NEW ZEALAND GEOTECHNICAL SOCIETY INC

DECEMBER 2017 issue 94

NZ GEOMECHANICS NEWS

Bulletin of the New Zealand Geotechnical Society Inc.

ISSN 0111–6851

SPECIAL FEATURE 2017 NZGS SYMPOSIUM

NON-LINEAR SOIL MECHANICS

UNDERSTANDING PATTERNS OF MOVEMENT OF SLOW LANDSLIDES

APPLICATION OF THE GOOD CHEAP FAST RULE IN GEOTECHNICAL ENGINEERING

RETAINING YOUR BUSINESS IS OUR BUSINESS

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Construct

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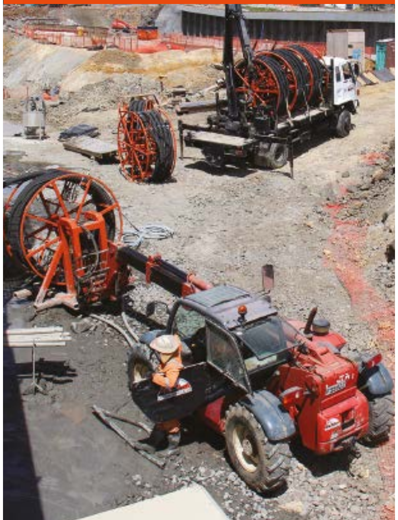

Improve

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.

With ground anchor technology advancing all the time, our association with Samwoo means our New Zealand clients will continue to have access to world-leading technology including:

- Removable compressive distributive anchors (SW-RCD)
- Removable distributive tension anchors (SW-SMART)
- Permanent compressive distributive anchors (SW-PCD)
- Permanent tensile frictional anchors (SW-PTF)

We have successfully completed some 5000m of removable anchors and the level of enquiry continues to rise as the sustainable benefits of this technology are realised - once a construction project is completed, there is nothing left in the ground that will obstruct future developments on adjoining properties.

Samwoo's anchor technology is economical, efficient and another way for Grouting Services to remain at the forefront of our industry.

If you're interested in working with us or finding out more about the results we've achieved for our clients call 09 837 2510 or visit our website.

Our multidisciplinary operation specialises in the fields of ground anchoring, soil nailing, drilling, post-tensioning and grouting. The combination of capability and depth of technical expertise makes us a market leader and supports our reputation for providing value engineered solutions to our customers.

GROUND ANCHORING	SOIL NAILING	GROUTING
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We're proud to be the sole distributor in New Zealand for Samwoo Anchor Technology, BluGeo GRP60 Bar, OVM Prestressing Systems, Tighter (Kite) Earth Anchors and Grout Grippa Grout Sock (Australasia).

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With drilling depths up to 25m and pile diameters up to 1,200mm, Brian Perry Civil is taking CFA piling in New Zealand to the next level.
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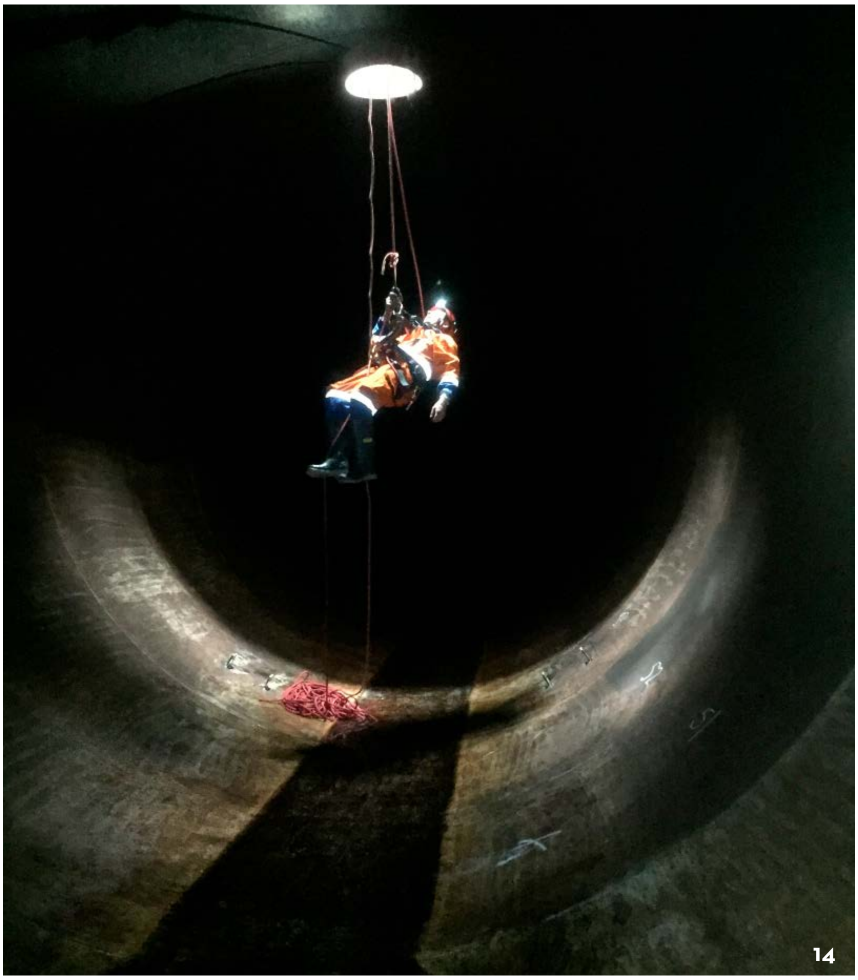


COVER IMAGE: NK2 Headrace, by John Underhill, Aecom in Auckland.

Tony is a Christchurch based University of Auckland educated civil and geotechnical engineer with over 27 years' experience. He has worked on projects throughout New Zealand, Australia, Malaysia (resident for 4.5 years), Vietnam, Hong Kong (resident 1.5 years), Fiji, The Solomon Islands, Vanuatu, the United Arab Emirates, and, the USA. Tony is now the NZGS Chair and employed by Tonkin & Taylor Ltd as their South Island Geotechnical Co-ordinator. Tony previously worked for Worley Consultants Ltd (1986 - 1990), Soil and Rock Consultants Ltd (1991 – 1993), and Woodward Clyde (NZ) Ltd / URS (NZ) Ltd (1993 – 2000).

Tony Fairclough
Chair, Management Committee

IT IS MY great pleasure and honour to be writing my first "Chair's Corner" report. As most of you will know, Charlie's term as NZGS Chair ended on 30 September 2017, and, I took the helm on 01 October.

Over these past five years, as a result of the Canterbury Earthquake Sequence and subsequent Royal Commission, the New Zealand Geotechnical Society has been directly involved in the development and issue of over 16 industry guidelines, reference publications and standards. This caused an unprecedented volume of work for your management committee, with all of the committee members being required to provide hundreds of hours volunteer input during each year of their service.

Gavin Alexander and Charlie Price were the NZGS Chair during the 2012-2014 and 2014 – 2017 terms respectively. I estimate that both Gavin and Charlie have been required to provide well over fifteen-hundred hours of service to the society during their term as Chair – a phenomenal effort by any measure.

During their terms Gavin and Charlie worked closely with the other key stakeholders, such as MBIE and EQC, to ensure all of the publications which carry the NZGS logo were of a high standard, relevant, up-to-date, practical, and most importantly, of tangible value and assistance to the wider industry. It is my humble opinion that these objectives were achieved in every case.

For the reasons described above, I thank Gavin and Charlie for their devoted service to the society over the past five years, and ask that each member who reads this article buys them a drink next time they see them.

At the time of writing this article, the 20th NZGS Symposium had just concluded. I thank all of the symposium organising committee for their hard work and service to the society over the past 18 months. All of you did a fantastic job and should be extremely proud of involvement in this noteworthy event. In particular I thank the Symposium Convenor, Mr Pierre Malan.

Dozens of symposium attendees have contacted me over the past few days, the overwhelming majority of whom have provided me with positive feedback for this event. Many of the members who I have spoken with stated that they believe the 20th NZGS Symposium was the best one they have ever attended. I have also received several suggestions for improvement, which are gratefully received, and will be appropriately documented to assist in the planning of future events.

I whole-heartedly agree with the majority in this case, and believe that the 20th NZGS Symposium was an extremely well run and structured event, and, all attendees would have gained valuable Continuing Professional Development (CPD) experience. The format of this symposium was structured quite differently to our more recent symposiums, with it being run as a "single stream symposium with nominated session chairs". Our older members will remember that this was the traditional format for the NZGS symposiums up to the late 1980's or early 1990's. This "old school" format has proved to be relatively popular, and I anticipate that at least some of our future symposiums will follow a similar structure.

Planning for the 21st NZGS Symposium will start during the December 2017 Management Committee meeting. Please do not hesitate to contact Teresa Roetman (secertary@nzgs.org) or my good self (chair@nzgs.org) if you would like to be part of the organising committee for our next symposium, or, if you wish to provide additional feedback on the recently-concluded 20th NZGS Symposium.

During the dinner function for the 20th NZGS symposium, the honour of "Life Member" was officially bestowed to the following three esteemed members for their outstanding service to the society:

- David Burns
- C.Y. Chin, and,
- Mike Stannard.

I warmly congratulate Dave, Chin and Mike and thank each and every one of them for their exceptional contribution and support of the New Zealand Geotechnical Society. We are a stronger, and more effective organisation as a direct result of your significant contributions.

During my term as Chair I intend to ensure that NZGS continues to focus on the following objectives which I believe are of high benefit to our members:

i) The preservation of a strong, collaborative working relationship with our relevant sister technical societies and key industry partners (i.e. SESOC, NZSEE, NZSOLD, CETANZ, MBIE, EQC).

ii) Pro-active planning and co-ordination of industry training programs which are of high value to our members and the wider industry,

iii) The co-ordination and sponsorship of visits and lectures by note-worthy national and international experts throughout New Zealand, and,

iv) Initiate ways to make the NZGS training programs and lecture series more accessible to members who are based outside of the main centres.

Please do not hesitate to contact Teresa Roetman, Eleni Gkeli or myself if you have any suggestions for future training courses or international speakers that you believe would provide valuable insight and/or raise the technical skills of our members in key areas of weakness.

Finally, I wish to bring to your attention the following upcoming conferences:

- The 12th Australia and New Zealand Young Geotechnical Professional Conference (12th ANZ YGP), Hobart; Tasmania, 07 to 09 November 2018,
- The 13th Australia New Zealand Conference on Geomechanics, Perth; Western Australia, 01 to 03 April 2019, and,
- The 20th International Conference on Soil Mechanics and Geotechnical Engineering, Darling Harbour; New South Wales, 12 to 16 September 2021.

Further details for the above conferences is provided within this issue of the Geomechanics News, on the Australian Geomechanics Society website (http://www.australiangeomechanics.org/) and on the New Zealand Geotechnical Society website (http://www.nzgs.org/). Please consider attending at least one of these events, and, note that the deadline for the 12th ANZ YGP nominations and abstracts is 15 February 2018.

I look forward to serving you all over the next two years in my capacity as NZGS Chair. Please do not hesitate to contact me via email on chair@nzgs.org if you wish to discuss any issue which you believe is of direct relevance to our membership,

Best Regards

Tony Fairclough
NZGS Chair, 2017-2019

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

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.

THE CLOSURE OF 2017 is a celebration of the strength of our society and our members. In this issue we focus on the 20th NZGS Symposium hosted by Napier on 23-26 November 2017. Pierre Malan and the organisation committee put a tremendous amount of work into organising the conference, MBIE Workshop on the 23rd November and the field trip on 26th November. We have re-published the two keynotes and two of the best conference contributions in this issue, for those who were not able to attend. We also have an international conference contribution from Gavin Alexander.

As we have closed the NZGS Symposium it is now time to start thinking about the 20th ICSMGE conference, which, it has just been announced, will be hosted by Sydney! This is the time for members to keep their eyes out for interesting case studies and to think about highlighting lessons learned from the wide variety of work we do in New Zealand. It is also time for companies to think about how they will support and encourage their employees to attend conferences.

We had a large number of submissions for the photo comp with the theme "better than being at the office". Clearly working in the field remains a core part of our business, and we are lucky to work in a number of interesting and photogenic locations.

The first series of engineering geology short courses ran in September very successfully. Look out for the report and photos from the course. Practitioners should also look out for the key changes to AGS 4. We have highlighted the key changes to make it clearer. Our young professionals are very active lately, with representation to the International YGP conference this year and preparations for the Australia-New Zealand conference in 2018. Let's keep up the great momentum and support for our young professionals.

We also have some updates from the NZGS and the international societies, including three new presidents. We welcome Tony Fairclough as the new NZGS chair, with the full committee elected in September given towards the end of this issue. We also have new ISSMGE and ISRM presidents.

Finally, this is my fifth issue and I will be transitioning out of this role over the next year. I would like to send out a call for volunteers for the role of Geomechanics News co-editor, to serve alongside Don Macfarlane and me for the June 2018 issue, after which I will step down.

Marlène Villeneuve

SYDNEY ICSMGE 2021

A GEOTECHNICAL DISCOVERY DOWN UNDER

ICSMGE 2021 IS COMING TO SYDNEY

Thank you for supporting the Australian Geomechanics Society's bid to host the 20th International Conference on Soil Mechanics and Geotechnical Engineering in Sydney, Australia in 2021.

We can't wait to welcome our profession's best and brightest minds from across the world to our International Convention Centre (ICC Sydney) on the stunning Darling Harbour.

It's an exciting time for our profession here in Sydney, with major redevelopment projects using new and innovative technology and techniques that will help the future of geotechnical engineering.

Join us in the harbour city in 2021 for a Geotechnical Discovery Down Under!

Visit us at www.icsmge2021.org for more information.

SEE YOU IN SYDNEY!

AUSTRALIAN GEOMECHANICS SOCIETY

News – In Brief

New Zealand **GEOTECHNICAL DATABASE STATISTICS**

John Scott

John is a CPEng with 30 years national and international geotechnical and project management experience in major civil engineering projects. He has a BE (Civil) degree for the University of Canterbury and a MEngSc degree from the University of New South Wales and is PMP qualified. John is currently a Senior Advisor in Reinsurance Research & Education for the Earthquake Commission. John previously worked for MBIE's Facilitating Canterbury Rebuild Team as their Geotechnical Advisor and for CERA assisting their policy advisors with the Port Hills red zoning policy. John has an interest in CPT testing dating back to 1987 where he helped prepare a design guide for use of CPT for the then Works Consultancy Services (now Opus).

The following graphics are some key statistics that may be of interest to NZGD users. The NZGD is co-funded by MBIE and EQC.
The makeup of the NZGD users by profession and year is varied, but dominated by geotechnical engineers (Figure 1), as would be expected.

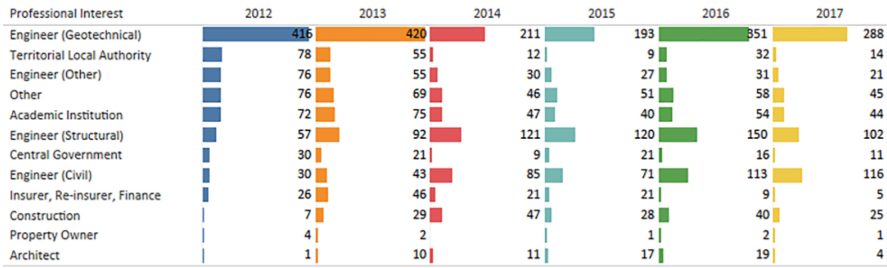
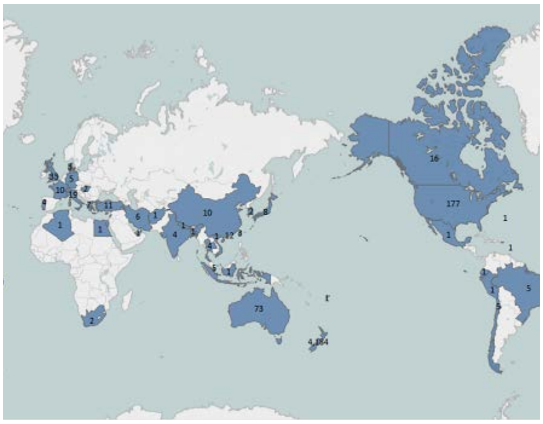
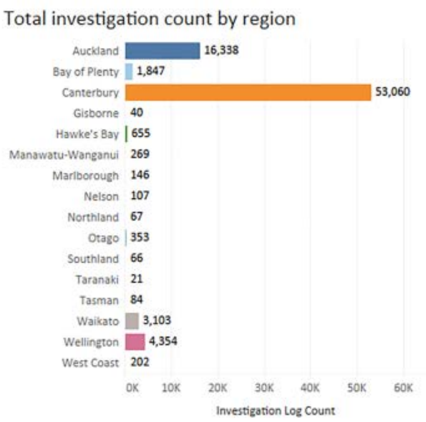


Figure 1: Users of the NZGD by profession



The approximately 4500 current users are spread internationally as shown in Figure 2. This illustrates the significance of this NZGD as an international research tool.

Figure 2: Users of the NZGD by location (International)



The spread of data throughout the country is shown in Figure 3. This spread, while encouraging in some areas of the country, also illustrates there is still some work to do in several areas including Gisborne, the western and central North Island and the lower third of the South Island. If there is anything you can do to help upload new data please don't hesitate to send a message to the NZGD support email.

Figure 3: Contributors to the NZGD by location (New Zealand)

FREE RAPID ASSESSMENT WORKSHOPS FOR GEO-PROFESSIONALS

GEOTECHNICAL PROFESSIONALS WHO are interested in applying their skills in a civil defence response are invited to attend a free MBIE workshop in early 2018.

The workshops will be run in Auckland, Wellington and Christchurch, and provide training on how to assess geo-hazards that could pose a life-safety risk to building occupants, as part of a rapid building assessment process. Once trained, participants will be added to MBIE's register of rapid building assessors who can be called on to help during a state of emergency, or during a lesser event in special circumstances.

This course is for geo-professionals (especially CPEng and PEngGeol) with at least three years' experience, including the assessment of geo-hazards and/or land instability.

The geotechnical field guide, published by MBIE earlier this year, is the accompanying resource and provides further information on the assessment process.

The all-day programme will include training on:

- the Civil Defence Emergency Management structure
- the rapid geotechnical assessment process and the role of the geo-professional in an emergency event
- identifying geotechnical life-safety hazards and standard descriptions
- decision-making around application of placards.
- completing assessment forms
- interacting with residents and building owners
- safety and wellbeing.
- Register to attend a workshop. https://mbie.wufoo.eu/forms/s1r0mgkg12171zb/

Please note that limited spaces are available.
Email your questions to buildingtraining@mbie.govt.nz

MINISTRY OF BUSINESS, INNOVATION & EMPLOYMENT
HĪKINA WHAKATUTUKI

AGS Data Transfer Format UPDATED *for New Zealand*

IN RESPONSE TO international changes, and some local requirements, our version of the Association of Geotechnical and Geoenvironmental Specialists (AGS) data transfer format has been updated and is now available for your use on the NZGS library (www.nzgs.org/library).

The concept of transferring geotechnical data between all industry parties in electronic format is widely used internationally and in New Zealand. The adoption of a standardised data transfer file format provides a significantly improved ability to access geotechnical data and will allow greater utilisation of the information obtained. The initial concept for the transference of geotechnical data in an electronic format was first identified and established a number of years ago in the UK by the Association of Geotechnical and Geoenvironmental Specialists (AGS – http://www.ags.org.uk). Since its initial release in 1992, the AGS format has undergone a series of successive updates that have reflected the needs of the industry. The AGS format is now in its Fourth Edition.

By basing the New Zealand version of the Format closely on the AGS (UK) Format, benefit is gained from the experience and lessons learned over the last 20 years as the Format has developed. We have now updated our version. The changes reflect the changes made internationally so that we remain compatible with international practice, as well as minor adjustments to New Zealand localisations (fully compatible with the international edition) to reflect changes in local practices.

The modifications are summarised in the document to enable easy updating of your databases or correspondence files. Examples include the additional SPT requirements of the Earthquake Geotechnical Engineering Module 2, and the increased use of the sDMT in everyday practice.

Please send comments to Modulefeedback@nzgs.org

CETANZ CPT GROUP

AS MANY OF you may be aware, a technical group has been set up in CEATNZ to represent the CPT operators in NZ. This has been referred to simply as the "CPT Group". The main driving force for setting up this group was to develop a minimum quality standard for the industry. This is to help provide engineers, clients and stakeholders with confidence in the quality of the data coming from the CPT contractors.

All 22 known CPT contractors in NZ have now joined this group. This shows an industry wide commitment to improving and maintaining high quality standards. The CPT Group has decided to adopt the ISO 22476-1 standard over the ASTM standard that is in current use. The process is to be started by a round of appraisals by a third-party auditor. Graeme Duske, a highly experienced geotechnical technician, has been appointed the auditor. He will be assisted by Allan McConnell of IGS in Australia. Each contractor will be assessed undertaking a CPT test against the ISO standard. Information obtained from these appraisals will form a picture of the current state of the industry. It will also highlight any issues that there may be with the ISO standard and how those are being dealt with by individual contractors. The information obtained will be collated and assessed with the intention of developing an industry best practice guideline. The best practice guideline, with the ISO standard at its core, will form the basis for a later round of audits. By passing those later audits the contractors will be award a certificate to demonstrate that they have met the required standard of quality.

The first round appraisals will be undertaken between January and March 2018. The best practice guideline is expected to be completed by October. NZGS members and other interested parties will be invited to comment on the guidelines before they are finalised. We are aiming to have all contractors audited to those guidelines by the end of 2018. From then on it is expected that the audit certification will be the benchmark for assessing competence in CPT testing. The longer term goal is to move towards IANZ accreditation for the test.

For more information regarding the CPT Group, please contact Marco Holtrigter: marco@g-i.co.nz

20TH NEW ZEALAND GEOTECHNICAL SYMPOSIUM NAPIER 23-26TH NOVEMBER 2017

Far left: Mike Stannard (left) and John Scott (MBIE).

PHOTOS: Teresa Roetman

13th Australia New Zealand Conference on Geomechanics

Perth Convention & Exhibition Centre 1—3 April 2019

The Organising Committee for the 13th Australia New Zealand Conference on Geomechanics is pleased to announce that Abstract Submissions will open on 1st February 2018. Details on the wide range of event themes are available on the Conference website.

