards from the ISSMGE Foundation is February 1, 2017. Further information is available on the website.

Mark Jaksa

Mark is Head of the School of Civil, Environmental and Mining Engineering at the University of Adelaide. Over the last 25 years Prof Jaksa's research at the University of Adelaide has concentrated on probabilistic methods, geostatistics, artificial intelligence, ground improvement, expansive soils and geo-engineering education. He has published over 125 journal and conference papers on these topics.

ISSMGE VP (AUSTRALASIA)

My 4-year term as Vice-President for Australasia draws to a close at the end of the Seoul Conference in September 2017. We need to advise the ISSMGE Secretariat by April 10, 2017, who the next VP (Australasia) will be. The usual 2:1 (AGS:NZGS) cycle has become 1:1 over the past 20 years. I have been charged with the task of exploring whether the NZGS has any desire to change the cycle to parity (1:1), as has recently been undertaken for the IAEG. The AGS does not have strong feelings about this but wishes to open it up for discussion in the spirit of good will.

International Society for Rock Mechanics

VOTING AT 2016 COUNCIL MEETING

Three issues required voting at the Council meeting:

1. Increase in Rocha Medal prize (from €1,000 to €2,000) – passed
2. Adoption of operational mode with FedIGS (greater interaction between member Societies) – passed
3. Selection of location for the 2018 International Symposium – Singapore was selected.

YOUNG PROFESSIONALS:

No progress on whether ISRM will promote a young member (under 35) group in its profile, and it may remain a National Group activity. Fostering of younger members (about 20% of members) is a recognised need, and aside from National Group activities there will be young professional group activities at particular rock mechanics conferences.

COMMISSIONS

There are 17 ISRM Commissions in the 2015 – 2019 term. Commission purposes and anticipated products, along with membership, are given on the ISRM website (links on https://www.isrm.net/gca/?id=153).

The Technical Oversight Committee (TOC) - Doug Stead (chair), Stuart Read and Norikazu Shimizu - has evaluated reports from each of the Commissions covering their activities over the last year (May 2015 – August 2016) and reported to the Board and Council meetings. Some commissions which are less active than others require attention, others need to improve interaction with related technical societies (e.g. undersea tunnels with ITA) or regional breadth of membership (often dominated by Asia), and the task of identifying gaps (mining being is noted as one) or where there is overlap between commissions is in progress.

ROCHA MEDAL (2017 & 2018):

Nominations for the 2017 award closed on 31 December 2016, with 14 theses nominated. The winner is Bryan Tatone (Univ Toronto, Canada), with the thesis “Investigating the evolution of rock discontinuity asperity degradation and void space morphology under direct shear”.

The award recognises the most meritorious PhD thesis in rock mechanics, and nominations for the 2018 award are due by 31 December 2017. Further details are on the ISRM and NZGS websites.

ISRM ON-LINE LECTURE

Two ISRM on-line lectures have been given over the last months:

a) 14th by Prof. Walter Wittke: “Design based on the Anisotropic Jointed Rock Model (AJRM) – Fundamentals and Case Histories".

b) 15th by Professor Nielen van der Merwe: “Predicting the probability of failure into the future".

Copies of the broadcasts are available on the ISRM website

COMMUNICATION

The ISRM website (www.isrm.net) has information on the society’s intent, structure and activities, including conferences, commissions, awards, products and publications. For those NZGS and AGS members affiliated to ISRM as individual members there is a members area with access to further products.

The ISRM Digital Library (https://www.isrm.net/gca/?id=992) is intended to make rock mechanics material available to the rock mechanics community. ISRM individual members can download up to 100 papers per year from the ISRM conferences. Regular means of communication (under ISRM information on the website) are the ISRM newsletter and the ISRM News Journal.