AUSTRALIAN GEOMECHANICS SOCIETY

Join us in Perth, Western Australia

www.anzgeomechanics2019.org

anzgeomechanics2019@arinex.com.au

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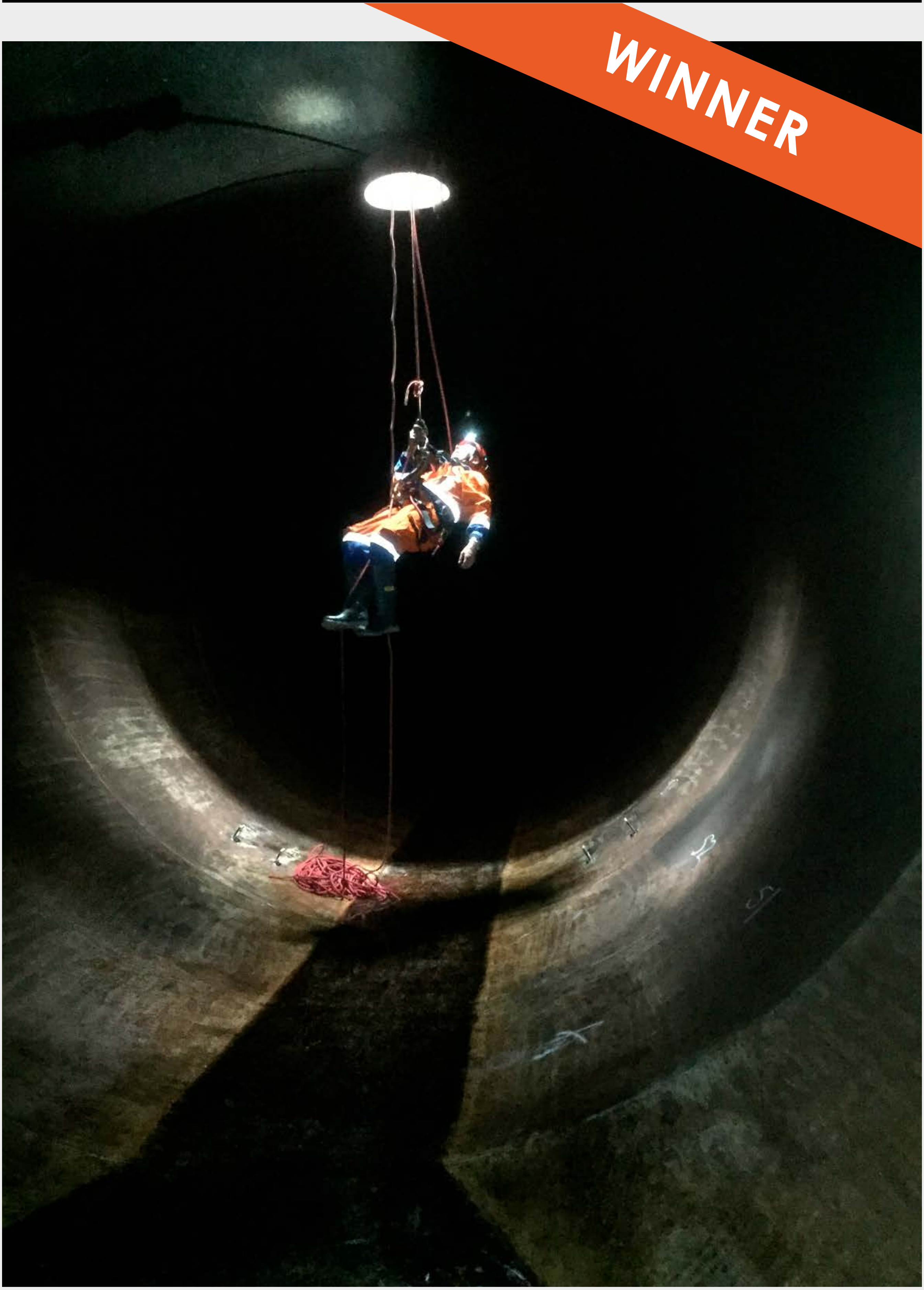
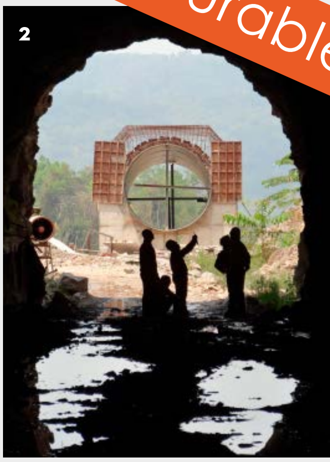


Above: Steve Bowden going into the penstock to carry out 3Dscanning for monitoring purposes, including Geotech issues. Site Maraetai hydroelectric power station, Mangakino. Photo: **Andres Martinez,** *Cheal Consultants Ltd, Taupo.*

honourable mentions



1. Sumner geotechnical remediation project, Christchurch. Photo: **Regan King**, *Jacobs, Christchurch*. **2.** NK2 Headrace, Photo: **John Underhill**, *Aecom in Auckland*. **3.** This photo was taken as abseilers worked to remove a block from the face using an airbag, State Highway 1, Kāikoura. Photo: **Jono Claridge**, *Opus, Christchurch*. **4.** RDCL's Rig03 - CPT testing to 60m at Padcal Mine, Philipines. Photo: **Tom Grace**, *RDCL, Hastings*. **5.** "The lazy geologist". The photo was taken in 2013 during my first year of working. The site is located in Gulf Harbour, Auckland and I was onsite to log machine borehole core and to complete a few hand auger tests. There just happened to be a couple of comfy seats on the property for me to have lunch and take in the views of Auckland city and Rangitoto Island. Photo: **Will Mathieson**, *KGA Geotechnical, Christchurch*. **6.** Drilling platform for new bridge at Kawarau Falls, Queenstown. Photo: **Jeff Bryant**, *Geoconsulting Ltd., Queenstown*.

Finding Good Drillers in New Zealand

Mel Griffiths
President New Zealand Drillers Federation

OVER THE PAST month I have been talking to quite a few members of the NZDF and what I am hearing is that most sectors of the industry seem to be very busy with no signs of slowing.

There is a boom at the moment in the building, construction and infrastructure sector in New Zealand, and this is driving the Geotechnical market including stabilisation work throughout the country. There are some large projects on around the country that have combined with the very wet weather over the past month to increase further works for stabilisation companies. This sector is looking very healthy and keeping everyone busy. The Water sector is not in the boom time of the high milk prices, but is very steady and most companies that I have talked to seem busy.

The minerals sector has starting to see some movement after a very slow period and there seems to be a bit more movement over the ditch in this area too. The quarry sector has been pretty constant, but this is hard to assess as there are many quarries in New Zealand, so it depends on the infrastructure projects that are close to the quarry and it comes down to supply and demand in these areas. But with all the feedback I am getting, it is a very busy time for most.

STAFFING CONCERN

One thing we all have to think about when things are very busy is staffing and resourcing up to help with demand. The pool of good qualified drillers is pretty low in New Zealand especially when things are busy. Most members I talk to say there is a big struggle to find good drillers. There are a few mineral and RC drillers coming back from Australia and this has been happening for some time, but the diverse nature of construction drilling projects, mainly in the Geotech and stabilisation works, can be hard to cross over.

Mineral and RC experience is obviously a lot different to Geotechnical and ground anchor drilling.

The expectation from Drillers is found to be a hard reality when they come back and they can be disillusioned when they first return to the drilling industry from Australia to New Zealand. This is mainly due to the high wages in the Australian mining Industry compared to a New Zealand Construction wage. The wages in New Zealand are increasing but are far from what they are used to, so this brings other challenges for our member companies.

So to offset this we, as employers have to look at other factors such as lifestyle opportunities to keep the staff and to give them more stable work environments. And only then can we try and keep good drillers here in New Zealand.

Mel Griffiths
Tel: 04 527 7346
Mob: 021 433 137
mel@griffithsdrilling.co.nz

Response to: Finding Good Drillers in New Zealand

THIS NEW ZEALAND DRILLERS presidents report was published in the latest edition (August /September) of the Australian Drilling Magazine and highlights the need for vigilance on the part of your members as per your expectations from your contracted Geotechnical Drilling provider.

Forty years ago when I entered the drilling industry as a geotechnical drillers offsider there was a sticker on the rig which Read (CORES WILL TELL) simply meaning 100% recovery should be the purpose of the trade . This was an impossible task given the rig types in use back then. Bridge sites would take months to complete with extremely poor recovery, which was just the order of the day. Senior engineers today will understand the frustrations of that time and today I still wonder what crystal ball is used to log core loss.

The purpose of our trade is still the same today but the means of attaining it has got a whole lot easier, safer, and certainly faster than ever before using 2017/18 technology. These new drill rigs themselves are without a doubt sophisticated works of art in the hands of the fully trained operators. 30 meter deep bridge pile investigation holes through gravel, sands silts and cobbles with 100 % core recovery using 150 hertz sonic technology is now the norm in a day work.

With the high production rates on the new technology rigs means that as an employer I require far less staff to achieve the required results. Combining that with 100% core recovery means work time on site is cut to a quarter, making retention of drillers within the company at an all-time high.

Sonic Technology has allowed us the ability to have drilled 275,000 meters of drilling since 2012 and allows us to carry out SPT testing continuously if required.

Staff all have Manufactures Operators Competency Certification (MOCC) which assures you that the operators are certified on the rig they run and all our operators have between 12 to 22 years' service with Pro drill on geotechnical drilling.

In addition to the 3 new Duo sonic rigs (which can operate in either Rotary core or Sonic mode), we also operate 3 new SLG rotary core rigs for tight access wire line coring projects, and 2 x 6622 geoprobe CPT rigs with SCPTu systems, and members of CETANZ.

Pro Drills overall operational performance is due to

1) Operating the latest technology drilling rigs and systems
2) Fully trained and certified operators
3) Drill Tracker operational software management system

The BOOM that is talked about in the article is real, and so is the lack of competent operators as also mentioned in the article, so what Pro Drill will offer you is a totally professional service, a no excuses policy and 100% commitment.

Russell Sherwin
Managing Director
Pro-Drill LTD

Conference centre at sunrise.

20th NZGS Symposium, 23-26 November 2017 – What in Earth is Going on?

WALKING INTO THE conference room I understood what the organising committee meant last year when they said that as soon as they saw the venue they knew it was perfect. The large, bright room with windows, set up with chairs around tables foretold that this symposium was about dialogue and the exchange of ideas. The single session format with reporters and digital question-and-answer using sli.do ensured that the conference achieved these goals. It took a lot of courage to try a different style and the organisers must be recognised for challenging the conventional conference format. I really enjoyed how the new format generated conversations amongst delegates about how we run our symposium. I look forward to seeing continued development of our symposium style over the next few years.

Feedback about the MBIE/NZGS Module day on the 23rd has been positive so far with attendees and module authors both saying that it was very useful. The symposium was kicked off by Ruth Allington, who set the tone by talking about professionalism in the mineral resources sector with many learnings for geotechnical professionals – communication, mediation, listening and the permeable boundaries between our areas of expertise. Dave Petley delivered an elegant presentation of pre-failure processes in creeping slopes using numerous case studies and a video that required an encore. I particularly enjoyed how he demonstrated the link between observation of the ground, lab experimentation and remote sensing to point us towards where the state of the art is going. John Atkinson encouraged us to carefully consider which failure criterion we use in soil mechanics and the risk involved in using oversimplified models. Mick Pender closed the symposium with a summary of his key observations over his career.

At the gala dinner the NZGS began issuing newly designed trophies for presenters of the NZ Geomechanics Lecture, starting with Michael Pender (1996), Warrick Prebble (2001) and Laurie Wesley (2004). We also presented life memberships to Mike Stannard and CY Chin for their significant contributions to the society. On the other end of the professional spectrum, it was great to see the large number of young professionals who attended. I must applaud EQC for sponsoring students and the companies who promote the development of their young staff by supporting their attendance. The symposium was followed up by a field trip to look at the uniquely young geology of Hawkes Bay. Tom Bunny did a great job organising the field trip, led by Kyle Bland from GNS. We also had a cameo by Warrick Prebble who is clearly able to interpret geomorphology even in places he has never been!

In this special feature we provide copies of Dave Petley and John Atkinson's keynote papers, followed by the session chairs' choices for best paper and best YGP paper. Congratulations to the best paper authors and commendations to the organising committee for putting together a fantastic and fun symposium.

Reported by Marlène Villeneuve

Top left and clockwise: Mike Stannard accepting a life membership; Mike Stannard, Charlie Price and Tony Fairclough working through an MBIE/NZGS module discussion; Some of the EQC funded students: Xiaoyu Chen, Heba Elsaidy, Mark Gray, Thomas Robertson, Romy Ridl, Francesca Spinardi, Jesse Merkle, Katherine Yates, Baqer Asadi, Sayed Hessam Sam, Yu Wang, Romain Meite; Kyle Bland describing the geological history of the Hawkes Bay region on the field trip; Delegates at the gala dinner; Delegates at the symposium.

Understanding patterns of movement of slow landslides

KEYNOTE

ABSTRACT

The movement of many landslides is controlled by the force imbalance associated with a reduction in shear resistance caused by a decrease in normal effective stress as pore water pressures increase. This basic premise might lead to an assumption that the movement rate has a simple relationship with the pore water pressure / normal stress state, but previously studies have shown marked differences in this relationship according to whether pore water pressures are rising or falling. This paper reviews examples from the literature in which high resolution monitoring allows the relationship between the movement rate and the pore water pressure / normal effective stress state to be determined. We show that a variety of relationships exist between these parameters with the key determinant appearing to be the peak movement rate of the landslide during the movement event in question. We propose that the key factor is whether the yield stress is exceeded. If so, rate and state friction may dominate; if not then creep decay may be critical.

Keywords: *landslide, movement, deformation, pore water pressure, strain rate, creep*

1 INTRODUCTION

Landslides are a pervasive hazard on the surface of the earth, responsible for an average of up to 14,000 fatalities per annum (Petley 2012). Triggered primarily by one or more of the effects of precipitation, seismic shaking or slope alteration by humans, landslides also induce substantial socio-economic impacts on society. In many cases these effects are magnified by a lack of insurance cover, resulting from both their socio-economic setting (the majority of loss-inducing landslides occur in comparatively poor countries across Asia and Latin America) and by an unwillingness by insurance companies to provide cover for mass movement hazards in most territories. The latter results from a perceived poor understanding of the geographic distribution of landslide hazard, and the difficulties of determining potential levels of consequential loss. The effects are to increase the impact of landslide hazards relative to other natural hazards.

Whilst the majority of human casualties are associated high velocity landslides, and in particular debris flows, mudflows and soil/rock avalanches, slow moving landslides can cause high levels of financial loss, and, in some cases, loss of life. Thus, understanding these landslides remains a priority. The simple mechanics of these landslides is well-understood in terms of the role of elevated pore fluid pressure leading to a reduction in normal effective stress, and thus failure, and the development of strain, when the yield strength is exceeded. However, observed patterns of movement are more complex than this simple relationship would imply, and are important in terms of understanding, and forecasting, future behaviour for any slow moving landslide. In this paper, the relationship between pore fluid pressure and the rate of movement of landslides is reviewed, demonstrating complex patterns that have hitherto not been fully understood. Interestingly, glaciers

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display similar behaviour, and a number of hypotheses have been proposed to explain these mechanics across the two types of mass movement. The viability of these models for landslides is examined, and a new framework is proposed to account for the observed complex behaviour in landslide systems.

2 PATTERNS OF MOVEMENT OF SLOW MOVING LANDSLIDES

2.1 A review of landslide movement patterns

It is well established that the movement rate in a slope has a non-linear relationship with pore water pressure (e.g. Bertini, et al, 1984; Gonzalez et al. 2008). In general, once movement has commenced small increments of additional pore water pressure lead to successively greater increases in movement rate; the relationship between pore water pressure and movement rate is sometimes characterised as being exponential. This has sometimes been characterised with a viscosity modification to the Mohr-Columb failure criterion, with some success in predicting the moment patterns of flow type landslides. These models predict a movement rate for any given value of pore water pressure in the landslide, regardless of the dynamic state of that pore water pressure. Of course in reality, factors such as the geometry of the landslide play a key role. Thus, for example, in a rotational landslide the mass becomes increasingly stable as strain accumulates, such that the relationship between strain rate and pore water pressure will change as movement develops (primarily because the static stress state will change). However, in a large landslide this changing relationship will require large strains to become significant.

Some landslides show a simple relationship between pore water pressure and rate of movement in monitoring data. Thus, for example, monitoring of the Vallcebre landslide in the Eastern Pyrenees of Spain showed a simple, non-linear relationship between velocity and the depth of ground water (i.e. the shear surface pore water pressure) (Corominas et al. 1999; Fig. 1). In such cases the movement rate of the landslide can be predicted for any groundwater level.

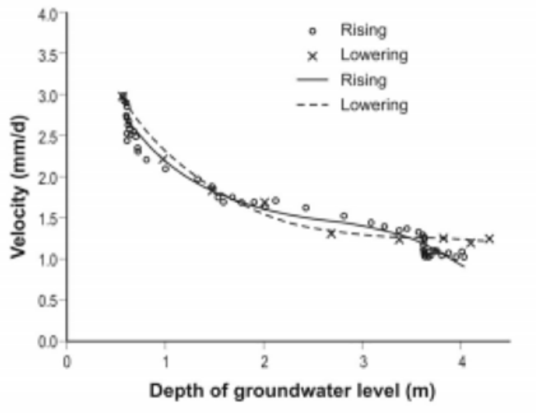

Figure 1: Patterns of movement of the Vellcebre landslide. Data from Corominas et al. (1999). Figure from Massey (2010)

Interestingly, however, there are a number of documented cases in which this relationship has proven to be more complex than might be expected, even when geometric factors have been taken into consideration. In particular, many landslides show a different movement response when pore water pressure is increasing in comparison to when pore water pressure is falling. But, surprisingly, there is no consistent relationship. The following sections provide some examples. Unfortunately though, there is a surprising paucity of published examples in which monitoring data is of sufficient quality to allow this relationship to be examined in detail.

2.2 La Valette landslide, France

La Valette landslide is located close to Saint-Pons in the Barcelonnette basin, in the Alpes-de- Haute-Provence region of France. Movement began in March 1982 as a reactivation of a pre-existing landslide (Van Asch et al. 2007). The landslide consists of an upper rotational slide that transitions into a mudflow as the displaced blocks degrade. It is large – the estimated volume is about 3.5 x 106 m3, the length is about 2 km and the shear surface depth is 25 to 35 m in the central part of the mudflow. The landslide moves at variable rates, with a total displacement rate of about 1 to 2 m per annum.

La Valette landslide is extensively monitored due to the threat that it poses to the community at the foot of the slope. Van Asch et al. (2007) presented monitoring data for the landslide during a phase of increased pore water (Fig. 2). As expected they found a non-linear relationship (hysteresis) between movement rate and groundwater level, but perhaps less predictably they also found that the movement rate when the ground water was increasing was substantially different from that when groundwater level was declining. In this case a rising groundwater level was associated with a higher movement rate than was the case with a falling groundwater level. This was found to be consistent across two substantial periods of movement. This behaviour appears to be a complex version of strain hardening, in which resistance to movement increases with deformation. In this paper we refer to this style of relationship between movement rate and pore water pressure as strain hardening behaviour. Note however that this is a more complex style of behaviour than is normally ascribed to strain hardening as the increased resistance appears to develop at the point at which pore water pressures start to fall, but not before.

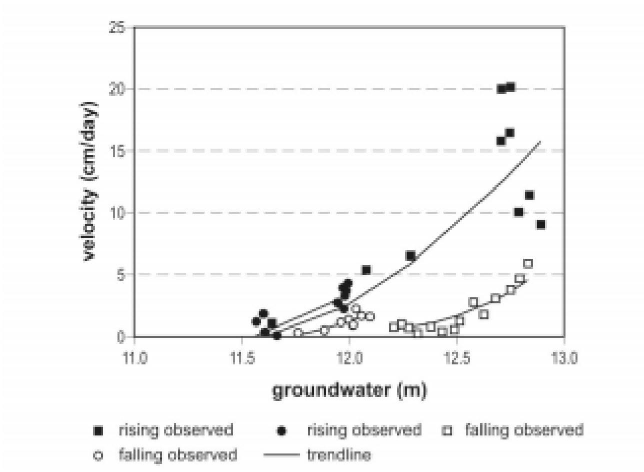

Figure 2: The relationship between velocity and groundwater level (depth above the shear surface) for La Vallette landslide. Data from Van Asch et al. (2007), figure from Massey (2010)

Similar behaviour has been seen in other landslides. Thus for example, Bertini et al. (1986) saw strain hardening behaviour at the Fasio San Martino landslide in Central Italy.

2.3 A reactivated landslide in central Japan

Matsuura et al. (2008) monitored both pore water pressures and landslide displacement in an unnamed reactivated landslide in weathered mudstone and silty sandstone in central Japan. This landslide was about 400 m long and 50 to 70 m wide. Movement occurred in response to increased pore water pressures driven by both precipitation and snowmelt. The basal shear surface, which was at a depth of 4 to 7 m, lay in highly weathered tuff. This was a comparatively fast moving landslide – rates of up to 50 mm per day were recorded – and between 9th September and 3rd December 1992 the landslide moved a total of about 1290 mm. During phases of increased rates of displacement the landslide displayed a strong pattern of hysteresis in the relationship with pore water pressure, but in the opposite sense to that displayed by La Valette (Fig 3). In this case displacement rates were comparatively slow as pore water pressure increased, and more rapid as it decreased. Particularly interesting is the observation that the landslide continued to accelerate even as pore water pressure started to fall. We term this a strain weakening behaviour, in which the landslide shows increased susceptibility to movement as strain develops, although once again the pattern of behaviour may be more complex than simple strain weakening would imply. In the case of the unnamed C. Japan landslide, this strain weakening behaviour was displayed in several movement periods. Matsuura et al. (2008) were not able to explain this behaviour, but suggested that it might be controlled by the geometry of the landslide:

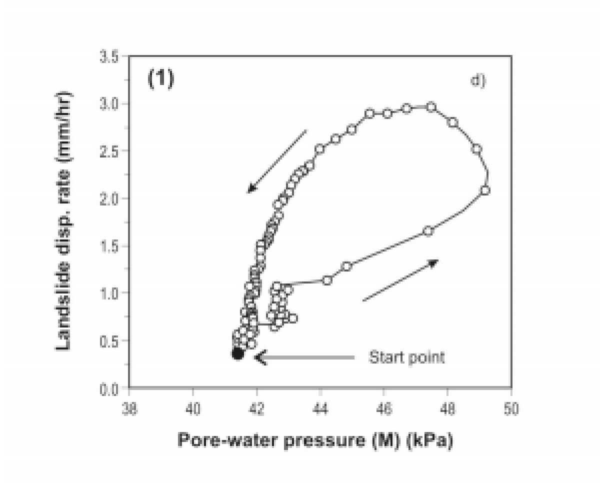

Figure 3: The relationship between the landslide displacement rate and the pore water pressure for the C. Japan landslide. Data from Matsuura et al. (2008),

"One reason for this may be that some aspects of landslide kinematics are controlled by the inclination of the sliding surface and the interaction between the moving body and side in the surrounding stable ground."

It is unclear as to how this mechanism would operate in a landslide of this type.

2.4 The behaviour of the Kualiangzi landslide in China

The Kualiangzi landslide on the margin of the Sichuan Basin of central China is a deep-seated translational bedrock landslide in interbedded mudstones and sandstones (Xu et al. 2016). This landslide is very large, with a width of about 1,100 metres and a length of up to 390 m, with an estimated volume of about 25.5 million m3. Movement occurs on an inclined shear surface in weathered mudstone located an average of 50 m below the surface. At the rear of the landslide there is an exceptionally large tension trough.

Xu et al. (2016) monitored movement on the landslide through 2013, finding that the displacement rate of the landslide increased in response to precipitation. Analysis of the movement of the landslide record suggests that it broadly shows the strain hardening type of behaviour (Figure 4). Interestingly, the two movement events represent different failure regimes – in the case of the first movement event, the calculated factor of safety of the landslide did not drop below unity. In the second movement event this was the case (Xu et al. 2016 suggested that it reached about Fs=0.93). Nonetheless in both cases the landslide showed the strain hardening style of behaviour.

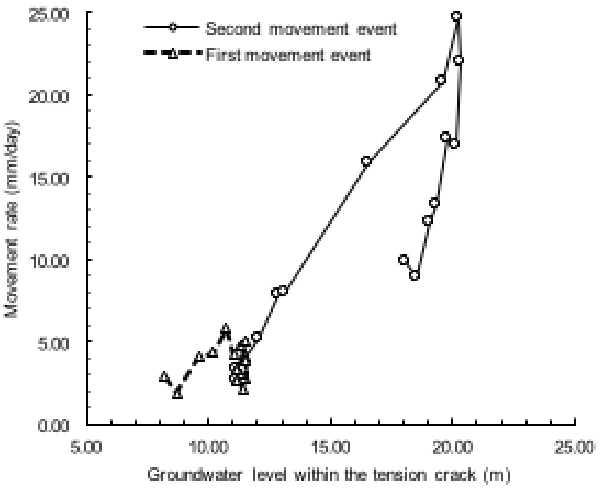

Figure 4: The relationship between the movement rate and the groundwater level in the Kualiangzi landslide, after Xu et al. (2016)

2.5 The behaviour of the Ventnor landslide in southern England

Amongst many other studies (e.g. Hutchinson and Bromhead 2002), Carey (2011) investigated the behaviour of the large (700 hectare), slow moving landslide complex upon which the town of Ventnor is built in the Isle of Wight in southern Britain. This is a complex landslide, involving differential block movement and the opening of grabens, on a very deep (>100 metre) shear surface (Hutchinson and Bromhead 2002). Movement rates are very low however, with peak velocities below 1 mm / day. This very large landslide has been extensively monitored, in particular in the area of a large graben structure that is developing at the crown of the landslide. At this location, both pore water pressure and displacement have been measured at various times.

A notable movement event occurred in the winter of the year 2000 in response to a prolonged and unusually wet period of weather. In this case, the strain weakening style of behaviour was clearly observed (Fig. 5), with movement rates being considerably higher on the falling limb than when pore water pressures were increasing.

2.6 The response of glaciers

Interestingly, it has long been observed that some glaciers also show hysteresis in movement in response to stress changes (Iken et al. 1983 for example). Shallow glaciers are in many ways landslides of ice, with sliding occurring either on a bedrock – ice interface or through deformation of a layer of till between the ice and the bedrock. An advantage of glacers is the relative ease with which the basal processes can be investigated (certainly in comparison with landslides), as it is sometimes possible to access the basal region. Considerable work has been undertaken to understand their dynamics.

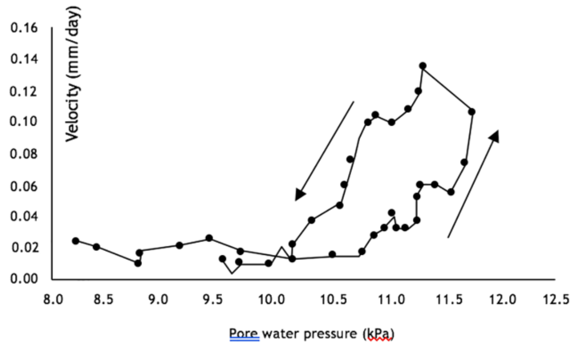

Figure 5: The relationship between landslide velocity and pore water pressure for the Ventnor landslide, after Carey (2011)

Basal tills show creep like behaviour below the yield strength, and a combination of creep and stick-slip behaviour (representing periods in which the effective normal stress reduces sufficiently to ensure that the macroscopic yield strength is lower than the externally imposed shear stress) results in a stepped movement pattern, in common with landslides. A key finding in till dominated systems is that creep rates often decay rapidly under constant stress forcing, once the initial perturbation that caused the movement to start has passed. Thus, till deformation provides a potential mechanism to explain this behaviour; creep rates have been observed to increase so long as normal effective stress is increasing, but decline under constant stress conditions. In a real system this would generate the strain hardening style of behaviour.

Glaciers examined by Damsgaard et al. (2016) shows this behaviour, with slow increases in velocity during phases of increasing pore water pressure, but rapid reductions in movement rate when pore water pressures peak. The authors point out that behaviour that is similar to that of glaciers is seen in landslides, and hence argue that the styles of deformation are directly analogous.

3 INSIGHTS INTO LANDSLIDE RESPONSE TO PORE WATER PRESSURE CHANGE FROM LABORATORY TESTING

Most laboratory testing of landslide materials does not provide insight into the response of materials to changes in normal effective stress. The vast majority of geotechnical tests use a non-representative stress path, in which deformation is driven by changing shear stress under conditions of constant strain rate. In drained tests, normal effective stress is not permitted to change; in undrained tests pore water pressure can be generated, but only as a response to the application of shear stress. To investigate the behaviour described above requires the use of the field stress path, sometimes termed the pore pressure reinflation (PPR) test (see Petley *et al.* 2005), in which normal and total effective stress are kept constant, and pore water pressure is varied. The PPR test is most commonly undertaken within triaxial or stress path equipment, which makes it simple to capture the pore water pressure increase phase, but challenging to deal with the subsequent reduction in pore water pressure conditions. Nonetheless this equipment can provide considerable insight.

Ng (2007) undertook a large suite of PPR tests on undisturbed residual soil samples from Lantau Island in Hong Kong. Some of these experiments involved a step-wise increase in pore water pressure (and thus a reduction in normal effective stress) with axial stress and confining pressure held constant, initially at less than the yield stress, but ultimately exceeding it. Fig. 6 shows the development of strain rate in two tests, for one of which pore water pressure was increased in steps of 10 kPa, once per hour, whilst in the other pore water pressure was increased constantly (ramped) at 10 kPa per hour. In the ramped test the strain rate increased exponentially with increasing pore water pressure (i.e. decreasing effective stress state). However, in the stepped test the strain rate initially increased at a rate substantially higher than that of the other test, but which subsequently declined. This initially high and then declining creep rate is a manifestation of the creep decay mechanism. This behaviour, which was seen consistently in the tests of Ng (2007), is similar to that observed in the till deformation of glaciers.

The initiation of final failure in these tests was also interesting. After the final pore water pressure step the stepped test sample (TC28) showed an initial increase in strain rate (Fig. 7), which then declined before increasing again, with the sample then proceeding to full failure. This appears to be evidence that the final failure event is

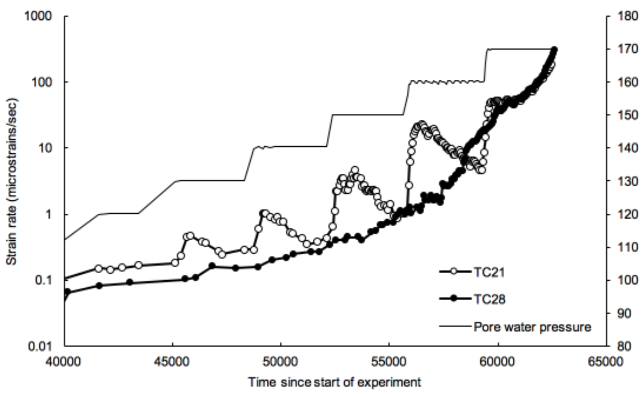

Figure 6: PPR experiment results for residual soil samples from Lantau, Hong Kong, after Ng (2008). Sample TC21 was subjected to increases in pore water pressure in 10 kPa steps (illustrated on the graph), whilst for sample TC28 porewater pressure was increased at the same average rate, but at a constant rate

associated with damage accumulation, as postulated by Petley et al. (2002). Interestingly, Carey and Petley (2014) saw similar behaviour, with final failure being observed in a long term creep test in which no change in effective stress state occurred.

3 DISCUSSION

From the examples described above, and from others in the literature (Table 1), is clear from a range of landslides that different patterns of movement can be seen in response to changes in pore water pressure. In all cases the relationship between movement rate and pore water pressure is exponential once FoS·1, but for some landslides the relationship may be considered to be strain hardening, in others strain weakening, whilst in a small number the behaviour may be strain neutral, or the response to pore water changes is weak (e.g. Taihape and Utiku). In none of the studies outlined above was a clear explanation given for the response observed. In all cases in which there were multiple movement events the landslide showed consistent behaviour. Thus, in each case it appears to be a fundamental property of the landslide in question.

The strain hardening style of behaviour has also been observed in glaciers moving through deformation of a basal till. Damsgaard et al. (2016) noted the similarity in behaviour seen between glaciers and some landslides. In the case of the glaciers that they modelled, creep was a distributed mechanism whilst slip involved some degree of strain localisation. Whilst a creep decay mechanism was observed, this applied below the critical shear stress. Interestingly, Damsgaard et al. (2016) also note that above the yield strength the rheology of the systems becomes rate independent. This view does not seem to be supported by the monitoring data for the landslides.

Name	Landslide type	Basal material	Peak movement rate (mm/day)	Pattern	Reference
La Valette	Translational mudflow	Weathered soil	200	SH	Van Asch et al. (2007)
Purbeck	Mudslide	Weathered clay	60	SW	Allison and Brunsden (1990)
C. Japan	Translational rockslide	Mudstone /silt	50	SW	Matsuura et al. (2008)
Kualiangxi	Translational rockslide	Clay	25	SW	Xu et al. (2016)
Slumgullion	Earthflow	Clay-rich debris	10	Neutral/SH	Schulz et al. (2009)
Utiku	Translational rockslide	Weathered clay	3.0	SH (? Role of pore water)	Massey et al. (2013)
Vallcebre	Translational rockslide	Colluvium	1.5	Neutral	Corominas et al. (1999)
Fasio San Martino	Earthflow	Clay-rich siltstone	0.4	SH	Bertini et al. (1986)
Taihape	Translational rockslide	Weathered clay	0.1	SH (? Role of pore water)	Massey et al. (2016)
Ventnor	Rotational rockslide	Weathered clay	0.1	Neutral / SW	Carey (2011)

Table 1: A summary of the landslide movement records analysed in this paper

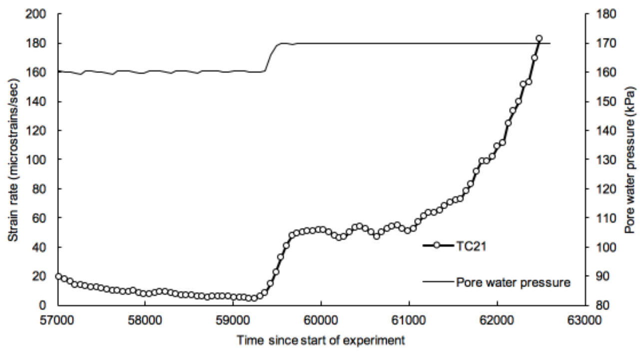

Figure 7: Detail of the last phase of the experiment on sample TC21, after Ng (2008).

Van Asch et al. (2017) described the strain weakening style of behaviour as a lower intrinsic viscosity and lower dependency on excess pore water pressure when the pore water pressure is rising. They noted that intrinsic viscosity is not necessarily a constant value, but recognised that a decrease in viscosity after pore water pressures have peaked is hard to explain. They propose that this may be an effect of the size of the landslide body, with different parts of the mass experiencing extension and compression at different times. This may explain a complex response to fluctuations in pore water pressure at any point in the landslide.

The various landslides for which there is adequate monitoring data to determine hysteresis behaviour in response to changes in pore water pressure are listed in Table 1 in order of peak movement rate. No obvious relationship between the pattern of movement described in the paper and landslide type or material can be seen, but there does appear to be a general relationship in terms of movement rate. On the whole (but not in every case), landslides with high movement rates show a strain weakening pattern (typically >20 mm / day), whilst those with low movement rates (typically <10 mm/day) show the strain hardening pattern. The exception to the latter is the Ventnor landslide, but note that this movement record is from the graben structure at the crown of the landslide, where the stress regime and deformation pattern may be highly complex.

This behaviour may be explainable by considering the deformation state. We hypothesise that slow rates of landslide deformation in these systems are typically occurring within the creep domain, where the dominant mechanism is plastic deformation through interparticle movement. In this domain, the creep decay mechanism suggests that under constant stress states the creep rate will decline; this behaviour is also seen in the experiments of Ng (1999) and in the behaviour of glaciers. Given this creep decay mechanism, it is inevitable that a reduction in pore water pressure will induce a dramatic reduction in the rate of movement.

On the other hand, the more rapid movement rates, and those associated with faster glacial movement, are associated with the strain weakening mechanism. In this case, Dansgaard et al. (2016) note that in glaciers and landslides this may well be associated with a localisation process that occurs once the yield stress has been exceeded. We propose that the key mechanism here is a rate dependent friction law that allows resistance to movement to change once movement has been initiated. Such a rate-dependent friction process, in which friction reduces with increasing movement rate, as outlined in Handwerger et al. (2016) would generate a strain weakening pattern of movement. On the other hand, a state and rate dependent friction law could also allow the development of strain hardening behaviour under the right circumstances.

As a consequence, the different styles of behaviour seen in landslides in response to changes in pore water pressure, and thus to the normal effective stress state, is probably related to two different processes. Below the yield stress, creep mechanisms dominate. In this case, the creep decay mechanism is critical, in particular when pore water pressures start to fall. In these circumstances, creep rate rapidly declines, causing the strain hardening style of behaviour to be displayed. On the other hand, above the yield stress rate and state dependent friction dominates, allowing both strain weakening and strain hardening styles of behaviour to be shown. In both cases, behaviour may be modified by local conditions associated with the geometry of the landslide, such as curvature of the shear surface and stress transfer between blocks.

CONCLUSIONS

Through a review of the literature we have shown that the relationship between pore water pressure / normal effective stress and the rate of movement of landslides is complex. Beyond the yield stress the movement rate generally has an exponential relationship with normal effective stress. However, it is also clear that the rate of movement may be different between increasing and decreasing pore water pressure states. We have demonstrated that during phases of slow movement landslides tend to show a strain hardening style of behaviour, whilst rapidly moving slide tend to show the strain weakening style. We suggest that this may be associated with the presence of two different mechanisms – below the yield stress creep mechanisms dominate, and thus the effects of creep decay mean that strain hardening is likely. On the other hand, above the yield stress, rate and state dependent friction becomes important, meaning that both the strain weakening and strain hardening styles of behaviour can be shown, depending on the frictional properties of the basal material.

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Non-linear soil mechanics

KEYNOTE

ABSTRACT

Routine effective stress analyses for failure of geotechnical structures use a linear Mohr-Coulomb envelope and routine analyses for ground movements use linear elasticity with constant Young's Modulus E' or one-dimensional modulus M'. Observations of soil behaviour show that strength and stiffness are non-linear and so the conventional simplifications do not match these basic observations. Measurements from several different soils show that their strengths can be represented by a simple power law similar to the familiar Hoek-Brown model for rock strength. For soft soil, stiffness for analyses of settlements of embankments may be approximated as linear. For stiff soil which is highly non-linear, a value of E' for analyses of settlements of foundations may be approximated to $E'_0/3$ where E'_0 is the stiffness at very small strain.

Keywords: *strength, stiffness, power law, laboratory testing, settlement*

1 SOIL STRENGTH

1.1 Conventional Linear Mohr-Coulomb Soil Strength Envelope

A linear relationship between the limiting shear *force* and normal *force* is attributed to Coulomb (1773) as discussed by Heyman (1972). Following the development of analysis of stress by Mohr the original Coulomb relationship was recast in stresses and this is the familiar Mohr-Coulomb strength criterion. In the original Mohr-Coulomb strength criterion stresses were total stresses and the relationship was mostly applied to unsaturated compacted fills. Terzaghi (1936) extended the original Mohr-Coulomb criterion to effective stresses and this is now the conventional criterion for soil strength.

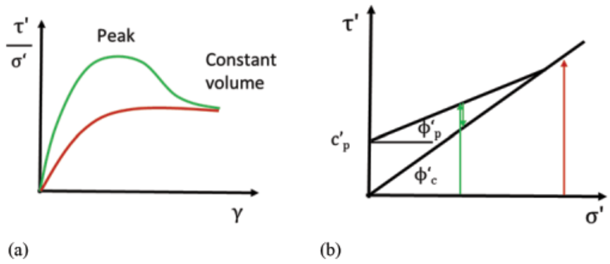


Figure 1: Mohr-Coulomb linear soil strength envelopes

In Figure 1(a) a typical stress –strain curve for soil has a peak strength at a strain of the order of 1% and a smaller constant volume or critical state strength at a strain of the order of 10%. (Clay soils may also have a lower residual strength after very large deformations on a slip plane.)

John Atkinson

John Atkinson has a Bachelor's, Master's and PhD in Civil Engineering from Imperial College. He has worked with contractors and consultants in UK and Australia on a variety of projects including design of earth-fill dams, design and construction of sewerage works, construction of precast concrete bridges, ground investigations and design of foundations, slopes and walls in soils and rocks, tunnelling and shaft sinking, loadings on large buried pipes, movement of granular cargo in bulk carrier ships and determination of soil and rock parameters for design. He is author of three textbooks on soil mechanics and foundation engineering. From 1976 to 1980 he was first lecturer and then senior lecturer in Civil and Structural Engineering at University College, Cardiff. In 1980 he was appointed Reader in Soil Mechanics at City University, London and he was promoted to a Chair of Soil Mechanics in 1985. He is and expert in investigation of soil behaviour in laboratory tests and in centrifuge model testing. After his retirement he has been appointed Emeritus Professor at City University, and continues his involvement with practice as Senior Principal of Coffey Geotechnics. He has lectured widely in UK and overseas on a variety of topics. He was the Rankine Lecturer in 2000, and also the Prague Geotechnical Lecturer in 2005.

It is well established that all these strengths increase with effective normal stress and they are conventionally described by linear relationships as shown in Figure 1(b) so the peak strength is given by

$$\tau'_c = c'_p + \sigma' \tan \phi'_p \qquad 1$$

and the constant volume or critical state strength is given by

$$\tau'_c = \sigma' \tan \phi'_c \qquad 2$$

It is also well established that at sufficiently large normal stresses soils do not have a peak strength only a constant volume strength so the peak and constant volume failure envelopes must meet as shown in Figure 1(b). The stress paths in Figure 1(b) correspond to the stress ratio strain curves Figure 1(a).

In conventional geotechnical engineering practice the linear Mohr-Coulomb strength envelope is used for routine analyses of ultimate limit states with a factor of safety of about 1.3 and it is used for routine assessment of serviceability limit states with a load factor of about 3.

1.2 Peak strength of soil

Figurez 2 shows a set of Mohr circles for the stresses at the peak state and the constant volume, critical state of samples of London Clay (Atkinson and Crabb 1997). The constant volume envelope shown by double lines is linear with parameters c' = 0 and $\phi'_c = 22^o$. The circles for the peak states have a curved envelope which passes through or very close to the origin. It should be noted that the stresses in the samples with the lowest stresses were only a few kiloPascals requiring very careful experimental techniques.

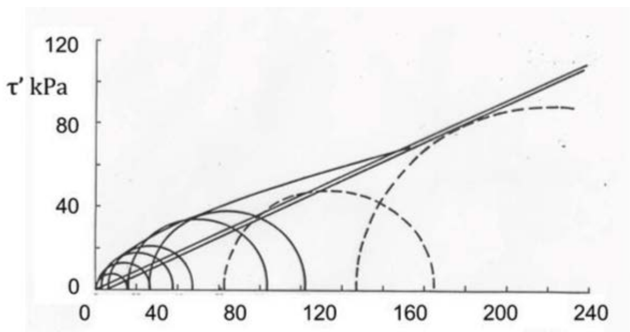


Figure 2: Failure envelope for London clay

Figure 3(a) illustrates the features of the peak envelope from Figure 2. The effective stress strength is negligible when the effective normal stress is zero at the point O. It is impossible to make a stable vertical slope in dry sand. In plastic clays the strength at zero effective normal stress may not be exactly zero but careful experiments show that it is very small and only a few kiloPascals (Atkinson 2007). At relative large effective normal stresses beyond the point Y soils have no peak strength only a constant volume strength. Between O and Y there is a peak strength at P which is larger than the constant volume linear strength envelope. It is impossible for a linear peak strength envelope to join the points O – P - Y.

It is worth recalling that Coulomb developed the linear relationship for unsaturated compacted fill in military earthworks (Heyman 1972). For such soils, and in total stresses, there is a substantial strength at zero total stress. Although the relationship between strength and total normal stress may not be exactly linear the original Mohr-Coulomb total stress relationship is not obviously incorrect for unsaturated soils. It was when the original total stress linear criterion was converted to effective stress by Terzaghi (1936) that it no longer correctly represented observed behaviour of saturated soil.

1.3 Non-linear soil strength

A convenient formulation for the non-linear peak envelope in Figure 3(a) is a simple power law such as

$$\tau'_p = A\sigma'^b \qquad 3$$

where A and b are parameters that depend on the soil grains and on the critical stress σ'_y. This simple power law is similar to the basic formulation of the Hoek-Brown (1980) criterion commonly used to describe the strength of rock. A similar power law was found by Atkinson (2007) for peak strengths of several overconsolidated clays.

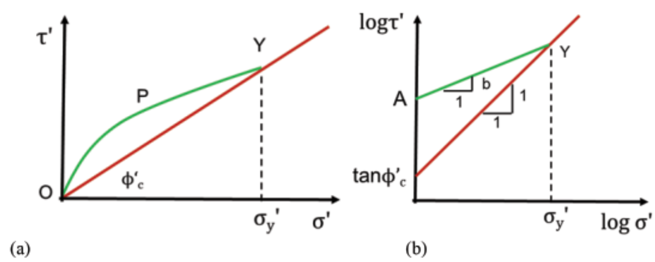
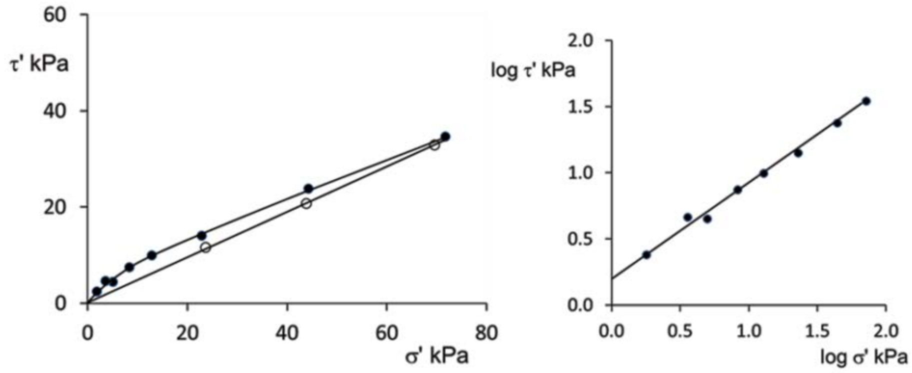


Figure 3: Non-linear soil strength envelopes

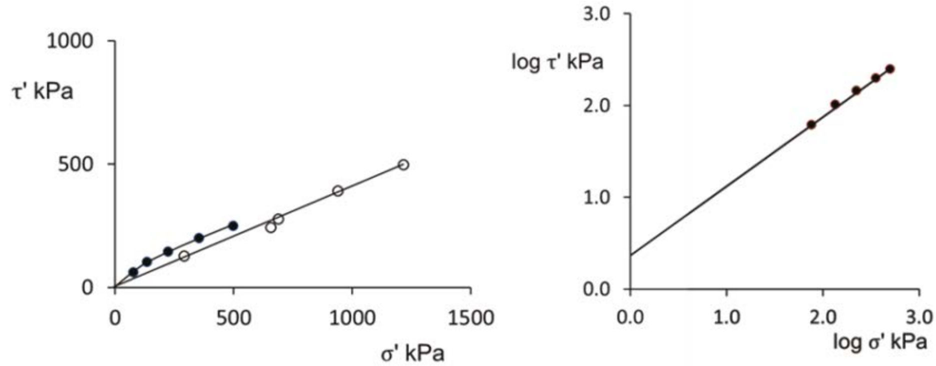
When the stresses are converted to logarithms equation 3 becomes

$$\log\tau'_p = \log A + b\log\sigma' \qquad 4$$

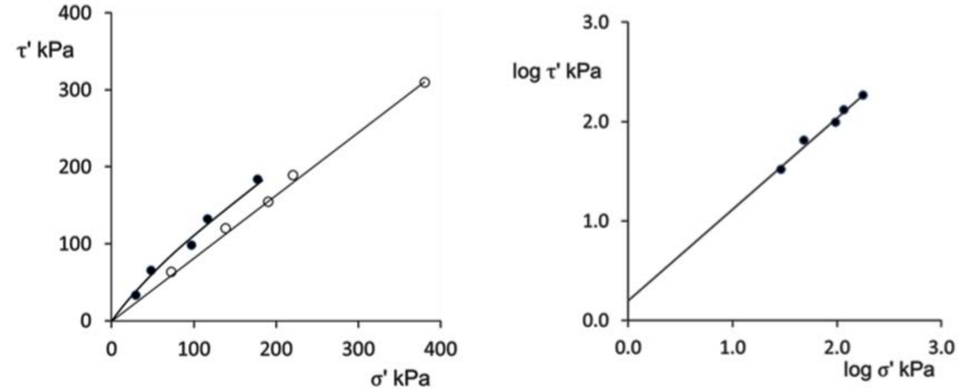
which gives a log-linear envelope as shown in Figure 3(b). Figures 4(a) to (d) show peak and constant volume strengths of some fine and coarse grained soils.