Other recent items on the website include:

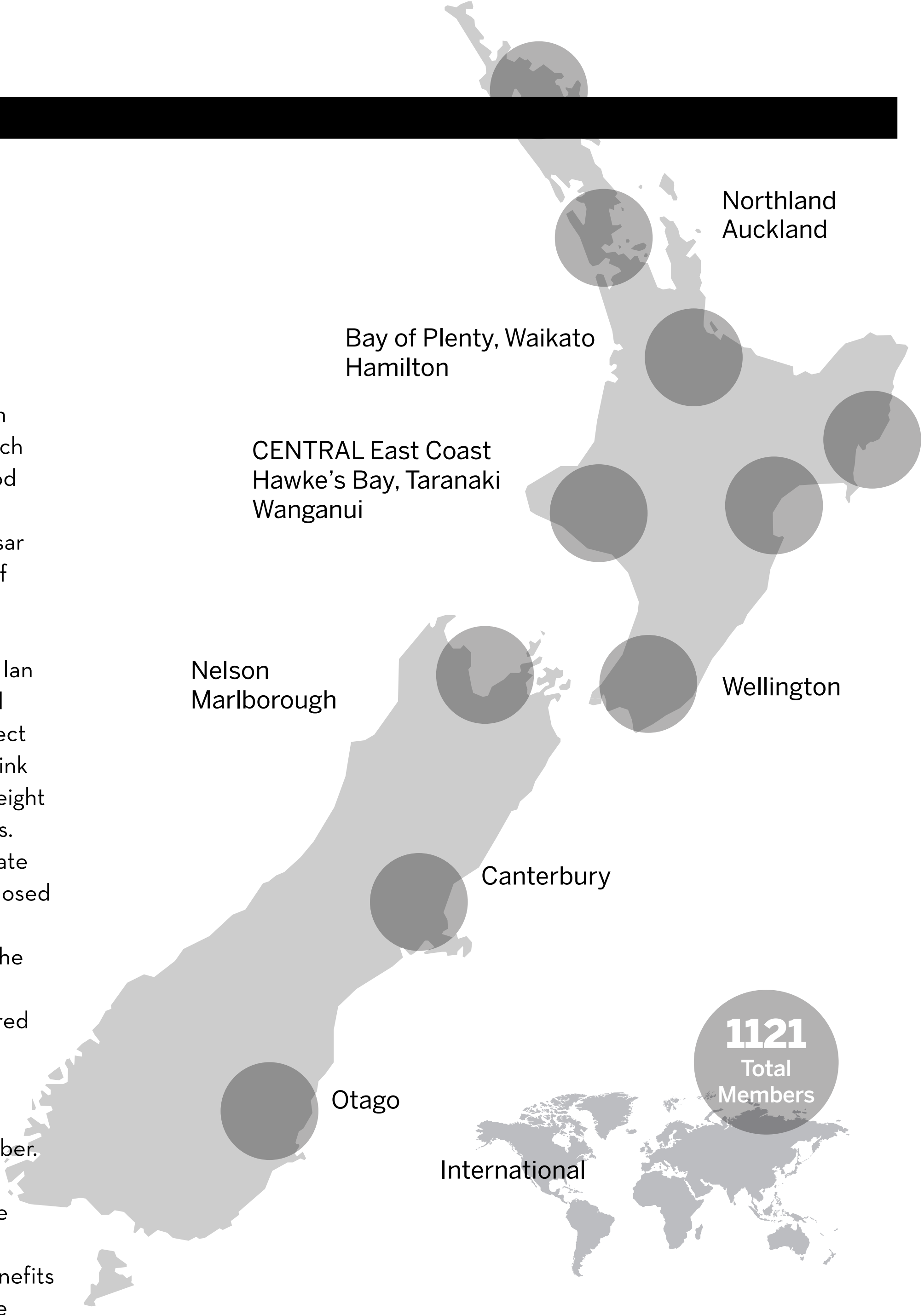
a) Video of unconfined compressive strength test (under testing methods, ex Korea University)

b) Video course on “Key Principles in Rock Mechanics” by Prof Jian Zhao - lesson 1 available

Stuart Read

Stuart Read is an engineering geologist with GNS Science. He obtained his degree, in engineering geology from the University of Canterbury, in 1971. His 43 years of engineering geological consulting and research experience has been in the evaluation, investigation, construction and refurbishment of engineering and mining projects. He has taken a leading role in the development of the rock and soil mechanics laboratory for GNS Science and has research interests in the strength and deformation properties of rock and soil masses.