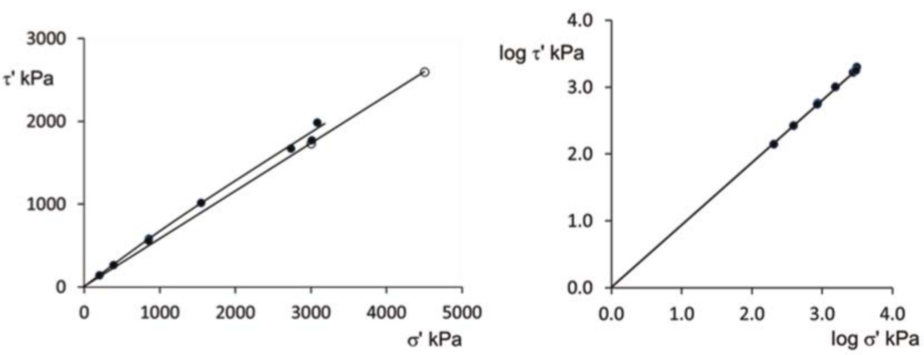
(a) Kaolin clay



(b) Glacial till



(c) Carbonate sand



(d) Quartz sand

Figure 4: Peak and constant volume strengths of soil

In the left hand diagrams in Figure 4 the constant volume critical state strengths are linear with arithmetic scales and conform to equation 2. In the right hand diagrams the peak state strengths are linear with logarithmic scales and conform to equation 4. Values for the parameters A and b in equation 4 and the constant volume friction angle ϕ'_c in equation 2 are summarised in Table 1.

Table 1: Values for parameters in Figures 4

Soil	A	b	$\phi'c$
Kaolin clay	1.6	0.72	25
Glacial till	2.8	0.72	22
Carbonate sand	2.0	0.83	39
Quartz sand	1.0	0.93	30

The data in Figures 4 show that the peak strength envelopes of several coarse and fine grained soils are non-linear. They all pass through or very close to the origin. These envelopes can be described by the simple power law in eqn 5 with the parameters given in Table 1. The large strain, constant volume, strengths of these soils are linear and are described by eqn 2 with zero cohesion and friction angles ϕ'_c given in Table 1.

1.4 Errors using a linear Mohr-Coulomb envelope

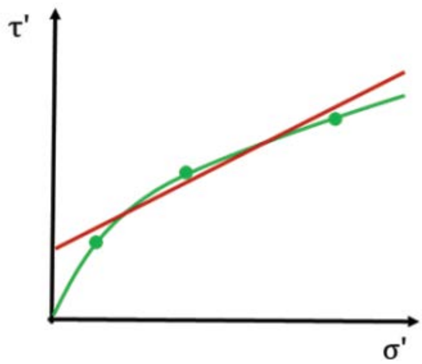


Figure 5: Fitting a linear envelope.

Figure 5 illustrates the errors implicit in the linear Mohr-Coulomb envelope. There are three data points obtained from a set of three triaxial or shear tests that define a non-linear failure curve and a linear Mohr-Coulomb envelope has been fitted through them. There is a mid-range of stress for which the soil strength is greater than that given by the linear Mohr-Coulomb criterion but at larger and smaller stresses the linear envelope is above the true curved envelope and it is unsafe. At small stresses relevant to stability of shallow landslides the errors using the linear Mohr-Coulomb envelope can be considerable (Atkinson, 2007, Atkinson and Crabb 1991).

2 SOIL STIFFNESS

2.1 Linear and non-linear stiffness

Stiffness is the relationship between changes of stress and changes of strain and stiffness modulus is the gradient of a stress- strain curve. These may be shear stress and strain giving a shear modulus G' or mean stress and volumetric strain giving a bulk modulus K' or a one-dimensional modulus M'. Figure 6(a) illustrates a general non-linear stress – strain curve. The stiffness at a point may be described as a tangent modulus $\delta\sigma'/\delta\varepsilon$ or as a secant modulus $\Delta\sigma'/\Delta\varepsilon$

Materials may be elastic or in-elastic and the criterion is what happens during a loading – unloading cycle. Elastic materials are conservative so (by definition) stress-strain curves for loading followed by unloading must be identical so no work is dissipated over the cycle as illustrated in Figure 6(b). Figure 6(c) illustrates the behaviour of a material that is linear but inelastic; the area within the loading – unloading loop is a measure of the work dissipated.

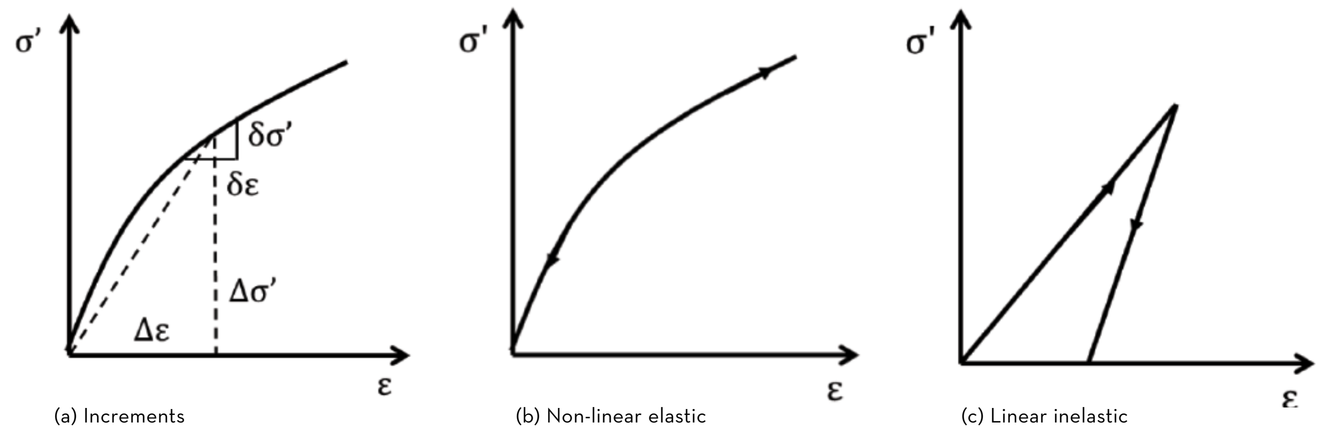
2.2 Conventional linear soil stiffness

Fig 7(a) illustrates a typical stress – strain curve of soil in a drained shear test with an unloading – reloading cycle. The stress – strain curve for a drained triaxial test would be similar. The gradient is the shear modulus G' and for a triaxial test it is Young's modulus E'. Fig 7(b) illustrates a typical stress – strain curve of soil in isotropic compression with an unloading – reloading cycle. The curve for one-dimensional compression would be similar. The gradient of the isotropic loading stress – strain curve is the bulk modulus K' and for one-dimensional loading it is M' = 1/mv where mv is the 1D compressibility.

In conventional practice ground movements are calculated assuming the soil to be linear with a single value for stiffness. There are several ways in which the value of stiffness is determined and the red lines illustrate some of these.

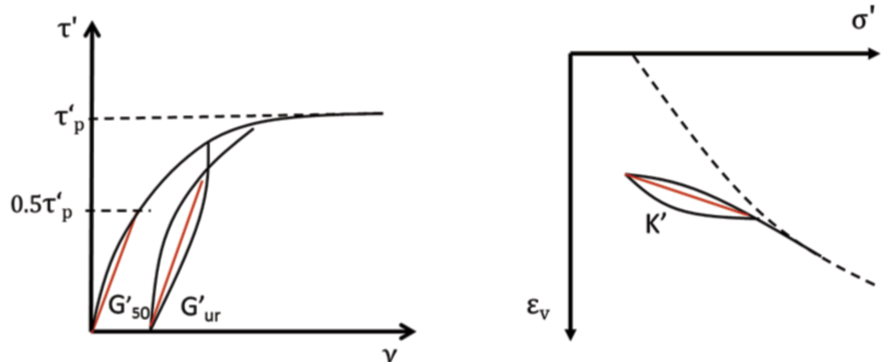
2.3 Non-linear soil stiffness and settlements of foundations

For any of the loading or unloading stress – strain curves shown in Figure 7 the stiffness decays with strain from the start of the loading or unloading stage. The decay of stiffness with strain is well-established (Atkinson 2000) and a typical stiffness decay curve is illustrated in Figure 8. The strains vary over several orders of magnitude from very small strain (<0.001%) to large strain (>1%). Different experimental



(a) Increments (b) Non-linear elastic (c) Linear inelastic

Figure 6: Material stiffness.



(a) (b)

Figure 7: Typical stress – strain curves for soil

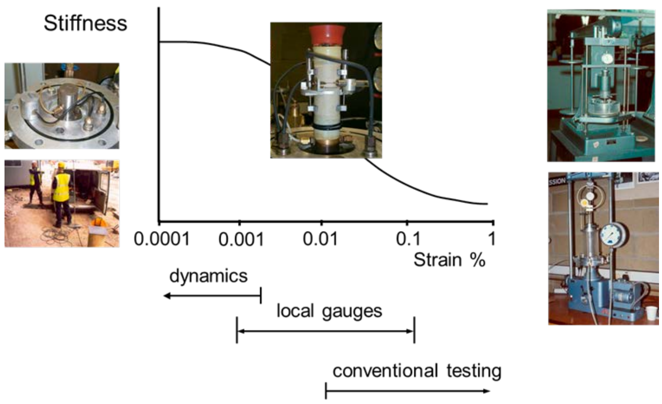


Figure 8: Measurement of soil stiffness

techniques are required to measure soil stiffness over different ranges of strain as illustrated in Figure 8.

In conventional triaxial or oedometer tests there several sources of error in measurement of strain and it is impossible to obtain reliable measurements smaller than about 0.01%. To measure smaller strains it is necessary to use local gauges attached to the sample but these are limited to strains of about 0.001%. Soil stiffness at very small strain less than about 0.001% may be found from measurements of shear wave velocity in the ground or in laboratory samples.

2.4 Calculation of settlements of foundations

Some soils in the ground are heavily overconsolidated and are relatively stiff while other soils are lightly overconsolidated and are relatively soft. Analyses of settlements on soft soil should be treated differently from analyses of settlements on stiff soil. In practice there are two general cases and these are illustrated in Fig 9.

Buildings and embankments apply similar bearing pressures, typically 100 to 200kPa. An embankment can tolerate relatively large settlements of 100mm or more and can be built on soft soil. If the depth of soft soil is 10m the strains are of the order of 1%. Most buildings can tolerate only relatively small settlements of the order of 10mm and for a foundation 10m wide the mean strains are of the order of 0.1%. Shallow foundations can only be built on stiff soil so buildings on soft soil are most often on piled foundations.

The strains resulting from the same loading $\Delta\sigma'$ are illustrated in Fig 9(c) and the corresponding stiffness decay curves are illustrated in Fig 9(d). The initial state of the soft soil is close to the normal consolidation line so the embankment loading moves the state from lightly overconsolidated to normally consolidated. The initial state of the stiff soil is far from the normal consolidation line and the state remains overconsolidated throughout the foundation loading.

Figure 10 illustrates an embankment on a layer of soft soil of limited thickness. The deformations approximate to one-dimensional. The soil state moves from an initial lightly overconsolidated state and ends at a normally consolidated state. Over this range the stress – strain can

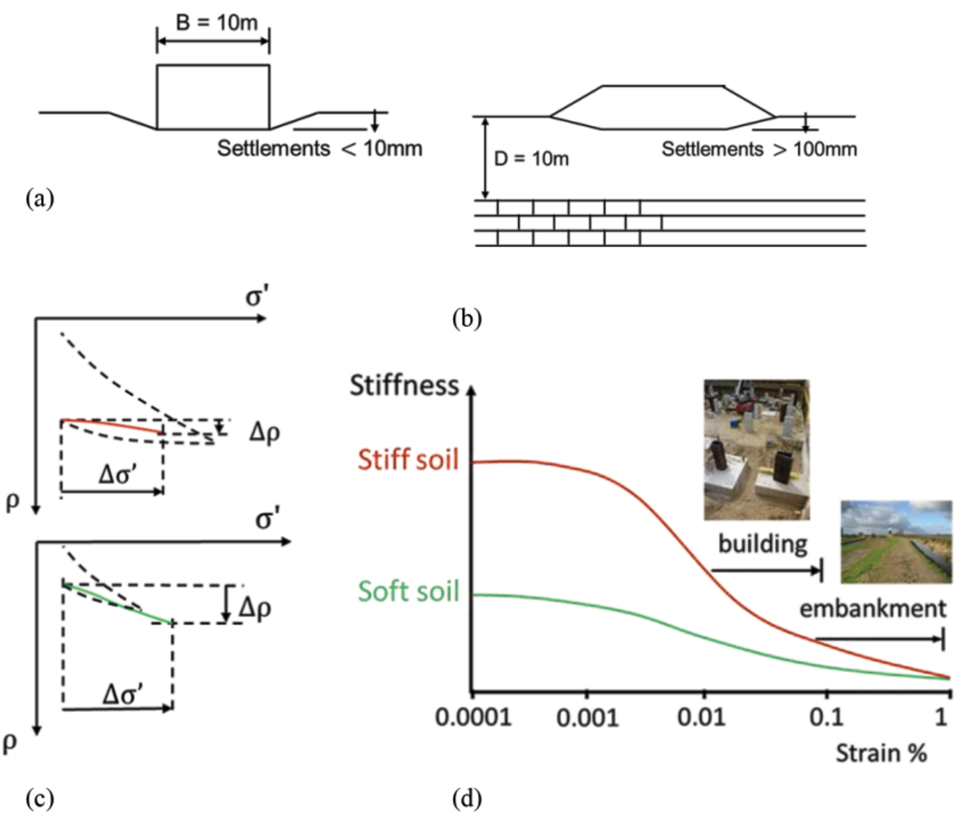


Figure 9: Settlement of structures on stiff and soft soil

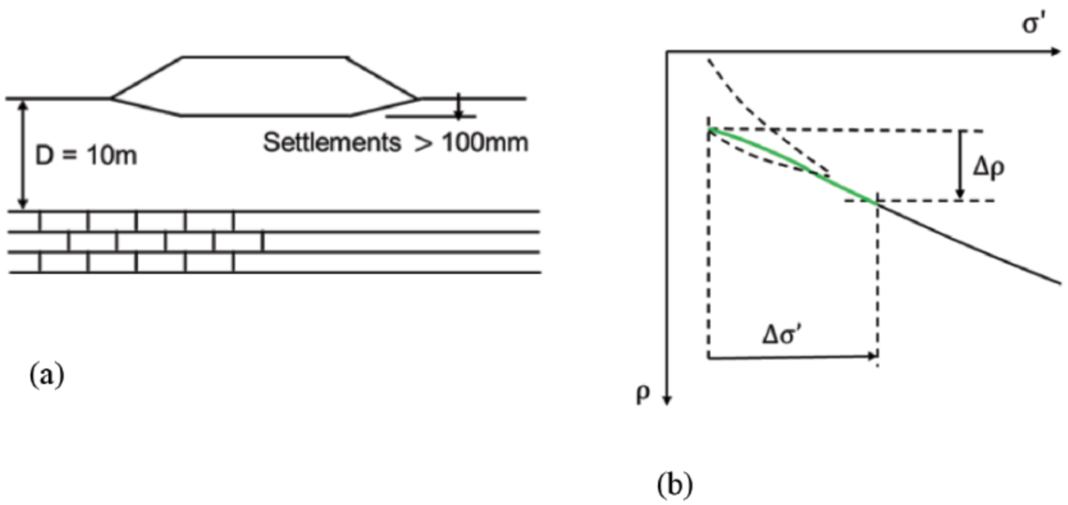


Figure 10: Settlement of an embankment on soft soil

reasonably be approximate as linear with gradient M'

Figure 11(a) shows a foundation on stiff soil and Figure 11(b) shows the load –settlement curve. The state remains inside the normal consolidation line and the soil remains overconsolidated. A typical design would require settlements less than 10mm and for a foundation 10m wide this is a strain of about 0.1%. A typical stiffness – strain decay curve for stiff soil is illustrated in Figure 11(c). For very small strain less than about 0.001% the stiffness is approximately constant. For larger strains the stiffness decays and becomes very small as the soil fails. As a simplification, sufficient for routine foundation design, the

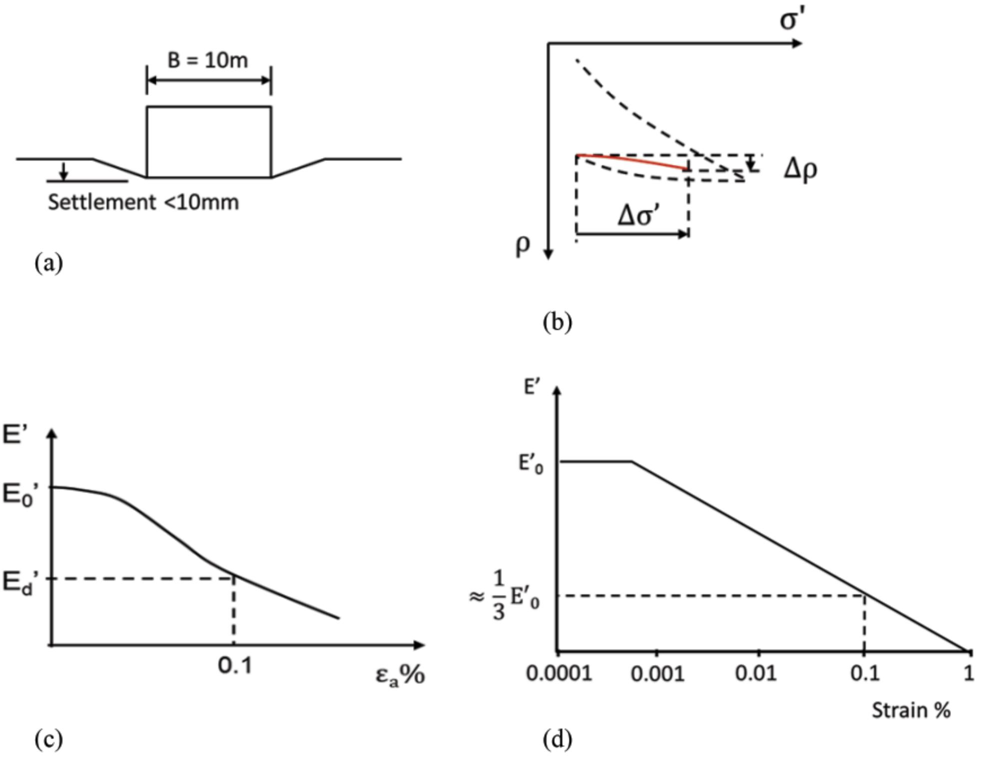


Figure 11: Settlement of a foundation on stiff soil

decay of stiffness with log strain can be taken as linear as shown in Figure 11(d).

Values for E'0 the stiffness at very small strains less than about 0.001% can be found from routine measurements of shear wave velocity either in situ or from tests on laboratory samples. (Atkinson 2000). Stiff soils typically reach a state of failure with very small stiffness at strains approximately 1%. From Figure 11(d) the stiffness for simple design of a foundation on stiff soil can be taken as E'o/3 corresponding to settlements Δρ/B of 0.1%.

In routine ground engineering practice the errors implicit in these simplifications are often no greater than the uncertainties in determining the ground conditions and soil parameters from routine in situ or laboratory tests.

3 SUMMARY

In routine ground engineering practice soil strength is described by a linear Mohr-Coulomb criterion and stress-strain behaviour is linear. Neither of these approximations adequately represent the general features of soil behaviour.

Peak strengths of a variety or soils measured over range of normal stresses can be described by a simple power law given by equation 3 which is similar to the Hoek-Brown failure criterion for rocks.

The stress-strain behaviour of stiff soil is highly non-linear and stiffness decays rapidly with strain. A reasonable approximation is to take a design stiffness as $E'_o/3$ corresponding to a strain of about 0.1%.

Strains below an embankment on soft soil are commonly of the order of 1% as the soil state moves from lightly overconsolidated to normally consolidated. Over this range the stress-strain behaviour approximates to linear with one-dimensional stiffness M'.

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Session Chairs' Choice 1

Dora Avanidou

Dora is a Senior Hydrogeologist at Beca Ltd. She is a professional engineer with a wide experience in water resources management, and environmental risk assessments. Dora holds a Master degree in Civil and Environmental Engineering from AUTH, Greece, a Master's and a Doctoral degree in hydrogeology from USC, USA. She is currently member of the steering committee of IAH - NZ Chapter.

Tools and methods to manage groundwater and settlement effects from the construction of an expressway on peat deposits

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Keywords: peat, groundwater, settlement, monitoring

ABSTRACT

The Mackays to Peka Peka Expressway is part of the Wellington Northern Corridor, identified as a Road of National Significance for New Zealand. The Expressway runs in close proximity to several wetlands of significant ecological and cultural value with unique fauna and flora, and to transport infrastructure and residential buildings built over peat deposits. Approximately 50% of the Expressway earthwork footprint is underlain by peat deposits that are typically 0.5m to 8.0m thick and are characterised as very soft, highly organic and compressible. Groundwater in the peat deposits is shallow and is directly connected to nearby wetlands. Expressway construction required either removal of the peat beneath its footprint or preloading and surcharging of it, to manage long term settlement. Both approaches involve significant potential alteration to the near surface groundwater system.

A comprehensive groundwater monitoring and settlement monitoring programme was established prior to construction commencing, with some 110 piezometers to record natural variations in groundwater levels and 100 settlement monitoring points. In order to differentiate construction effects from normal seasonal variations, a statistical approach was developed for setting both high and low trigger levels for the 22 telemetered piezometers located in and around five sensitive wetlands. Recorded groundwater levels and settlement have generally remained within the consented levels. Some exceedances were recorded however a pragmatic approach to monitoring and management of these exceedances allowed works to continue with minimal disruption and no adverse environmental effects.

1 INTRODUCTION

The MacKays to Peka Peka (M2PP) Expressway Project is one of eight sections that make up the Wellington Northern Corridor, identified as one of the 'Roads of National Significance' (RoNS). The Expressway consists of approximately 18km of four lane median-divided Expressway from the Raumati Straights in the south through to Peka Peka in the north (Figure 1). The Expressway runs in close proximity to several wetlands of significant ecological and cultural value, transport infrastructure and residential buildings built over peat deposits. Groundwater in the peat deposits is shallow and is directly connected to nearby wetlands. The Expressway is predominately on embankments, rising up to 10m high at crossing points. Expressway construction activities required embankment construction with localised preload and surcharge or excavation and replacement of peat below the groundwater table to manage long term settlement (Figure 2).

Additionally construction of stormwater devices for treatment, conveyance and attenuation of run-off, and short-term groundwater take for construction water supply, have the potential to change groundwater level and might result in ground settlement, or changes to water levels in existing wetlands, ecological systems or water supplies.

Figure 1: Project Location and Extents

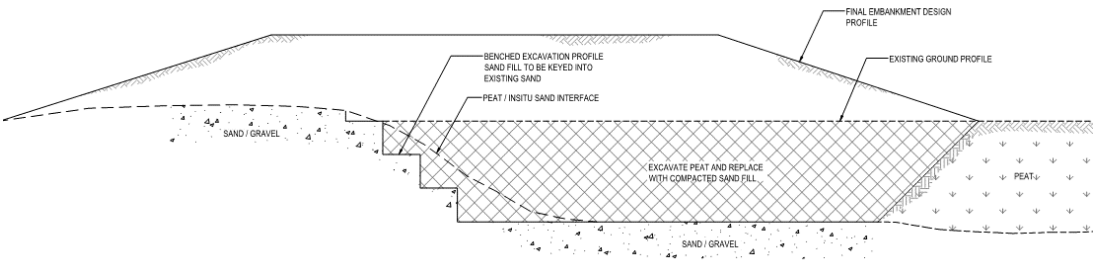

Figure 2: Typical cross section: Peat undercut for embankment on peat and sand

During the project consenting process an assessment of groundwater effects (France and Anderson, 2012) was undertaken, in order to assess potential changes to the existing groundwater regime as a result of the Expressway construction and operation. The effects on groundwater were assessed by the development of regional and area-specific 2- and 3-dimensional computer groundwater models, calibrated to water level monitoring data, and taking into consideration the modelling work carried out in the past. The associated Groundwater Management Plan (GMP)

(Williams, 2013) defined the best practicable options for groundwater monitoring and management practices and procedures to minimize impact on the environment. Furthermore an assessment of ground settlement effects (Coe, 2012) was also undertaken associated with the construction and operation of the Expressway and the expected effects of the settlements on existing buildings, services and transport infrastructure. The Settlements Effects Monitoring Plan (SEMP) (Ainsworth, 2013) defined the settlement monitoring procedure and measures to manage ground settlements associated with the Expressway.

2 GEOLOGICAL AND HYDROGEOLOGICAL SETTING

The Expressway route is bounded, in the east, by the Tararua Ranges. These are steep greywacke hills which have formed by tectonic activity along NESW oriented faults such as the Ohariu fault (which runs along the base of the hills). The Expressway crosses the coastal plain west of the Ranges, an area which has been further shaped by repeated cycles of glaciation that have occurred in the past two million years. During the glacial cycles, sea levels were approximately 120m lower than present, as water was locked in ice-sheets and glaciers. The Tararua Ranges held valley glaciers during these times and physical weathering of rock, combined with sea level fall, contributed to severe erosion in the Ranges generating alluvial fans. These processes, in combination with longshore drift, formed large coastal plains. With each large scale tectonic movement, the rivers altered course and slowly migrated north and south across the alluvial fans depositing gravels, sands and silts. Episodic flood events resulted in finer materials (silts and clays) being deposited further away from the river channels, and in between such events, areas of peat developed in low lying areas between dunes. Sand dunes inter-finger with the peat deposits and rise up to 20 m in elevation along the coast.

The groundwater regime consists of unconfined aquifers in the Holocene sand and peat deposits above a series of unconfined aquifers in the Pleistocene sand and alluvium layers. The alignment passes through the Waikanae Groundwater Zone (WGZ), one of six broad groundwater management zones on the Kāpiti Coast. The key aquifer horizons within the WGZ are the deep Waimea Aquifer and Parata Aquifers, from which the Kapiti Coast District Council (KCDC) production wells abstract water for public water supply. Domestic wells in the area generally abstract water from the shallow Pleistocene and Holocene Sands.

A large number of wetlands occur within the WGZ. Wetlands and lagoons have typically formed in the low lying areas between dunes where peat has been deposited and where the groundwater level is very close to the surface. Wetlands are generally thought to be points of groundwater "discharge" with flows largely sustained by shallow groundwater (Gyopari, 2002). However there is also evidence that some wetlands within the Kapiti Coast are "recharge" wetlands fed by rainfall and run-off perching on the low permeability peat (Allen, 2010). Data collected and modelling carried out as part of this project confirms that both types of wetland occur, depending on the particular conditions at each site.

3 MONITORING

3.1 Groundwater Monitoring

A comprehensive groundwater monitoring programme was established prior to construction commencing, with some 110 piezometers (Figure 3) monitored monthly for at least one year to record natural variations in groundwater levels.

This baseline data has been used to identify natural seasonal groundwater level variations and to set alert and action levels (Figure 4). The groundwater alert level was set at the lowest recorded level (preconstruction) minus the predicted drawdown reduced by 25% or 200mm. A further






100mm was allowed for the action trigger level. Similarly high trigger levels have been set to check against surface ponding. All piezometers were measured monthly, with monitoring frequency increased to twice weekly in active construction areas.

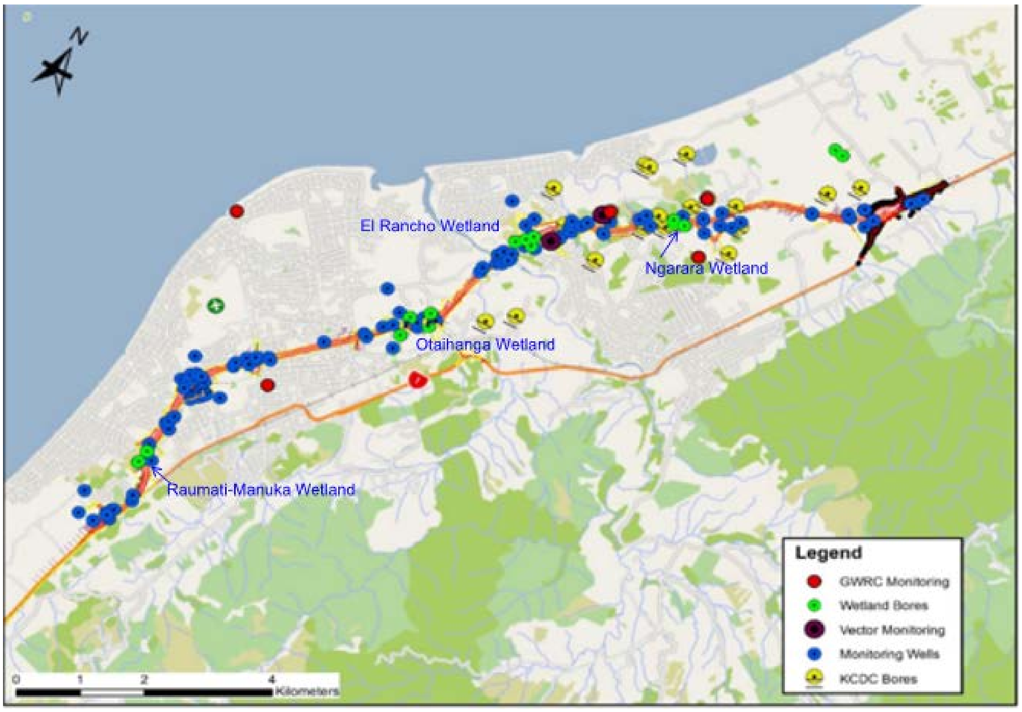

Figure 3: Overview map of groundwater level monitoring

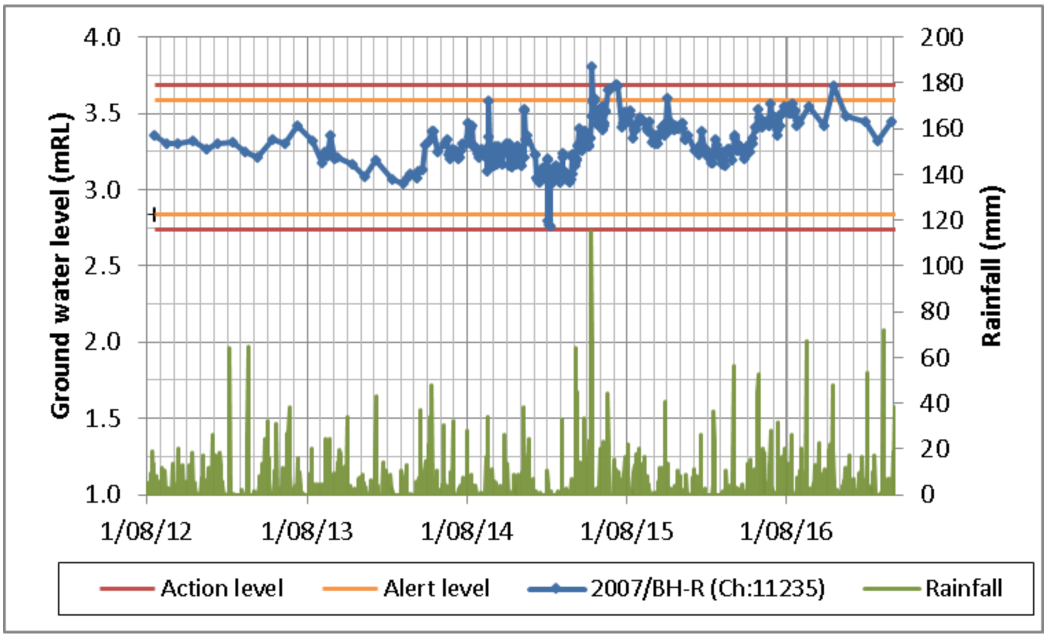

Figure 4: Groundwater levels measured in piezometer 2007/BH-R plotted against trigger levels and rainfall data

Five wetlands: Raumati Manuka, Otaihanga Northern & Southern, El Rancho and Ngarara were identified as "sensitive" for this project and 22 telemetered piezometers were installed to monitor the groundwater levels in these areas (Figure 3). As mentioned above the Kāpiti Coast wetlands

were formed in different ways according to the local ground and groundwater conditions and the water level in some will naturally vary more than in others. The baseline monitoring data was reviewed to identify likely hydrogeological behaviour of each wetland.
In Raumati-Manuka and Otaihanga wetlands groundwater level data indicates a downwards gradient, suggesting that rainfall is held up on the near-surface peaty soils, but slowly infiltrates through them to recharge the underlying sands and gravels. The El Rancho wetland is complex and the northern part of the wetland exhibits a downward gradient and the southern part exhibits a downward gradient with recharge to the underlying soils during the winter months, but is fed by the underlying aquifer during the summer months. In the Ngarara wetland groundwater both recharges and drains the aquifer beneath it. Because even small changes in groundwater level outside the normal seasonal variation may have a deleterious effect on a wetland and the limited number of measurements that were available for the establishment and understanding of the normal wetland water levels, a statistical approach was developed for calculating both high and low trigger levels.

Using data from telemetered piezometers screened in the same shallow aquifers outside the project area and monitored by the Regional Council for many years, the "expected" water level was calculated for each project piezometer based on a multiple linear regression analysis for each reading. A variation of the expected value increased by the margin of error of the 80% confidence prediction interval outside the expected water level triggers an alert for the monitoring bore (Figure 5).

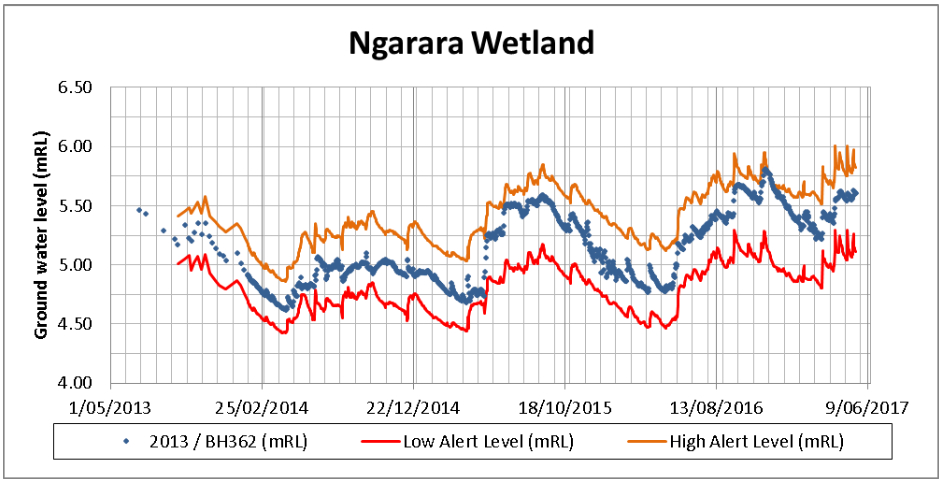

Figure 5: Groundwater levels plotted against statistically calculated trigger levels

This statistical approach was also employed to check exceedances in piezometers that were in active construction zones (earthworks within 200m) but where the works were not anticipated to influence the groundwater levels. The monitoring data demonstrated that the statistically calculated triggers take into consideration district-wide changes in groundwater levels and more clearly distinguish natural effects such as weather patterns (a low trigger level alert during dry summer months) from those resulting from construction activities (pumping during excavation) than triggers set as a standard difference as illustrated in Figures 6a and 6b. This approach allowed for minimum disruption to the construction works from unwarranted alert or alarm readings, and no adverse environmental effects.

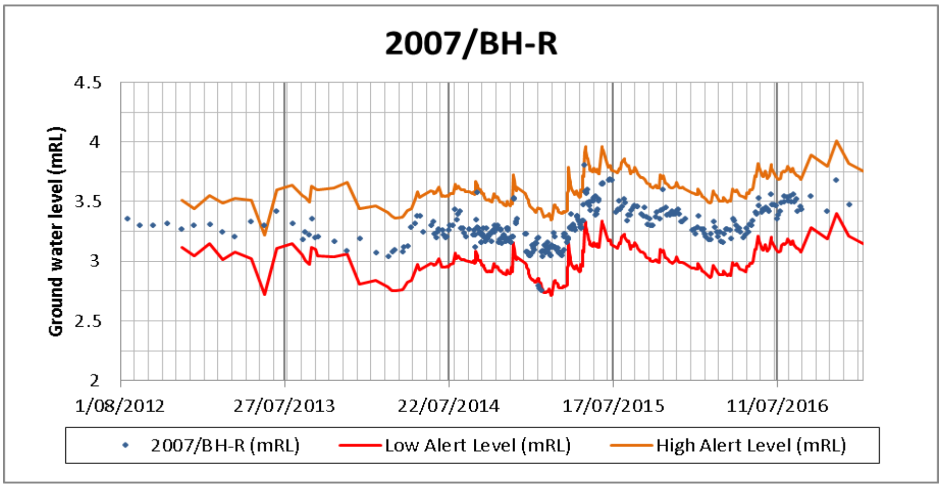

Figure 6a: Observed Water Levels in piezometer 2007/BH-R plotted against statistical trigger levels calculated based on regional data

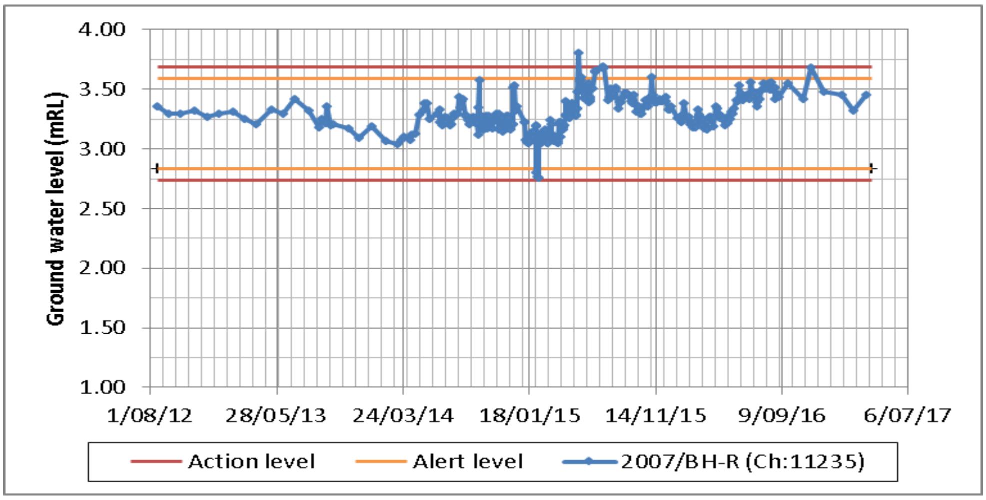

Figure 6b: Observed Water Levels in piezometer 2007/BH-R plotted against constant trigger levels

3.2 Settlement Monitoring

Consolidation settlements from embankment construction (preload and surcharge and groundwater lowering from excavation and replacement of peat) have been analysed in 13 cross sections along the length of the expressway. The location of one of the cross sections analysed is shown in Figure 7. Figure 8 presents the predicted combined settlement for that location and Figure 9 the monitoring results from one of the marks in that section.

Approximately 100 survey marks along the length of the expressway were installed and regularly monitored to provide information to compare to the settlement estimates. Monitoring marks were placed as far as practical to match with cross sections that have been used for the settlement estimates and extended out from the Expressway where settlements were expected to be greater than 12.5mm. Marks were also placed at specific stormwater features where groundwater drawdown of more than 0.1m was predicted and they coincided with groundwater level monitoring piezometers in selected locations. The marks were installed and monitored for vertical movement with 13 sets of baseline values taken during the year prior to the Expressway construction commencing. The lowest preconstruction value was used as the base value to calculate differences in vertical movement (where positive movement indicates settlement). The trigger level for each mark was set equal or lower than the expected/ estimated settlement at that location (Figure 9).

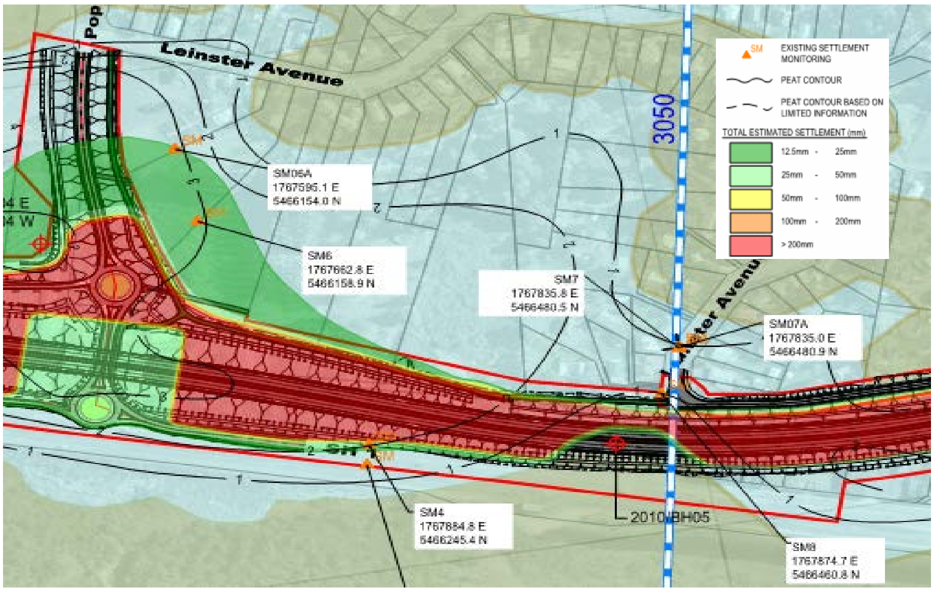

Figure 7: Settlement Markers in Cross Section 3050

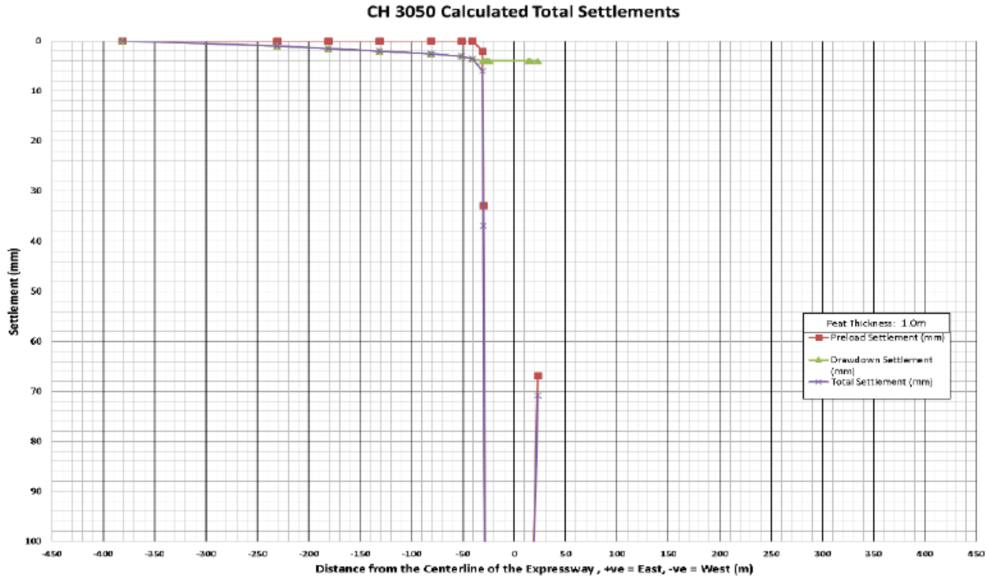

Figure 8: Predicted settlement at Cross section 3050

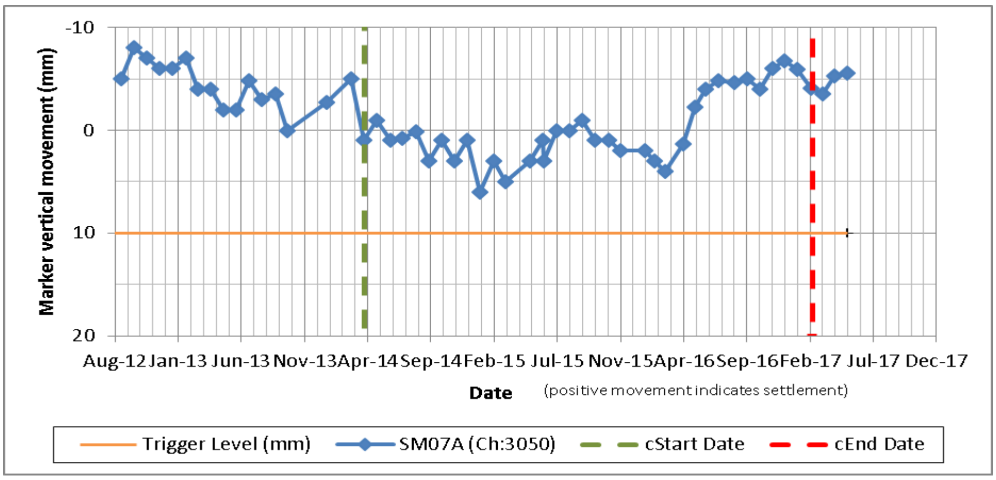

Figure 9: SM07A mark vertical movement (cross section 3050)

Monitoring frequency following the establishment of the baseline was reduced to quarterly and increased to monthly when active construction started (earthworks commenced within 200m of a particular location). When pavement construction was completed the monitoring is reduced to quarterly for 6 months and to half yearly for two years following that. The predicted settlements were generally less than 25mm beyond the edge of the earthworks. In areas of thicker peat deposits, the predicted settlement was in the order of 25 to 50mm up to 20m from the earthworks footprint, reducing to less than 25mmm beyond this. Monitoring data generally were within the magnitude and range of predicted settlements (Figures 8 and 9). Two exceedances were recorded (i.e. monitoring results indicated movement outside the expected range) and additional monitoring was undertaken and an assessment of effect of this movement was undertaken.