Branch reports



AUCKLAND

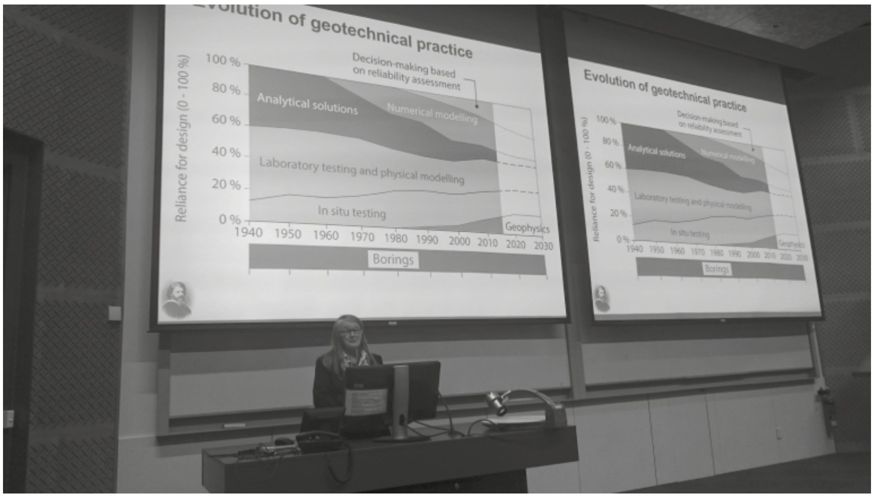
The second half of 2016 has been successful for the Auckland branch with many presentations and good attendance.

In July Mike Dobbie from Tensar presented on the performance of reinforced soil structures under extreme water, soil and seismic loads. In August Ralf Konrad and Ian Manley of Gaia Engineers shifted the focus to a New Zealand project talking about Tauranga Eastern Link and the innovative use of light weight fill to overcome settlement issues. September began with the flatplate dilatometer workshop and was closed out by a fascinating presentation by Professor David Petley from the University of East Anglia on the occurrence of earthquake triggered landslides following large seismic events.

A highlight of the year to date was the Rankine Lecture in October. Dr Suzanne Lacasse from the Norwegian Geotechnical Institute presented on hazard, risk and reliability. She highlighted the benefits of probabilistic modelling and the importance of having open discussions with clients about where risks lie and what the consequences are. With over 130 people in attendance this was one of the largest events the Auckland branch has ever held.

The year will be concluded with three presentations in late November and early December from Brian Perry Civil disseminating learnings in dynamic ground improvement, MBIE on their new rock fall guidelines and Fugro on optimising lab testing.

We would like to extend a thank you to all our sponsors Geofabrics, Gaia Engineers, Ground Investigations, Beca, and Brian Perry Civil that have



Above: This photo is of the Rankin Lecture in Auckland with Dr Suzanne Lacasse.

SEE THE EVENTS DIARY OR WWW.NZGS.ORG FOR FUTURE EVENTS

supported these events.

We would also like to thank Luke Storie for over seven years of service to the Auckland branch. Luke is stepping down as a branch coordinator at the end of the year.

If you are interested in supporting the society by joining the Auckland team or through sponsorship please contact the coordination team Eric and James.

WAIKATO

The Waikato Branch has some activities in planning, but these haven't come into fruition yet. These future Waikato Branch presentations include:

- GDB Replacement (Geostructural aspects of a large bridge replacement) – December/ January
- Update on Waikato Basin Faulting – later this year
- Waikato Expressway – Huntly Section site visit – date TBC

Call to Arms!!! If any members, practitioners, contractors, suppliers have ideas for possible presentations then please contact the NZGS Waikato Branch Co-ordinators Andrew Holland (Andrew@hdc.net.nz) and/or Kori Lentfer (KoriL@CMWgeosciences.com).

BAY OF PLENTY

Bay of Plenty branch members have enjoyed two fantastic events since our last report. In June Dr Vicki Moon and Pip Mills presented the current state of knowledge on the geomechanics, microstructure and cyclic response of sensitive soils in the Bay of Plenty region. Branch members enjoyed refreshments to lubricate the networking before this talk courtesy of Perry Geotech Ltd. September saw Prof. David Petley make a stop on his speaking tour of New Zealand to speak on Earthquake Triggered Landslides and the interesting and complex interactions present in these events. Pro-Drill provided the spread for this event, and even managed to feed the contingent of Waikato Earth Science students who attended. It was great to see the next generation of geotechnical professionals mingling with the current.

In November we have Andy Dodds booked in to present on the Gerald Desmond bridge – a topic sure to generate interest from a wide cross section of the Bay of Plenty engineering fraternity. And early in the new year we have a local supplier keen to throw us a welcome back barbecue which is sure to be popular on a sunny Tauranga evening.

We have received continued support from the Bay of Plenty Polytechnic with a venue in central Tauranga. We express our thanks to the polytechnics Civil Engineering program for this support.

Attendance at our events this year has been fantastic and we look forward to the continued enthusiasm of our local branch members. If you have a site or project which you would like to show off to your fellow geotechnical professionals, please get in touch.

HAWKE'S BAY

Nothing to report.

WELLINGTON

The recent quiet period for the Wellington Branch is well and truly over! Over the last month we have been host to a couple of great quality presentations including the 55th Rankine Lecture on Hazard, Risk and Reliability in Geotechnical Practice presented by Suzanne Lacasse; and Earthquake Triggered Landslides – Understanding Complex Mechanisms, presented by David Petley.

Do you have an idea for your local branch meeting? Your local coordinators are keen to hear your ideas and are always oen to offers of assistance! See the folowing pages for a list of friendly contacts

Above: September saw Prof. David Petley make a stop on his speaking tour of New Zealand to speak on Earthquake Triggered Landslides

Above: The 55th Rankin Lecture in Wellington on Hazard, Risk and Reliability in Geotechnical Practice with Dr Suzanne Lacasse.

★ GEO-NEWS WEEKLY E-NEWSLETTER ★
Our new weekly email lists all notices and Branch announcements normally sent to members, but in one email. Please send items to include to **secretary@nzgs.org**

Both of these presentations had a good turnout and were very well received. The Wellington Branch would like to thank Suzanne and John for their time, and also all those who attended.

Leading into the Christmas period we are looking forward to the upcoming presentation on the MBIE Rockfall Protection guidelines. Complete details of the presentations can be found on the NZGS website.

As always we kindly ask for members to get in touch with any ideas they may have for presentations. Do not be shy, we are a friendly bunch and all presentation ideas are eagerly accepted.

Above: Here is a photo from a Christchurch Presentation - Michael Dobie presentation on 27 July if we can add this in the branch area of the society news as a filler

NELSON
Nothing to report.

CANTERBURY
In the last quarter we have had the following presentations:

- Dr Carlo Lai on Non-Conventional Methods for Measuring Low-Strain Dynamic Properties of Geomaterials. Joint presentation with QuakeCoRe. 15 November. About 20 present.
- Rankine Lecture 13 October. About 60 people present.
- Professor David Petley on Earthquake Triggered Landslides Understanding Complex Mechanisms. 14 September. About 40 people present.
- Nigel Wilson from Ground Anchor Systems on Restricted Access Stabilisation. 23 August. About 20 People present.

Turn out is getting better at the presentations overall.

OTAGO
Nothing to report.

AUCKLAND

Luke Storie

Luke is a geotechnical engineer at Tonkin + Taylor. He recently completed his PhD at the University of Auckland on SFSI in the earthquake performance of multi-storey buildings on shallow foundations. Previously, following graduation from the University of Auckland with a BE(hons) and BA conjoint degree in 2009, Luke worked as a graduate geotechnical engineer on projects in New Zealand and Australia.

luke.storie@gmail.com

Eric Torvelainen

Eric is passionate about soil stiffness, SSI and liquefaction. A Canterbury graduate, he works in T&T using numerical methods to solve complex problems, such as wind turbine foundations, bridges, multi-storey and in-ground structures.

ETorvelainen@tonkin.co.nz

James Johnson

ames is a Senior Geotechnical Engineer with Beca Ltd in Auckland. He has a BSc (Hons) (2009) in geophysics and mathematics and a MEngSt (Hons) (2012) in geotechnical engineering from the University of Auckland. He has worked on variety of large infrastructure projects around New Zealand, Europe, and North Africa where he has gained significant experience in soil-structure interaction.