4 CONCLUSIONS

A comprehensive groundwater monitoring and settlement monitoring programme was established prior to construction commencing. In order to differentiate construction effects from normal seasonal variations, a statistical approach was developed. Recorded groundwater levels and settlement have generally remained within the consented levels. Some exceedances were recorded however a pragmatic approach to monitoring and management of these exceedances allowed works to continue with minimal disruption and no adverse environmental effects.

The monitoring data demonstrate that the statistically calculated triggers take into consideration district-wide changes in groundwater levels and more clearly distinguish natural effects such as weather patterns from those resulting from construction activities than triggers set as a standard difference.

5 ACKNOWLEDGEMENTS

The authors would like to thank the Mackays to Peka Peka Expressway Alliance, and in particularly the NZ Transport Agency, for permission to publish this paper.

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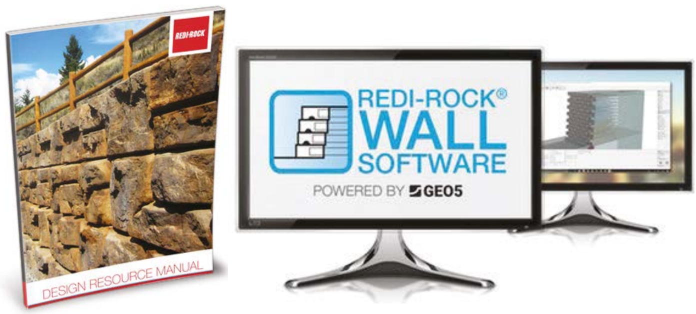

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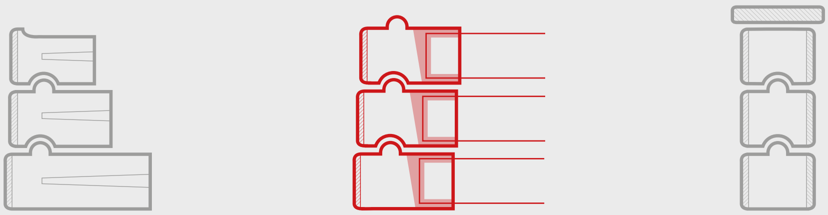

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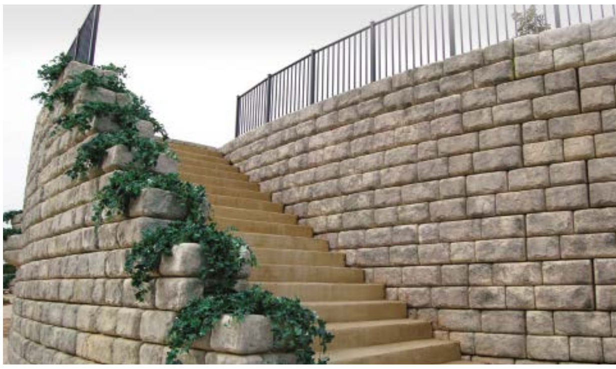

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Session Chairs' Choice 2

Gislaine Pardo Tobar

Gislaine is a PhD candidate at the University of Auckland, in the Department of Civil and Environmental Engineering. Currently, the main focus of her research is to assess environmentally friendly materials as an alternative to mitigate soil liquefaction, trying to combine multidisciplinary cooperation through chemical-environmental and geotechnical areas.

Experimental study on the self-healing effect of laponite on the liquefaction resistance of sand

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Keywords: earthquake, liquefaction, cyclic simple shear, countermeasure, nano-material

ABSTRACT

Currently, there is an increasing interest in providing more sustainable solutions for mitigating liquefaction and interdisciplinary work has started to introduce the use of nano-materials for this purpose. These materials have some advantages when compared with traditional techniques. For instance, they could be injected at low pressures or delivered into the natural ground water flow to treat a target area through passive remediation, which leads to less carbon emission production compared with normal grouting. In addition, after they gel, the nano-materials can provide increased shear strength and cohesion. This study focuses on the application of laponite, a synthetic nano-clay with the same structure as natural clays. Laponite suspensions have thinning behaviour characteristic in which its viscosity decreases with increase in shearing rate while keeping its gel properties. Moreover, after the shearing load is removed, the suspension recovers its viscosity in a self-healing process. In this study, the shear resistance of laponite suspension is studied by rheological measurements and the effect of the addition of 1% laponite (by weight) on the liquefaction resistance of the host sand is evaluated through cyclic simple shear tests. Results indicate that the number of cycles required to reach liquefaction is increased considerably for low shear stress conditions. In addition, by re-testing samples that have undergone liquefaction, the ability of the mixture to "heal" after liquefaction is confirmed. The results reaffirm the potential of laponite as an environmentally-friendly material for ground remediation purposes.

1 INTRODUCTION

To provide more sustainable solutions to mitigate soil liquefaction, interdisciplinary work has started to introduce nano-materials for soil remediation. Some of them, e.g. colloidal silica, bentonite and laponite, have been proven to be effective in increasing the liquefaction resistance of soils.

Colloidal silica is a chemical grout than can provide cementation and can restrain shear strain occurrence (e.g. Gallagher and Mitchell, 2002; Gallagher et al, 2007). Some researchers have performed cyclic triaxial tests (Gallagher and Mitchell, 2002), resonant column tests (Spencer et al., 2007), and centrifuge model tests (Conlee et al., 2012) and they found that colloidal silica suspensions injected into clean sand can increase the shear modulus and reduce strain deformation. These tests have reported that 10% by weight of colloidal silica suspension is enough to reduce the susceptibility to liquefaction. Colloidal silica seems to be very promising in passive site remediation application near existing structures.

On the other hand, laponite and bentonite are nano-clays that can modify the pore fluid (e.g Rugg et al., 2011; El Mohtar et al., 2013; Santagata et al., 2015; Ochoa-Cornejo et al., 2016) i.e. convert it to a solid-like fluid and delay the generation of excess pore water pressure. Bentonite and laponite suspensions make use of the hypothesis that highly plastic clays increase the resistance to liquefaction (Santagata et al., 2015).

Laponite has received attention in recent years due to its thixotropic properties that make it promising in mitigating liquefaction. Figure 1 schematically describes this phenomenon. Laponite's thixotropic behaviour is related to the flat-disc shape of its crystal (Figure 1 a, b) and its chemical composition that naturally makes its surface to be negatively charged and its edges positively charged (Barnes, 1997). When dry, its particles conglomerate with each other and form an aggregate of silt/clay size. In suspension, the particles disperse and arrange themselves in a kind of house-of-cards formation (Figure 1 c), and with time, this formation become stronger. Under very small strain, laponite suspension forms a gel that behaves like an elastic solid. When the applied shear stress is larger than a threshold value (or yield shear stress), i.e. $\tau \geq \tau_{ys}$, the network breaks and the material flows with a decreased viscosity (Figure 1 d, e). When the shear stress is supressed and the suspension goes back to rest, the structure is recovered and the resistance to flow increases again (Barnes, 1997). This is an indicator of a self-healing material that can be stable to resist new cyclic shear stress episode.

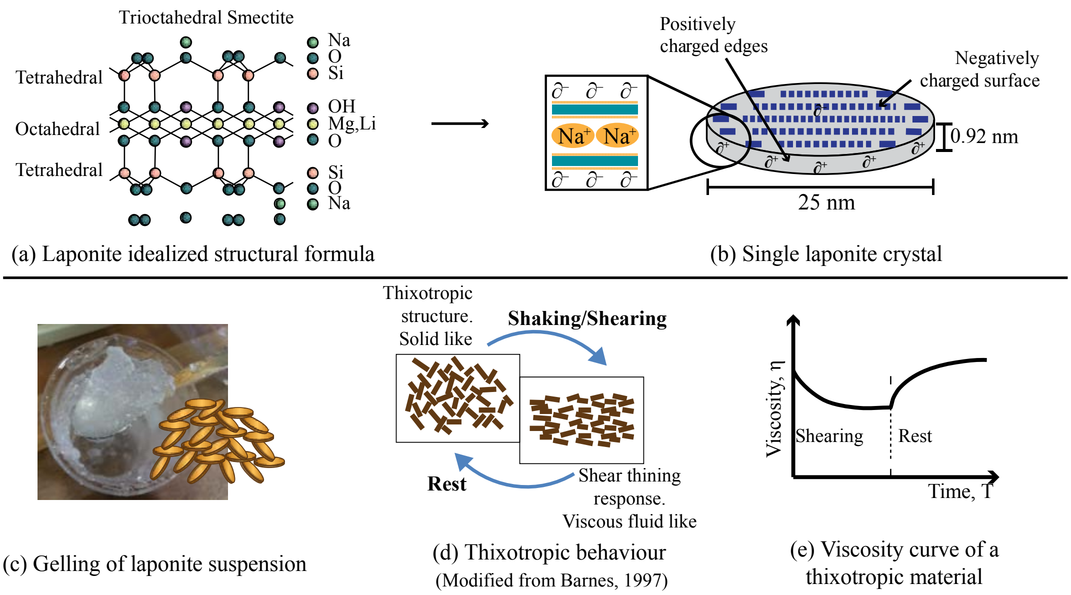

Figure 1: Scheme of thixotropic behaviour on laponite suspensions

Some researchers have studied this material and its potential to mitigate liquefaction. Ochoa-Cornejo et al. (2014) performed cyclic triaxial tests to study its effect on clean Ottawa sand, and they found that 1% laponite by dry mass of sand could increase the number of cycles to liquefaction from about 100 to 600 under similar shear stress. Santagata et al. (2015) performed dynamic oscillatory measurements to compare bentonite with laponite suspensions, and resonant column tests to study their effect on pure Ottawa sand's dynamic properties. They observed that sand mixed with 3.25% laponite suspensions had similar dynamic response to one with 10% bentonite. However, this material is still in evaluation stage and more research is needed to define its applicability in mitigating liquefaction.

In this study, the effect of adding 1% laponite to pure sand (by weight) is evaluated through cyclic simple shear tests and rheological characterization. The main objective is to assess the self-healing capacity of this material and its contribution to the liquefaction resistance of the host sand.

2 MATERIALS USED AND METHODOLOGY

2.1 Test materials

The host sand used in this study is the Mercer sand, sourced from the Waikato River. Figure 2 shows the grain size distribution of this material. It is a very uniform soil (coefficient of uniformity, C_U=2.1). To assess liquefaction, this material has been studied in loose condition.

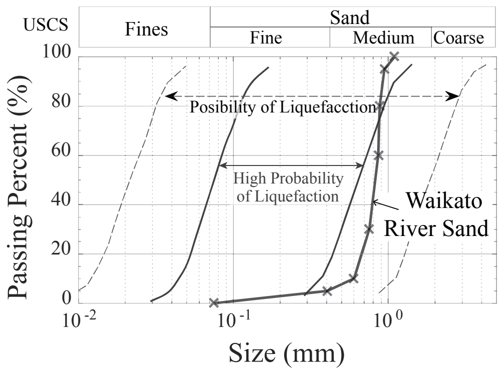

Figure 2: Grain size distribution curve of the host sand

On the other hand, laponite is a synthetic nano-clay with a structure similar to natural clays. It possesses no hazard to humans or to the environment. Laponite has a wide range of application, for example, it is usually used in cosmetics, as surface coatings, in some environmental applications as a barrier to trap gasses or contaminants, as well as in biomedical applications (e.g. Kwak, 2010; Tritschler et al., 2016).

In this study, Laponite RD was used, which is a general purpose rheology modifier. When a rheology modifier is dispersed in a fluid, it changes the way in which this fluid deforms when a shear load is applied. In this case, laponite changes the way the water flows when the cyclic loads are applied. To quantify the flow characteristic of laponite suspensions, rheological measurements were performed to measure the shear resistance of laponite suspension and to evaluate the gelling process. In addition, to study the effect of laponite on the liquefaction resistance of sand, undrained cyclic simple shear tests were performed.

2.2 Methodology

The soil samples were prepared by a modified slurry deposition method (Ishihara et al., 1978; Khalili and Wijewickreme, 2008). This method is useful in preparing highly gap-graded samples or, in this case, in preparing uniform sample consisting of granular material and nano-clay in saturated conditions. This method is very replicable and samples with a sand skeleton void ratio of e_{sk}=0.7581 can be prepared, with less than ±2% of variation. The laponite treatment was fixed at 1% by weight of sand. Then, the amount of water required to have 100% saturation was computed, leading to a laponite concentration by weight of water as 3.4%.

The procedure to prepare the samples is illustrated in Figure 3. First, laponite suspension with a concentration of 3.4% was prepared by pouring the laponite powder in deaired-deionized water, and mixed with magnetic stirrer for 20 minutes at about 1100 rpm. Then, enough suspension was poured into the dry soil, and the mixture was stirred until a uniform slurry was produced. The remaining suspension was poured into the simple shear base, and the slurry previously prepared was poured inside the mould with a spoon (Figure 3c). In some cases, gentle tamping was applied to fit the sample within the target volume. Finally, the excess amount was removed and collected in a container to compute the amount of treatment, which varied from 0.99% to 1.15% of laponite by weight of sand.

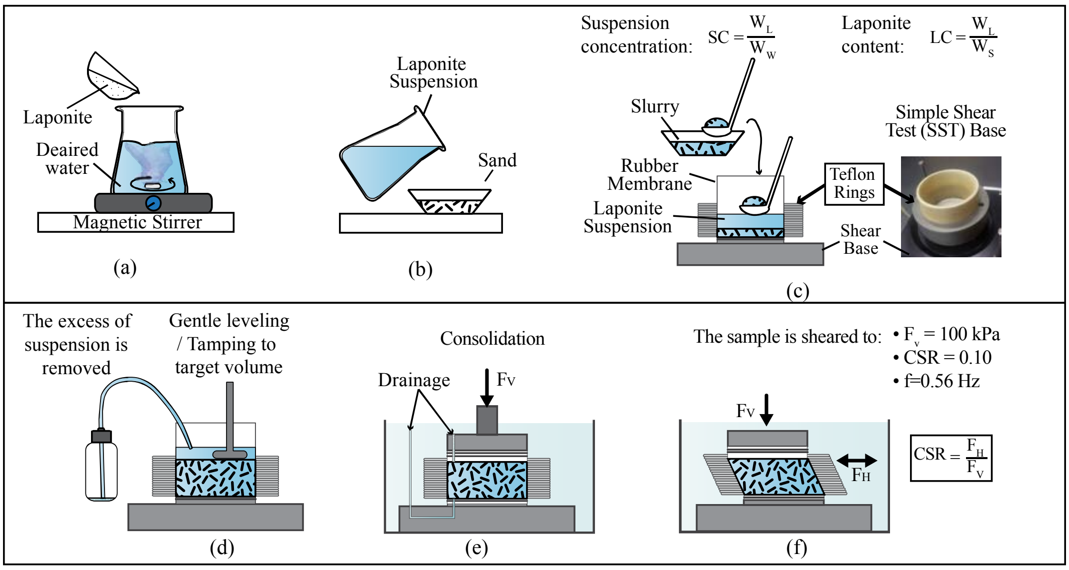

Figure 3: Sample preparation for specimens tested on CSST

2.3 Rheological measurements

Suspensions of 3.4% laponite by weight in water were prepared as described above (Figure 3a) and stored in a sealed container. The rheology tests were performed using a Physica UDS 200 rheometer with a cone-plate measuring device (Figure 4a). To avoid errors due to evaporation, the tests were conducted to last no more than 1 hour, and only a small portion of the suspension was used at each time with the rest of the batch stored in the sealed container. The first tests were performed almost immediately after the suspension was prepared, and then the tests were repeated with different specimens from the same suspension every 10 hours for 2 weeks.

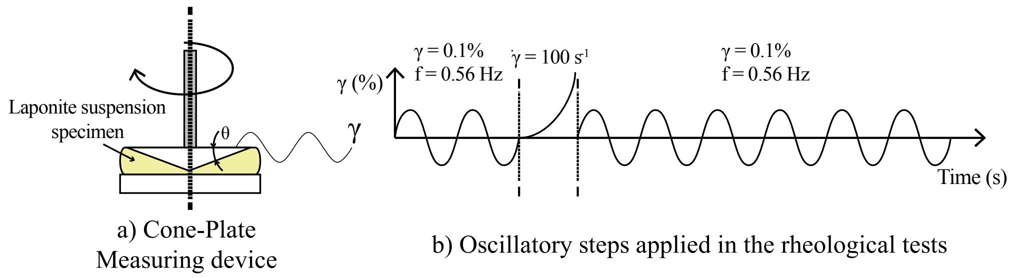

Figure 4: Rheological measurements

Each test consisted of 3 steps (Figure 4b). In the first step, small strain amplitude oscillations (γ=0.1%) were applied at a constant frequency 0.56 Hz. The purpose of this step is to assess the current shear strength of the sample, so it did not last for more than 10 minutes. In the second step, a high shear rotation $\dot{\gamma}$=100 s^{-1} was applied, with the purpose of breaking the internal structure of the developing gel. Finally, the sample was allowed to recover under small strain oscillations γ=0.1% at f=0.56 Hz, and the recovery was evaluated in terms of the time required to reach the same shear resistance that the sample had at the beginning of the test (i.e. in Step 1).

2.4 Cyclic Simple Shear Tests (CSST)

Specimens with a diameter of 63 mm, and height of 24 mm were tested in cyclic simple shear machine. To study the capability of laponite to recover, sand samples treated with laponite were tested in three phases, as depicted in Figure 5. First, the samples were consolidated with vertical load of σ'_{v0}=100 kPa (Figure 3e) for 72 hours. This was followed by the cyclic shearing stage, in which the cyclic shear stress ratio (CSR=τ/σ'_{v0}) was kept constant at 0.1. These two steps were repeated two more times (Phases 2 and 3).

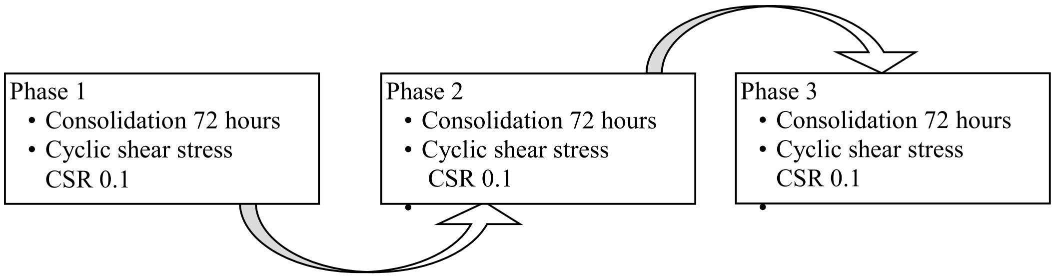

Figure 5: Scheme of the phases in testing laponite-treated sand samples

The sand skeleton void ratio at the beginning of each phase was computed, and pure sand samples were prepared with similar void ratios and compared with the response of laponite samples at each phase. The pure sand samples were consolidated for about 2-3 hours and then tested at CSR=0.1.

3 RESULTS AND DISCUSSION

3.1 Rheological measurement results

Figure 6a shows the shear modulus of the suspension versus the elapsed time since it was first prepared. From this result, it can be observed that at least 72 hours are required for the gel to have at least 80% of its final shear strength. This observation is consistent with those obtained by other researchers (e.g. El Howayek, 2011; Santagata et al., 2014; Ochoa-Cornejo et al., 2016).

On the other hand, Figure 6b shows the results only from the recovery step (described in Section 2.3). The markers' colour is proportional to the elapsed time since the suspension was first prepared. The results indicate that after enough time has elapsed (T_e>3.5 h), the recovery time is almost instantaneous, i.e. within minutes, the shear strength has recovered. Consistent with Figure 6a, for samples just prepared, the time to reach a steady value is about 72 hours.

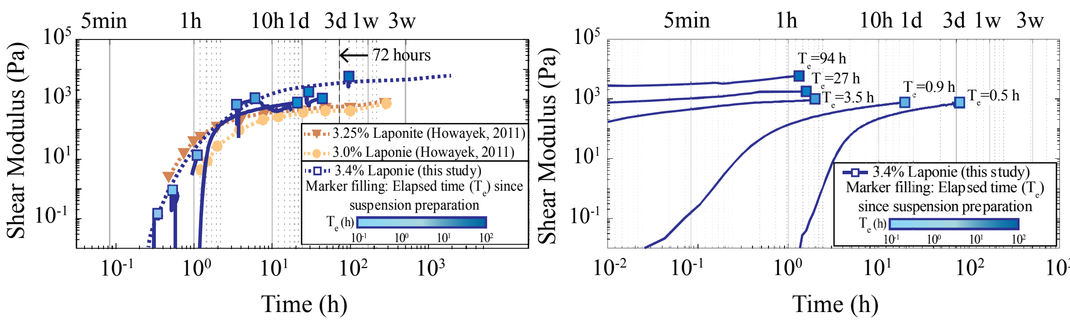

Figure 6: Time effect on laponite suspensions: (a) elapsed time since suspension preparation; and (b) recovery step

3.2 CSST results

Figure 7 shows the results from cyclic simple shear test for *CSR*=0.1, in terms of increase in excess pore water pressure ratio and double amplitude shear strain development. In the figure, the size of the marker is proportional to the relative density after consolidation, while the colour of the marker is related with to the time that elapsed since the sample was first prepared, i.e. white colour for less than 3 hours and black colour for more than 216 hours. The first column represents the first phase of the tests in which the samples were prepared with a skeleton void ratio of e_{sk}=0.758±0.015. The samples treated with laponite shows a gradual development in pore water pressure ratio until the 10th cycle, afterward the rate of development of pore water pressure increases, reaching liquefaction (i.e. r_u≈1) just a little later than the pure sand samples. Columns 2 and 3 show the results of re-testing Laponite samples two times after liquefaction. The initial void ratio before consolidation was estimated to be e_{sk}=0.710±0.015 and e_{sk}=0.681±0.003, respectively, for Phases 2 and 3. These samples were compared with pure sand specimens prepared with similar void ratios and the effect of time and self-healing is clear in the graphs.