James.Johnson@beca.com

SEE THE EVENTS DIARY OR WWW.NZGS.ORG FOR FUTURE EVENTS

WAIKATO

Kori Lentfer

Kori is a Engineering Geologist. He graduated in 1998 with a BSc(Tech) in Geology, followed by Masters study at Waikato University and an MSc thesis in Engineering Geology from Auckland University in 2007. Kori has worked for consultants based in the UK, Europe and the Middle East.

koril@cmwgeosciences.com

Andrew Holland

Andrew is a Director of HD Geotechnical. He studied engineering at the University of Auckland, graduating in 2002.

Andrew's experience includes geotechnical investigation, assessment and design for infrastructure, buildings and development. Andrew is a Chartered Professional Engineer (CPEng).

Andrew@hdc.net.nz

BAY OF PLENTY

James Griffiths

James is an Engineering Geologist with Beca in Tauranga. After a previous life working in outdoor education and guiding on the Fox Glacier for 7 years, James studied Geology at Otago University, graduating in 2014 with a BSc (Hons). James has worked on site hazard assessments, geotechnical site investigations and ground modeling for a broad range of clients and market sectors.

James.Griffiths@beca.com

Kim Rait

Kim is a Geotechnical Engineer with Beca Ltd. She completed a BSc(Hons) in Mathematics and Statistics at the University of Canterbury before working in accountancy for several years. Kim then returned to UC to complete a PhD in Geotechnical Engineering and has been working at Beca on various small projects over the last year while completing her thesis.

Kim.Rait@beca.com

WELLINGTON

Ayoub Riman
Ayoub is a senior geotechnical engineer with more than 10 years of experience gained in several countries in the Middle East, Africa, Australasia and Europe. He has experience in the analysis and design of foundations, soil improvement and treatment, deep excavations, cut and cover tunnels, land reclamation, slope stability, seismic assessments
ARiman@tonkin.co.nz

Dolan Hewitt
Dolan is an engineering geologist with five years of experience. Dolan has worked in Western Australia in mine resource geology and planning. He now works for Opus and has been involved in geotechnical investigations and risk assessments for infrastructure and land development throughout New Zealand.
Dolan.Hewitt@opus.co.nz

NELSON

Paul Wopereis
Paul is Principal Engineering Geologist with MWH Global based in Nelson.
Paul has worked at MWH since 2001 and is currently involved in projects in New Zealand and Fiji. Previously Paul was a senior exploration geologist with L & M Mining Ltd and has worked on mining and exploration projects in New Zealand and South America.
Paul.J.Wopereis@mwhglobal.com

CANTERBURY

Jennifer Kelly
Jen is a Senior Engineering Geologist working for Riley Consultants in Christchurch. She has a BSc (Hons) geoscience from St Andrews (2004) and an MSc in geotechnical engineering and management from Birmingham University (2011). She worked in the UK for 8 years on large infrastructure projects before moving to NZ in 2013 and gaining great experience here and in the Pacific islands.
jkelly@riley.co.nz

Sam Glue
Sam is a Geotechnical Engineer working for Tonkin & Taylor in Christchurch with 9 years experience working throughout New Zealand and Australia. Sam graduated from Canterbury with a BE (Civil) in 2006 and is passionate about being involved in the construction of major infrastructure projects that will withstand the test of time and earthquakes.
SGlue@tonkin.co.nz

OTAGO

David Barrell
David is a geologist and geomorphologist at GNS Science in Dunedin. South Island born and bred. Since joining GNS Science, 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.
d.barrell@gns.cri.nz

Teresa Roetman
I live up in the Waitakere Ranges in Auckland, far from the rush of traffic and noise. Sitting at my desk, looking out to the bush clad hills full of birds happily chirping in the sun I feel blessed to be part of this wonderful environment. I love these hills, hiking the tracks with my son and daughter, paddling in the rivers and streams, seeing weta's and glowworms, hearing the wildlife, not to mention the fantastic views of the surrounding city. We love the west coast beaches, the black sand, the wild surf. When I am not working for the NZGS I enjoy all the "wild west" has to offer.