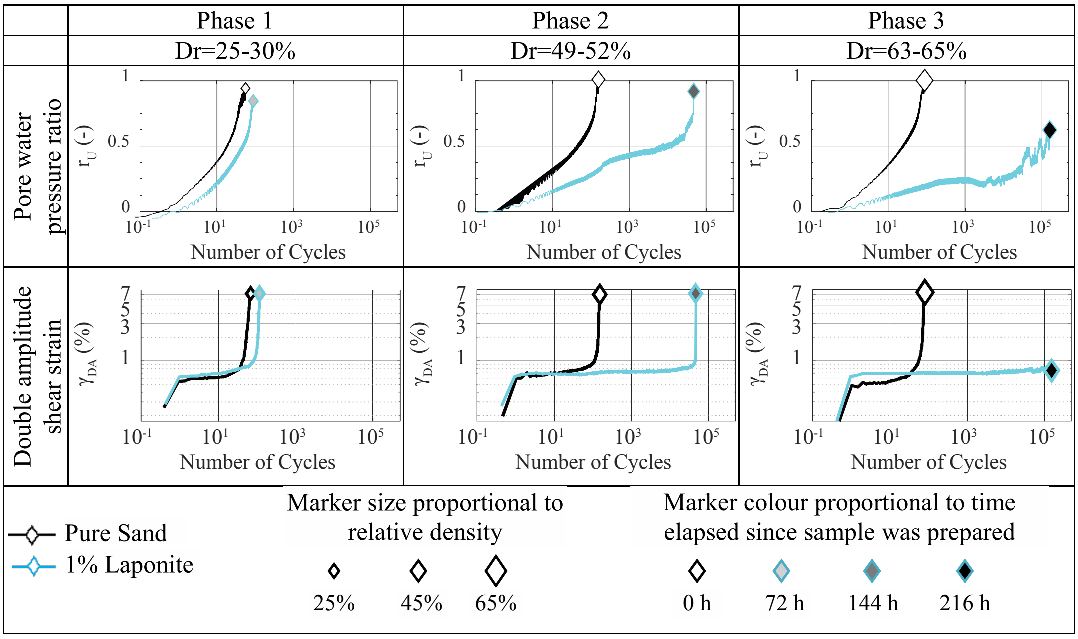

Figure 7: CSST results for *CSR*=0.1

These results are consistent with the rheology results shown in Figure 6, in which laponite suspension recovered very fast (within an hour), and the suspension continued to gain shear strength with time. Thus, in Phase 2, the number of cycles required for liquefaction to occur increased by about three orders of magnitude with the treatment; in Phase 3, the sample underwent more than 10^5 cycles with shear strain less than 1%. Moreover, during Phase 3, the excess pore water pressure oscillated more at the end of the test, possibly indicating that the sample was recovering while the test was in progress.

4 CONCLUSIONS

This paper presented the results of rheologic measurements on laponite suspensions and cyclic simple shear tests on laponite-mixed sands at *CSR*=0.1, in which the self-healing characteristic of laponite was confirmed. The results from the rheometer tests indicated that at least 72 hours are required after the suspension has been prepared for the fluid to provide enough resistance and to behave as a gel; after that period, if shear stress was applied, the suspension yielded, but it recovered within one hour. These results were consistent with what was obtained in the cyclic simple shear tests, where the same sample was subjected to 3 phases of consolidation and cyclic shear application. Even though the void ratio of the sample decreased after each phase, the improvement was considerable when compared with pure sand samples with similar relative density. Therefore, the samples did not only recover, and being capable to resist a new set of cyclic stresses, it endured even more cycles because the laponite suspensions continued to harden.

These results indicated that with only 1% of laponite by weight of sand, it was possible to increase considerably the liquefaction resistance of pure sand. However, the results of CSST suggested that more than 72 hours may be required for effective treatment. After a sufficient time had elapsed after the sample preparation (more than 144 hours), the sample underwent more than three orders of magnitude of cyclic loading application than pure sand samples with similar relative density to liquefy. However, given the thinning behaviour of laponite suspension (i.e. decrease viscosity with increase in shear), such effectiveness observed for *CSR*=0.1 many not occur for higher *CSR*; more tests are currently planned to study the response at different levels of *CSR*, and different elapsed times and relative densities.

Laponite is currently being used in preliminary stages as a soil stabilizer, but the results presented in this paper reaffirmed their potential for ground remediation purposes. In practice, laponite could be injected or deposited in the ground and allowed to flow in passive remediation, where it could provide a more environmentally-friendly alternative to stabilize the soil since it poses no hazard to people and environment, and could help reduce carbon emission.

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Performance of a stone column foundation system subjected to severe earthquake shaking

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ABSTRACT

The Deans Stand at Lancaster Park (formerly AMI Stadium) in Christchurch, New Zealand is of modern reinforced concrete design and construction. It is largely supported on a hybrid foundation system comprising ground beams and stone columns that extend part way through a relatively thick layer of liquefiable sands and silts. Portions of the structure were supported on screw piles taken to greater depth. This paper describes the design objectives and the performance of the foundation system during the Canterbury earthquake sequence (CES) of 2010-2011. The Canterbury earthquake sequence included several strong and damaging ground motions, with the most severe shaking being from the 22 February 2011 Mw 6.3 event, which had an epicentral distance of 6km from the stadium. That event resulted in peak ground accelerations at the stadium of approximately 0.5g horizontally and 0.7g vertically. Extensive liquefaction and subsequent ground settlement was recorded in the vicinity of the stadium.

Damage to the Deans Stand included bulging, loosening and contamination of the stone columns and large global and differential settlement of the structure. The damage sustained during the earthquakes has resulted in the stadium not being used since the February 2011 earthquake. Physical and numerical investigations of the site and foundation system performance at Lancaster Park commenced in 2011. The physical investigation findings have been compared with data obtained before and during stone column construction, providing valuable insights into the behavior of stone columns under strong earthquake shaking. Evidence of stone column contamination and of the loosening of densified ground between the columns is presented. Further insights are obtained from comparison of the performance of an earlier and similar sized stand at Lancaster Park which was supported on partial depth stone columns with a thick ground floor raft slab rather than isolated ground beams. The raft slab foundation appears to have performed much better than the ground beam system during the earthquake sequence.

1 INTRODUCTION

The Canterbury Earthquake Sequence (CES) of 2010-2011 caused widespread significant damage to buildings and infrastructure across Christchurch City, including those at Lancaster Park stadium. The most severe shaking was due to the 22 February 2011 magnitude Mw6.3 earthquake with an epicentre approximately 6km from central Christchurch.

Lancaster Park Stadium was the largest sporting and events venue in Christchurch with a seated capacity of approximately 40,000. The stadium sustained substantial damage to foundations and structure as a result of the earthquake sequence, and in particular the February 2011 earthquake, and, as at 2017, the facility remains closed.

Engineering damage assessments following the earthquakes included site inspections/observations, testing of various materials in the building and foundations and theoretical predictive analysis of foundation and structure damage due to shaking and ground movements.

Although all of the facilities were damaged to varying degrees, this paper concentrates only on the foundation performance of the most recently built of the two large stands, the Deans Stand. It highlights, in particular, the behavior of stone columns under strong earthquake shaking and the value of heavy structural raft foundation systems at liquefiable sites. More complete details of the entire stadium and the earthquake effects on it are presented in Whittaker et al. (2017).

This paper documents an important and rare case history of the seismic performance of foundations bearing upon ground treated with vibro-replacement stone columns. It will be useful for practitioners faced with ground improvement and foundation system design for liquefiable sites.

2 LANCASTER PARK STADIUM

2.1 Description

At the time of the Canterbury earthquake sequence Lancaster Park stadium comprised four main spectator stands and associated access structures including ramps, stair towers and link bridges, refer Figure 1. The Hadlee (north), Tui (south), Paul Kelly (west) and Deans (east) stands are named after either prominent local sports people or commercial sponsors. The oldest existing Hadlee Stand opened in 1995, Tui and Paul Kelly Stands were opened in 2000, and Deans Stand in 2010.

Gavin Alexander

Gavin is a Senior Technical Director in Beca's Geotechnical Engineering group. He is a Fellow of Engineering New Zealand and a past Chair of the NZGS. Gavin worked on Lancaster Park (the former AMI Stadium) with his structural colleagues and international experts for several years to help Christchurch City Council understand the potential level of damage to their asset resulting from the Canterbury Earthquake Sequence. This paper sets out some aspects of that work. It was first presented earlier this year at PBDIII, Performance Based Design in Earthquake Geotechnical Engineering, held in Vancouver. Gavin is grateful to the NZGS for awarding him a travel grant from the 6ICEGE fund to attend that conference.

2.2 Typical Ground Profile

A generalized pre-earthquake soil profile beneath Lancaster Park is presented in Table 1. The sand unit presents a wide range of relative densities and varying silt content. As a result, parts of the unit are more susceptible to liquefaction than others. Groundwater levels at the site are high, typically within 1-2m of the ground surface.

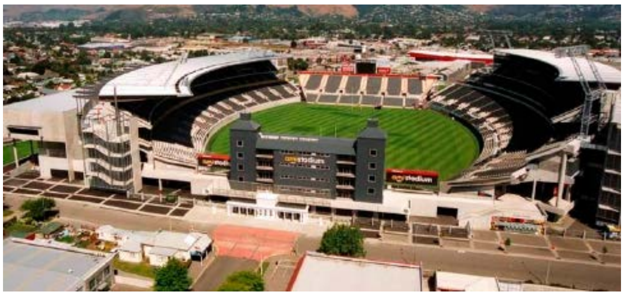

Figure 1. Lancaster Park stadium prior to the earthquakes looking south. Deans Stand is on the left and Paul Kelly Stand on the right

Table 1. Generalized Soil Profile.

Soil layer	Depth to top of layer (m)	SPT N value (measured)
Upper Silt, with sand layers, some organics, soft in places[1]	0	2-15 (5 typ.)
SAND, loose to dense interbedded layers	2-3	3-50+ (22 typ.)
LOWER SILT, soft	16-19	0-24 (7 typ.)
Riccarton GRAVEL, dense to very dense	23-24	50+

[1]partially replaced with hardfill during construction

2.3 Foundation and Structural Form

The foundation and structural construction type and form of the two main stands are briefly described in the following sections. Figures 2 and 3 show cross sectional diagrams of the two main stands.

2.3.1 Deans Stand

Deans Stand, designed in 2008, is a three level covered stand, curved in plan, with an overall seated capacity of around 13,500 spectators. Associated ancillary structures include a multi-level access ramp hall, north and south stair towers and an attached corporate hospitality space (Deloitte Lounge) at level 4. The stand superstructure is of a similar arrangement to Paul Kelly, comprising radial shear wall/frame structures and a circumferential moment frame along the length of the stand.

Foundations beneath the main stand are a composite system comprising shallow reinforced concrete ground (grade) and tie beams on stone columns. Ground improvement was undertaken to limit static settlement and to protect the structure against liquefaction induced ground and foundation failure.

This ground improvement was constructed only beneath the footprint of the main part of the Deans stand structure. It comprised vibro-replacement - stone columns extending to a depth of approximately 9m below ground and concentrated in bands centred under the structural grids. Stone columns were notionally 900mm diameter, constructed on a variable sized (up to 2.7m) triangular grid. The minimum area replacement ratio was approximately 10%. Heavier structural loads were concentrated on defined rows of stone columns. The stone columns are stiffer than the ground around them, so they carry a large proportion of the gravity load from the overlying structure. They improve shallow bearing capacity and reduce settlement under static loads. The liquefiable ground below the stand was only treated to partial depth, with the intention of creating a non-liquefying crust or soil raft. A raft slab was not provided at ground level; instead, flexible pavement was constructed between the radial and circumferential ground beams.

The foundations for the adjacent access ramp hall, stair towers, link structures and Deloitte Lounge comprised screw piles extending to a depth of approximately 15m, with reinforced concrete pile caps and tie beams.

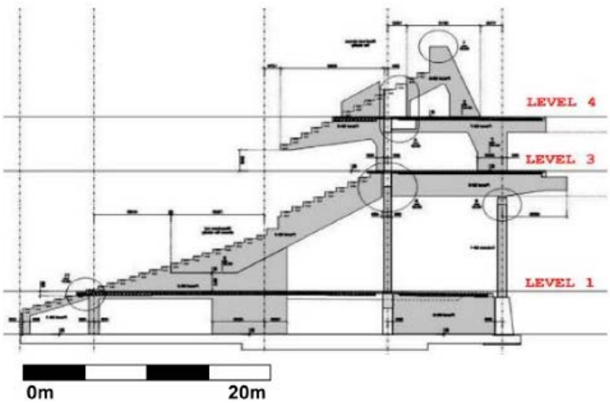

Figure 2. East-west section through Deans Stand

2.3.2 Paul Kelly Stand

The Paul Kelly Stand, designed in 2000, is a four level partially covered structure, curved in plan, with an overall seated capacity of around 17,000 spectators plus corporate boxes and hospitality lounges.

The stand structure is similar in form to Deans. The foundations are a composite system comprising reinforced concrete shallow strip footings, for the lighter loaded field side structure, and a continuous raft beneath the main heavier loaded part of the superstructure, sitting on a block of improved ground.

Ground improvement comprised vibro-replacement using a triangular grid of stone columns extending to a depth of approximately 9m below ground and covering the footprint of the stand and the associated structures. Stone columns were notionally 600mm diameter at 1.4m centres, corresponding to an area replacement ratio of approximately 16%. Heavier building loads were more uniformly distributed over groups of stone columns than at Deans. As at Deans, the liquefiable ground below the stand was only treated to partial depth, thereby attempting to create a non-liquefying crust to support the building.

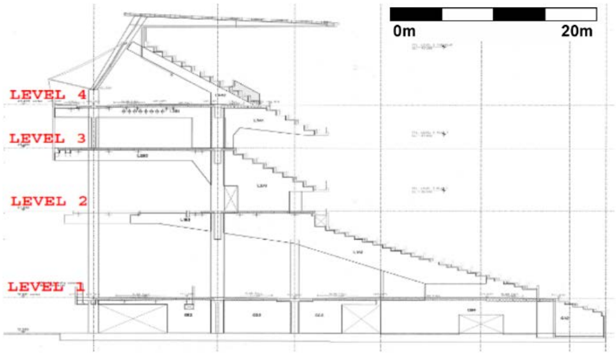

Figure 3. East-west section through Paul Kelly Stand

3 CANTERBURY EARTHQUAKE SEQUENCE

The Canterbury earthquake sequence of 2010 and 2011 included several strong and damaging ground motions including; 4 September 2010 (Moment magnitude M_w=7.1), 22 February 2011 (M_w=6.3), 13 June 2011 (M_w=5.6 and M_w=6.3) and 23 December 2011 (M_w=5.8 and M_w=6.0). Numerous aftershocks exceeding M_w=5 also occurred.

The most severe shaking at Lancaster Park Stadium occurred during the 22 February 2011 earthquake. Although the earthquake was of relatively moderate magnitude, and the duration of strong shaking was only a few seconds, the earthquake epicentre was shallow and very close – approximately 6 km south-east of the stadium. That event resulted in peak ground accelerations (PGA) at the CCCC seismograph 700m northwest of the stadium of approximately 0.5g horizontally and 0.7g vertically. According to the Canterbury Earthquakes Royal Commission reports (2012), response spectra derived from ground motions measured at stations around the central city exceeded the 1 in 2,500 year acceleration response spectra given by the relevant NZ structural design standard, NZS 1170.5:2004 (2004). To put this in context, the stands were designed for 1 in 1,000 year shaking.

4 PERFORMANCE OF SITE AND STRUCTURES

Widespread damage, a result of both earthquake shaking and foundation/ground settlement occurred to all of the Lancaster Park Stadium structures. The area around the stadium experienced considerable liquefaction due to the 22 February 2011 earthquake. Large amounts of ejected sand (sand boils) were observed in the area. Significant total and differential settlement of commercial and residential buildings was evident, together with uneven land surfaces.

Our assessment of liquefaction potential and resulting settlement occurring within and beneath the stone column zones from the February 2011 earthquake used the method of Idriss et al. (2008). It indicated that significant liquefaction occurred within the stone column zone. We determined that the upper 3m of the sand layer in the stone column zone liquefied, (3-6m depth) and that where the ground improvement was more successful (6-9m depth) the sand was much less susceptible to liquefaction. Below the ground improved zone, the sand was determined to be more susceptible to liquefaction.

Ground level changes and building settlements were measured by level surveys and by reference to LiDAR data from before and after the earthquakes. Total settlement of between 200 and 500mm was recorded at the Deans Stand following the February 2011 earthquake. Marked differential settlement was apparent between elements of structures on different foundation types, and across the structures themselves. The main body of the Deans Stand settled between 200 and 500mm (refer Fig.4), with each end tilting backwards around 100mm and the central portion settled approximately 300mm relative to the ends and tilted towards the field by 150mm. Those parts of the Deans Stand supported on screw piles settled less, typically 150 to 200mm.The pattern of settlement is shown on Figure 4. It is unclear whether the observed settlements occurred during or after earthquake shaking. Examples of surface expressions of ground damage at the Deans Stand are shown in the photographs in Figures 5 and 6.

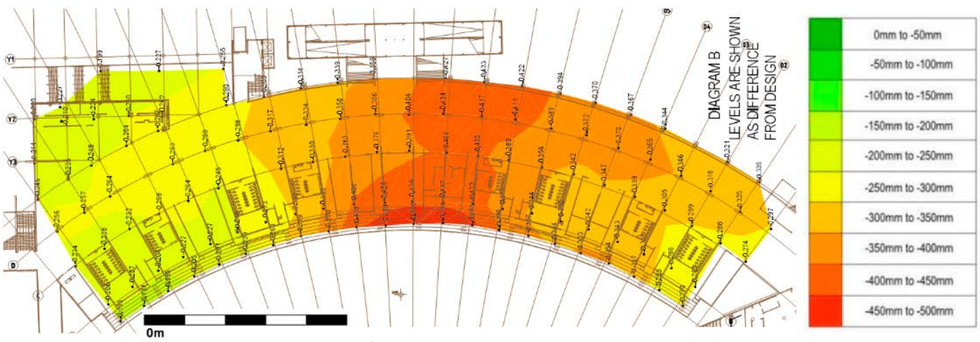

Figure 4. Deans Stand surveyed settlement contours

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Figure 5. Deans Stand ground surface disruption around piled area

Figure 6. Deans Stand pavement bulging between ground beams

The Paul Kelly Stand settled less than Deans, and much more uniformly. It tilted away from the playing field fairly evenly by around 100mm at roof level.

The extent of surface expressions of liquefaction within the confines of the stadium was generally less than for the surrounding area.

5 INVESTIGATIONS

The February 2011 earthquake significantly exceeded design ground shaking levels, and liquefaction was evident from field observations of ejecta and from the settlement which occurred between, beneath and beyond the stone columns. Calculations and field investigations were undertaken to reach this conclusion. The field investigations comprised:

- Test pits to expose the upper portion of stone columns beneath and adjacent to structures
- Machine boreholes advanced through the stone columns, and
- Cone penetration testing of the ground between the stone columns and in the playing field, well beyond the stone column treatment area.

Laboratory testing of clean and contaminated stone column material was undertaken.

Numerical analysis of pore pressure migration and of stone column performance under applied structural loads was carried out. Pre– and post-earthquake conditions were modelled when studying the structural load effects, including consideration of stone column performance under a future design level earthquake. Further details of this analysis are summarized in Whittaker et al. (2017).

6 INVESTIGATION FINDINGS

The investigations demonstrated that earthquake shaking and liquefaction had affected and damaged the stone columns in a number of ways. Our hypotheses, and some of the supporting evidence, are set out below.

The stone columns were contaminated with fine silty sand as liquefaction ejecta flowed in to and upwards through them during and following earthquake shaking. This aspect is discussed further below. It was estimated that this has reduced their drainage performance by approximately 50%, and they were left less able to relieve excess pore water pressure build-up (the precursor of liquefaction) in future earthquake shaking.

The stone columns beneath the stands have bulged into the surrounding liquefied soils resulting in them dilating and losing strength and stiffness. This aspect was investigated numerically using simplified finite element model simulations of a single column, as described in Whittaker et al. (2017).

The progression from a static pre-earthquake state to a liquefied post-seismic condition was modelled first, and mechanisms that occurred in and around the column, including permanent deformations, were observed. Next, the load carrying capacity of the stone column prior to the 2010/11 earthquakes was estimated using the models, and compared with that which is likely to exist today.

The strength reduction within the stone columns was compounded by contamination with fine grained soils, and the net effect was estimated to be in the order of 15 to 20% loss of original strength. As a result, they were considered to be less able to support structural loads under a future liquefaction event. This loss of capacity is expected to inevitably lead to additional settlement of heavily loaded stone columns during future earthquake shaking, particularly under the Deans Stand.

The perimeter stone columns and the lower portion of the stone columns directly under the stands have been subjected to pore water pressure migration from the surrounding and underlying liquefied soils. When combined with earthquake induced shearing, dilation and loosening of these columns is expected to have occurred. This reduces the ability of the perimeter columns to protect the internal columns from the effects of liquefaction beyond the stands. It also reduces the ability of the lower portion of all of the columns to prevent liquefaction between them and thus increases the potential for bulging of the lower part of the columns in future earthquakes.

Liquefaction between the stone columns reduced the confinement of the columns markedly. Following dissipation of excess pore water pressures, the locked in lateral confining pressure developed during stone column construction has been lost, with confinement recovering from the liquefied condition to the equivalent of normal consolidation. This aspect is discussed further below. It was concluded that around 30 to 35% loss of confinement of the stone columns has occurred.

Most critically, when considering future performance of ground that has been densified by vibration, the investigations carried out beneath the Deans Stand to date indicate that liquefaction that has occurred between the stone columns has led to loss of the densification of the surrounding ground achieved by stone column construction. This aspect is discussed further below. While the loss of densification may not be vital for static performance of the foundation system, it was assessed to have significantly reduced performance during future earthquakes.

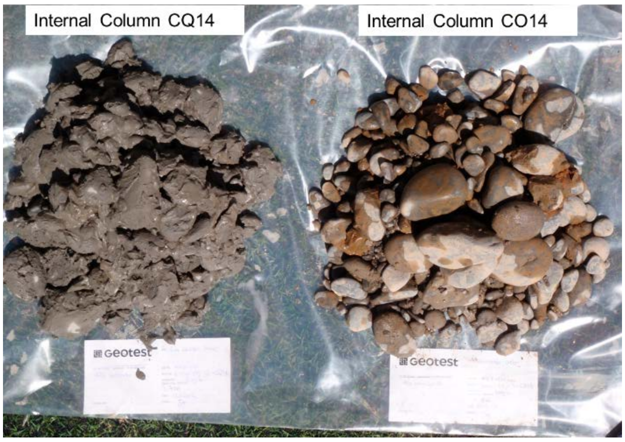

Figure 7. Dirty and clean samples of stone column material – shallow

6.1 Contamination

Most of the stone columns exposed in pits beneath the Deans Stand showed evidence of fine silty sand contamination, while most of the pits excavated beyond the stand on the field side show the columns to be relatively clean. Photographs of samples recovered from near the top of stone columns at Deans are presented in Figure 7, and deeper samples from sonic bores put down through columns are shown in Figure 8.

It is postulated that the weight of the structure drove liquefied soils beneath and surrounding the stone columns into and up the columns directly beneath the stand, and that the effect was much less pronounced within the perimeter columns. This is consistent with the pattern of ejecta and pavement bulging and blistering evident at the Deans Stand.