Please remember to contact the Management Secretary (Teresa) if you wish to update any membership, address or contact details. If you would like to assist your Branch, as a presenter or sponsor, or to provide a venue, refreshments, or an idea, please drop a line to your Branch Co-ordinator or Teresa. If you require any information about other events or conferences, the NZGS Committee and NZGS projects, or the International Societies (IAEG, ISRM and ISSMGE) please contact the Secretary on secretary@nzgs.org You may also check the Society's website for Branch and Conference listings, and other Society news: www.nzgs.org

Management committee

POSITION	NAME	Email
Chair	Charlie Price	Chair@nzgs.org
Immediate Past Chair	Gavin Alexander	Gavin.Alexander@beca.com
Vice Chair and Treasurer	Tony Fairclough	Treasurer@nzgs.org
Elected Member	Kevin Anderson	Kevin.Anderson2@aecom.com
Elected Member	Guy Cassidy	GCassidy@Engeo.co.nz
Elected Member	Eleni Gkeli	Eleni.Gkeli@opus.co.nz
Elected Member	Sally Hargraves	sally@tfel.co.nz
Management Secretary	Teresa Roetman	secretary@nzgs.org
Co-opted Member	Ross Roberts	editor@nzgs.org
NZ Geomechanics Co-Editor	Don Macfarlane	editor@nzgs.org
Co-opted NZ Geomechanics Co-Editor	Marlene Villeneuve	editor@nzgs.org
YGP representative (Co-opted member)	Frances Neeson	frances.neeson@opus.co.nz
IAEG Australasian Vice President	Mark Eggers	Mark.Eggers@psm.com.au
IAEG NZ Representative	Doug Johnson	DJohnson@tonkintaylor.co.nz
ISSMGE Australasian Vice President	Mark Jaksa	Mark.Jaksa@adelaide.edu.au
ISSMGE NZ Representative	Mick Pender	M.Pender@auckland.ac.nz
ISRM Australasian Vice President	Stuart Read	S.Read@gns.cri.nz Management Committee

EDITORIAL POLICY

***NZ Geomechanics News* is a biannual bulletin issued to members of the NZ Geotechnical Society Inc.**

Readers are encouraged to submit articles for future editions of NZ Geomechanics News. Contributions typically comprise 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 of any length**
- **feedback on papers and articles published in NZ Geomechanics News**
- **news or technical descriptions of geotechnical projects**
- **letters to the NZ Geotechnical Society or the Editor**
- **reports of events and personalities**
- **industry news**
- **opinion pieces**

Please contact the editors (editor@nzgs.org) if you need any advice about the format or suitability of your material.

Articles and papers are not normally refereed, although constructive post-publication feedback is welcomed. 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 will be forwarded to the author for a right of reply. The editors reserve the right to amend or abridge articles as required.

The statements made or opinions expressed do not necessarily reflect the views of the New Zealand Geotechnical Society Inc.

NEW ZEALAND GEOTECHNICAL SOCIETY INC

NEW ZEALAND GEOTECHNICAL SOCIETY INC

The New Zealand Geotechnical Society (NZGS) is the affiliated organization in New Zealand of the International Societies representing practitioners in Soil mechanics, Rock mechanics and Engineering geology. NZGS is also affiliated to the Institution of Professional Engineers NZ as one of its collaborating technical societies.

The aims of the Society are:

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.

All society correspondence should be addressed to the Management Secretary (email: secretary@nzgs.org).

The postal address is
NZ Geotechnical Society Inc,
P O Box 12 241,
WELLINGTON 6144.

Letters or articles for NZ Geomechanics News should be sent to editor@nzgs.org.

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

Full details of how to join are provided on the NZGS website http://www.nzgs.org/about/

ADVERTISING
NZ Geomechanics News is published twice a year and distributed to the Society's 1000 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. NZGS aims to break even on publication, and is grateful for the support of advertisers in making the publication possible.

TYPE	BLACK AND WHITE	COLOUR	SPECIAL PLACEMENTS		SIZE
			INSIDE FRONT OR BACK COVER	OPPOSITE CONTENTS PAGE	
Double A3	-	$1400	$1600 (front A3)		420mm wide x 297mm high
Full page A4	$600	$700	$1000	$1000	210mm wide x 297mm high
Half page	$300	$350	-		90mm wide x 265mm high 210mm wide x 148mm high
Quarter page	$150	$175	-		90mm wide x 130mm high

Flyers/inserts	From $700 for an A4 page, contact us for an exact quote to suit your requirements as price depends on weight and size.

Notes
1. All rates given per issue and 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
5. Advertisers are responsible for ensuring they have all appropriate permissions to publish. This includes the text, images, logos etc. Use of the NZGS logo in advertising material is not allowed without pre-approval of the NZGS committee.

National and International Events

2016

8-10 DECEMBER 2016
Bengaluru, India
5ICFGE - 5th International Conference on Forensic Geotechnical Engineering

2017

18-20 JANUARY 2017
Villars - Switzerland
International Workshop - Advances in Multiphysical Testing of Soils and Shales

14-16 FEBRUARY 2017
Iran
5th International Conference on Geotechnical Engineering and Soil Mechanics

12-17 FEBRUARY 2017
Cape Town, South Africa
AfriRock 2017 - International Symposium

19-22 FEBRUARY 2017
Denver, Colorado, USA
Engineering Solutions for Sustainability: Materials and Resources (ESS: M&R 3) Symposium

5-9 JUNE 2017
Switzerland
PRF2017 – Progressive Rock Failure
https://www.prf2017.ethz.ch/

13-15 JUNE 2017
Ostrava, Czech Republic
International Symposium EUROCK 2017

20-22 JUNE 2017
Ostrava, Czech Republic,
EUROCK 2017 International Symposium

25-28 JUNE 2017
San Francisco, California, USA
51st US Rock Mechanics Geomechanics Symposium (ARMA2017)

15-19 JULY 2017
Sharm el-Sheikh, Egypt
GeoMEast2017 - Innovative Infrastructure Geotechnology

16-19 JULY, 2017
Vancouver, Canada
PBD-III Vancouver 2017 – The 3rd International Conference on Performance Based Design in Earthquake Geotechnical Engineering

1-6 SEPTEMBER, 2019
Reykjavik, Iceland
ECSMGE 2019 – XVII European Conference on Soil Mechanics and Geotechnical Engineering

6-7 SEPTEMBER 2017
Leeds, UK
Second International Symposium on Coupled Phenomena in Environmental Geotechnics

16-17 SEPTEMBER 2017
Seoul, Republic of Korea
6th International Young Geotechnical Engineers' Conference (6th iYGEC)

17-22 SEPTEMBER 2017
Seoul, Korea
19th ICSMGE-Seoul 2017 - Unearth the Future Connect Beyond

2-7 OCTOBER 2017
Capetown, South Africa
AfriRock 2017 – International Symposium

23-26 NOVEMBER 2017
Napier
NZGS 20th Symposium - What In Earth Is Going On: Balancing Risk Reward, Regulation and Reality

29 NOV - 1 DECEMBER 2017
Lake Wanaka Centre, Wanaka
Geoscience Society NZ conference

LINKS ARE AVAILABLE FROM THE NZ GEOTECHNICAL SOCIETY WEBSITE WWW.NZGS.ORG

★
UPCOMING EVENTS
★

2017
TWO COURSES TO LOOK OUT FOR

A short course on Engineering Geology, towards the end of the 1nd half of 2017

Short courses on Soft Soil Engineering and Rock and Soil Slope Stability later in 2017

2019

JUNE 2019
Rome, Italy
7 ICEGE 2019 – International Conference on Earthquake Geotechnical Engineering

The Earthquake geotechnical engineering guidelines

FEEDBACK is requested to modulefeedback@nzgs.org

Overview of the Guidelines Series
Released February 2016 Geotechnical investigation for earthquake

Engineering
Released November 2016

Identification, assessment and mitigation of Liquefaction hazards
Released July 2010 as 'Module 1' Updated May 2016

Earthquake resistant foundation design
Released November 2016

Ground improvement of soils prone to
Document under review

Liquefaction

Specification of ground improvement for residential properties in the Canterbury region
Release November 2015

Seismic design of retaining walls
Development to commence in 2017, Expected release in 2017

Earthquake slope stability
Development to commence in 2017

MINISTRY OF BUSINESS, INNOVATION & EMPLOYMENT
HĪKINA WHAKATUTUKI

NEW ZEALAND GEOTECHNICAL SOCIETY INC
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