The stone columns would inevitably have contained a proportion of silty sand introduced from the surrounding ground during the construction process, as noted by Seed et al. (2003). That fine “contamination” is the residue from flushing by large volumes of water during construction, so is expected to contain a much smaller proportion of silty sand than liquefied materials entering the columns following strong earthquake shaking. It is inferred that construction induced “contamination” would readily be mobilized by groundwater flow from the surrounding soil under earthquake shaking.

It is further postulated that the stone columns filled with fine silty sand during the initial significant liquefaction event. Some of that material was observed to have made its way through the ground level pavement in the Deans Stand and manifested as ejecta causing bulging of the asphalt surface. This is shown on Figure 6. The liquefied sand is assumed to have stopped flowing once liquefaction induced subsidence ceased.

The sand remaining in the columns will need to be re-fluidized by earthquake shaking before the columns will work as drains. This re-fluidization will require several significant earthquake pulses, at least. As a result, the internal stone columns will not now work as effectively as drains in the initial period of future earthquake shaking.

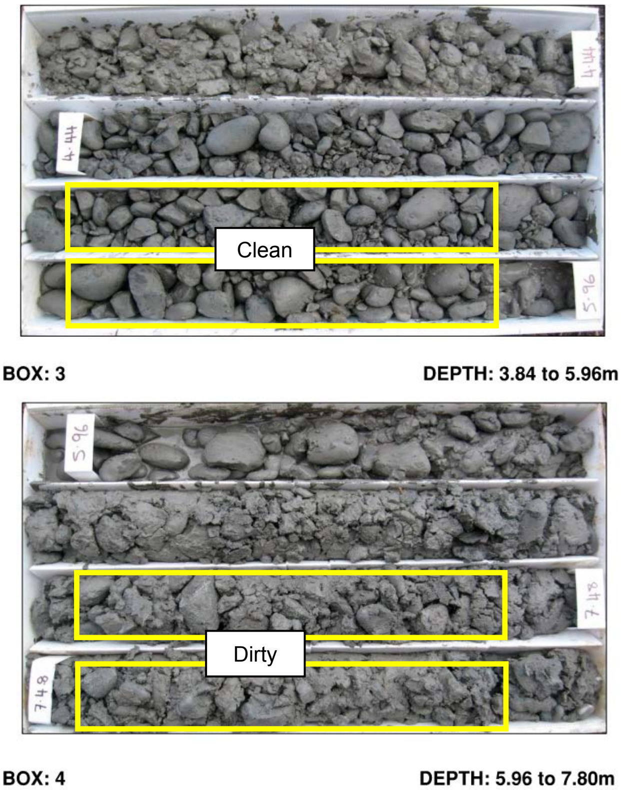

Figure 8. Dirty and clean samples of stone column material – deeper samples (BH3001)

A comparison of gradation curves is shown in Figure 9 and clearly demonstrates the higher fines content in the contaminated (dirty) sample.

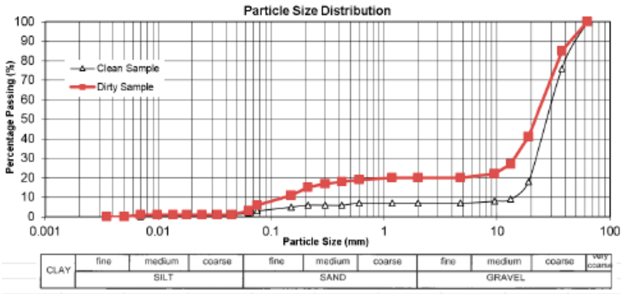

Figure 9. Grading curves of clean and dirty stone column materials (shallow samples)

6.2 Liquefaction between stone columns

Stone column construction densifies surrounding sandy soils, locking in increased lateral stresses. Liquefaction of soil between the columns leads to the loss of these locked in stresses, returning the ground to a normally consolidated state. Furthermore, the post-shaking dissipation of excess pore water pressure from liquefied layers upwards and downwards to non-liquefied layers leads to a degree of strength loss in those non-liquefied layers.

We investigated four locations at the Deans Stand, carefully exposing groups of stone columns so we could locate cone penetration tests (CPTs) mid-way between columns. The CPT results show the ground between the stone columns to largely have returned to its pre-improvement, normally consolidated, state. This is demonstrated by the upper and lower quartile plots of CPT tip resistance presented on Figure 10, which are overlaid on the equivalent range of pre- and post-improvement CPTs presented by Tonkin and Taylor (2012).

7 DISCUSSION

The design intent for the stone column portion of the composite foundation systems supporting both main stands at Lancaster Park stadium was similar – to form a non-liquefying crust that was thick and extensive enough to prevent global instability and reduce post-earthquake settlement. At Deans Stand, there was also a requirement for the stone columns to strengthen and stiffen the near surface silts to limit gravity load settlement to 25mm.

The ultimate design level of earthquake shaking was exceeded by the February 2011 earthquake, with no global failure of the main stands. On that basis, the stone column ground improvement system has achieved its primary (life safety at ULS) design objective. Damage to the stone columns and the ground between them does, however, appear to have significantly compromised their future performance.

Most clearly, the stone columns have been contaminated by silty fine sand. The partial depth of treatment appears to have contributed to this contamination – a direct connection was available to the liquefied ground beneath the columns.

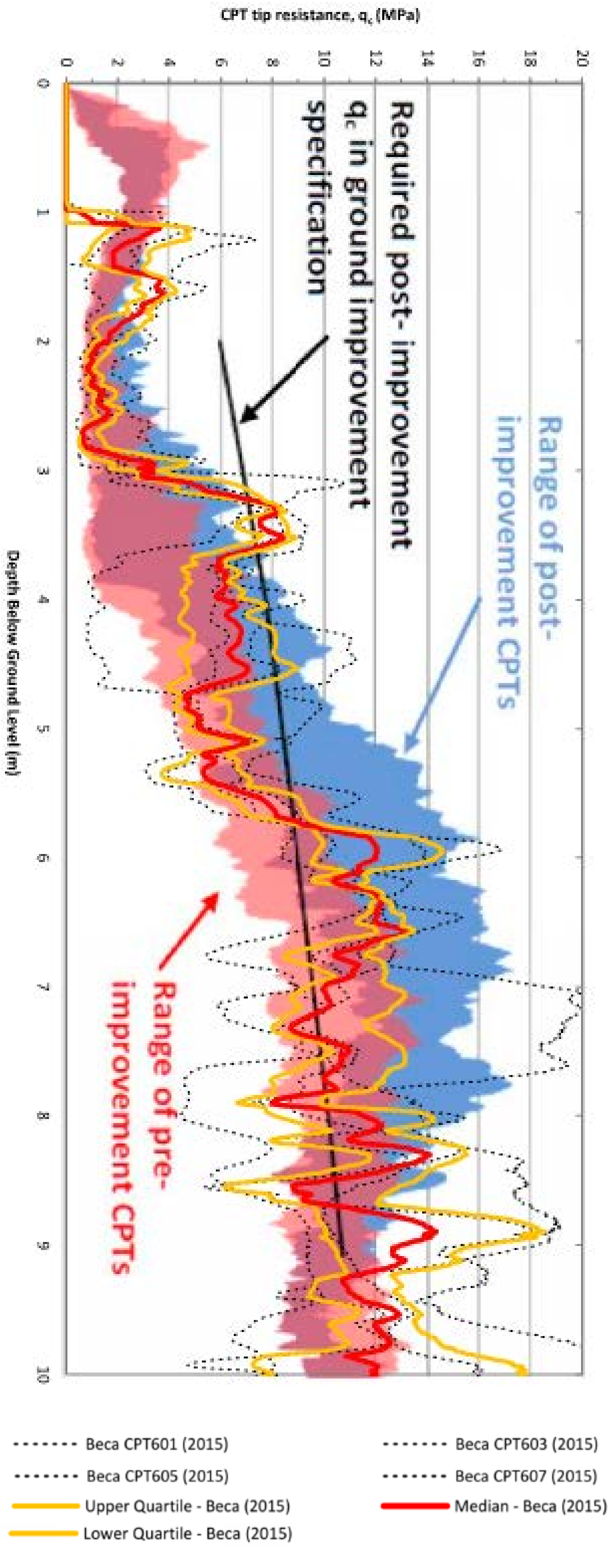

Figure 10. CPT data comparison (Underlying image from Tonkin and Taylor 2012)

Less obviously, but clearly apparent from our investigations, the densification of the surrounding sandy soil achieved by stone column construction has been lost as a result of the ground improvement zone “capacity” or design basis having been exceeded. This aspect is of particular relevance for any ground improvement project that relies on densification and that is designed for a relatively low return period event – for example for the same return period as a normal importance level building.

Well designed buildings may, in many cases, be economically repairable following an earthquake that exceeds design loadings. Ground beneath them that has been improved by densification and has subsequently lost that densification is much more difficult (and expensive) to repair. In the case of the Deans Stand, a concrete lattice solution was developed, as it was not physically possible to re-densify the stone column zone without demolishing the stand.

Partial depth ground improvement to mitigate liquefaction can appear to offer significant cost advantages over full depth treatment. In this case, it met its objective – a crust was maintained, even with liquefaction occurring within the stone column zone, and life safety achieved. Total and, therefore, potential differential settlement is larger than with full depth treatment, and will be more difficult to predict reliably. As a result, there is greater potential for total economic loss following a ULS event, particularly where no structural raft slab is provided.

Comparison of the measured pattern of settlement at Deans and Paul Kelly provides valuable insights into the benefits of a structural raft slab as part of a composite foundation system where liquefaction is expected. Such a slab more effectively confines ejecta than a flexible pavement, and can stiffen the structural response to differential settlement.

Deans Stand, with no structural raft, was subject to sagging and twisting in response to local stone column bulging under concentrated structural loads from the ground beams together with settlement resulting from different levels of gravity load and varying thicknesses of liquefied ground beneath its footprint.

Paul Kelly Stand, with a structural raft beneath the heavily loaded portion, remained essentially “straight” along both major axes and predominantly tilted backwards on the underlying liquefied ground in response to the differing gravity load across its footprint.

8 CONCLUSIONS

The performance of the Deans Stand at Lancaster Park in the February 2011 Christchurch earthquake has provided some valuable data and insights into the performance of stone column ground improvement that has been subjected to shaking levels considerably greater than design values.

While the overarching life-safety design intent of the ground improvement was achieved, the stone columns have been sufficiently heavily damaged that they cannot be relied upon during a future design event. This has had a significant effect on the economic viability of stadium repair. There appears to be merit in designing ground improvement measures that rely on densification to a higher level of earthquake shaking than the building they are supporting.

Elimination of a raft slab, and adoption of partial depth ground improvement, present capital cost savings that may be attractive in the early stages of a project. Both of these apparent cost savings can, however, impact on the economic viability of repairs following a large earthquake.

9 ACKNOWLEDGEMENTS

The authors gratefully acknowledge the permission of Christchurch City Council to publish this paper. Acknowledgement is also made of the significant technical contributions to the investigations by Earth Mechanics, Inc., California, and of review inputs from Tonkin and Taylor, the ground improvement designers, in the course of those investigations. The insightful review comments on earlier drafts of this paper by Dr CY Chin of Beca are acknowledged, with thanks. Finally, the support of the NZ Geotechnical Society in relation to presentation of this paper is appreciated.

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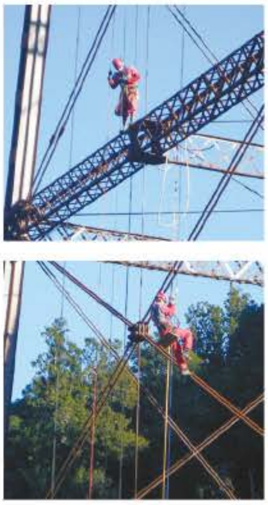

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Application of the Good Cheap Fast Rule in Geotechnical Engineering – David Lorier-May

ABSTRACT

There is a commonly known golden rule regarding services to clients under the title Good- Cheap-Fast. The rule is that services provided can ultimately comprise two of the options at any one time. For example, if the client wants the service good and cheap it won't be fast and vice versa. This rule can also be applied to geotechnical investigations.

The main purpose of geotechnical investigations regardless is to obtain and provide data sufficient to characterise site soil conditions and to provide parameters for design. A geotechnical investigation will also reduce uncertainty about ground conditions and reduce risks associated with foundation performance and cost variations in construction for a client.

When the client's expectation of cost of the geotechnical investigations lead to the reduction in scope of the investigations, it can potentially lead to uncertainty in ground conditions and increased risk of unforeseen ground conditions, which in turn could potentially lead to unsatisfactory foundation performance and/or increased cost in construction and future repairs.

This paper discusses the challenges faced with small to medium scale site investigations in the wider Auckland area regarding the tensions between client expectation of costs of ground investigation and what is considered by the geotechnical profession as good practice. It also discusses common issues facing geotechnical professionals during the entire phase of the project between the initial site investigation through to construction based on the author's local experience of projects within the Auckland region in the last 5 years.

1 INTRODUCTION

A geotechnical investigation is the assessment of ground conditions within a development and surrounding land. A geotechnical investigation and the understanding of ground conditions on a development site is an essential part of any project (MBIE 2016) whether it is the construction of large scale commercial buildings, subdivisions, public transport networks, public services or small scale private dwellings and other structures. In fact, the largest risk to a project often lies in the ground (GBICE 1991).

The two main purposes of geotechnical investigations are to identify geotechnical conditions of the ground on a site to provide geotechnical data for analysis and design and to reduce uncertainty about ground conditions to reduce and manage the risk of significant variations in the cost of a project (MBIE 2016, AGS 2006, CGIS 2016).

What is considered good practice for geotechnical services for a development project is summarised in Figure 1.1. It outlines the ideal sequence of events in an investigation and the monitoring and certification that follows.

The ideal service would begin with a study of the proposed development plans and a desk top study of existing information about the site and the surrounding land. A basic geological model of the ground conditions would then be developed and a scope of work together with a cost and time estimate would be formulated.

David Lorier-May

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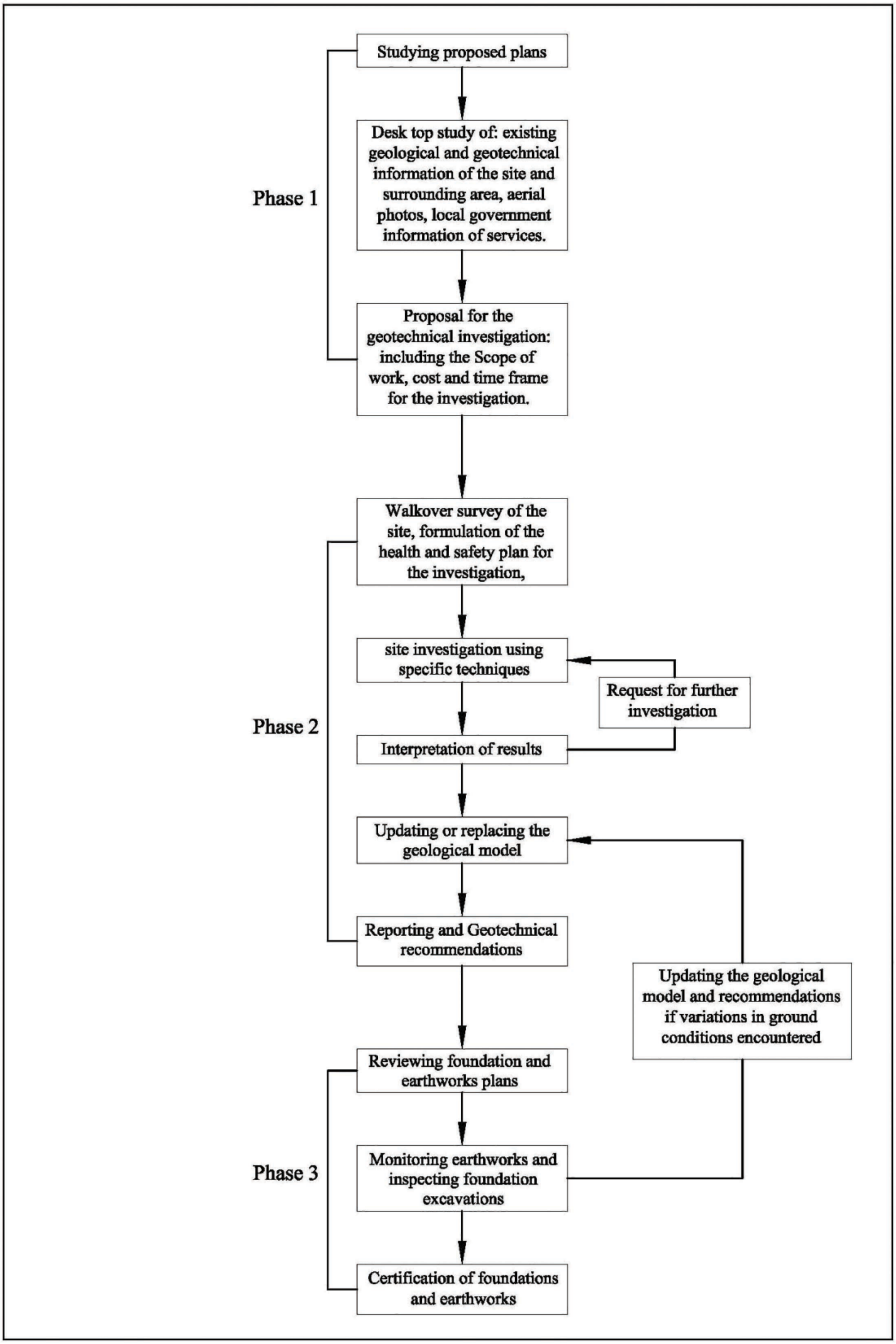

Figure 1.1: Flow Chart of ideal geotechnical service for projects

Once the proposal is accepted then the investigation would commence. The results of the investigation would either confirm the original geological model or lead to the development of a new model. Geotechnical recommendations for the proposed development would then be given in the geotechnical report. Although not essential, if the same geotechnical professional is included through to the construction phase of the project they would also review the final plans to be submitted for building consent to ensure that the plans are in accordance with the recommendations made in the report. The geotechnical professional would then go on to monitor earthworks and construction to confirm that the original geological model is valid. During the construction stage changes to the model and geotechnical recommendations may be made if ground conditions encountered during excavation and construction turn out to be different than what was initially anticipated.

The risk of issues occurring during construction due to unforeseen ground conditions depend on the quality and scope of the initial site investigation and whether the recommendations made in the geotechnical report are followed by the developer and contractor. As will be shown in the following sections below best practice above may not always be followed for a variety of reasons.

2 UNDERINVESTIMENT IN INVESTIGATIONS

Even though site investigation costs are a very small proportion of the overall cost of a site development (Clayton et al 1982), tensions between geotechnical professionals wanting to provide adequate data for foundation design and the developer, builder or owner wanting to minimise cost early on in a project are very common (MBIE 2016). If unexpected and undesirable ground conditions are encountered, which have a large impact on the cost and success of a project, it is often because of underinvestment in geotechnical investigations of a site (CGIS 2016, MIBE 2016). This underinvestment could lead to inadequate site investigations (CBICE 1991) There is strong evidence of a direct link between underinvestment in geotechnical investigations and large overruns in project costs (Figure 2.1) (CGIS 2016, MBIE 2016, McDonald 1994). It must be noted that significant variations in claims by piling contractors arise more from poorly known ground conditions than most other causes (Clayton et al 1982).

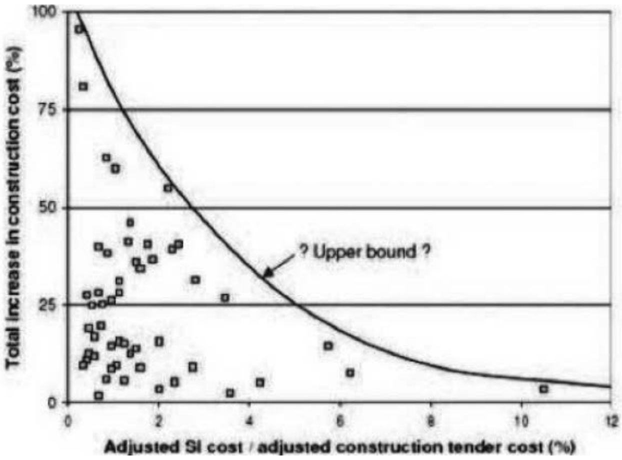

Figure 2.1: Impact of site investigation (SI) expenditure on UK highways contracts (MBIE 2016, McDonald et al 1994)

A possible cause of underinvestment in geotechnical investigations could be clients accepting proposals for investigations based on lowest price not best value or scope (CGIS 2016, MBIE 2016). This puts pressure on a Geotechnical Professional to push the scope of the investigation down in order that the job is won. Such situations can arise in a day to day practice. There is a possibility that these investigations would not achieve either purpose of an investigation, as stated above in the introduction. In contrast, cost savings in a project can be achieved if site investigations are carried out that have adequate scope to reduce uncertainty in the ground conditions (AGS 2006, MBIE 2016, CGIS 2016).

As well as underinvestment in geotechnical investigations, in some cases geotechnical reports have also been used for building consent applications that had not been issued for that purpose (Price et al 2015). It appears to be common to have contractors requesting site inspections of house construction where building consent have been issued and the only reports for the sites were reports that had been issued for the subdivision resource consent. In some situations, it was also found that in these reports there had been recommendations of further site-specific investigation that apparently were not picked up or not considered during the building consent application process. Usually when this occurs the recommended action is to insist on drilling additional boreholes on the site to confirm that the ground conditions within the specific site were as anticipated in the original report.

3 ISSUES DURING CONTRUCTION

The writer has worked as an Engineering Geologist for 14 years around the Auckland region. In that time, some projects have experienced problems during the earthworks and construction phase of a project (Figure 1.1) because the client, project manager or builder have tried to fast track to save time and to achieve savings. This has occurred by either not following specific recommendations provided for the work for a variety of reasons including; having not read or understood the recommendations, to finish the work quickly, they are unaware of their obligations due to inexperience, incompetence or a combination of the two, or they think the recommendations are unnecessary. In some situations, where no geotechnical investigation was carried out on a site the builder has noticed a potential problem with the ground and sought geotechnical advice and assistance.

The above situations have occurred on more occasions than they should and has often resulted in significant costs added to a project. Some typical examples are provided:

3.1 Case Example 1

Figure 3.1 shows an excavation up to 3m deep located within about 1m of the property boundary, which was excavated vertically without proper sequencing or temporary support. The geotechnical recommendations were to either excavate the cut to construct the wall in stages or use temporary support to keep the cuts stable. The contractor decided to ignore the recommendations or were unaware of them and they excavated the cut all at once without temporary support and without any monitoring and inspection by a geotechnical professional. When this was discovered they were strongly recommended to install temporary support for there was insufficient room to batter the slope back to a safe temporary angle. The temporary support they did install was completely inadequate nor properly designed.

Unfortunately, a large rain storm event occurred a few days later and as a result a significant part of the cut slope failed. Figure 3.2 shows the failure of the cut and the inadequate nature of the temporary support the contractor installed. Fortunately, the slip did not propagate to the boundary and a large temporary timber retaining wall was designed and installed by the structural engineer to stabilise the cut. However, this wall added significant cost to the project and caused a delay in construction.

3.2 Case Example 2

Figure 3.3 shows an excavation of up to 3.5m high where a slip occurred. Two geotechnical reports were written for this site prior to the earthworks being carried out. The second report, requested by Council, focused on the stability of the proposed cut and included a review of the structural engineer's excavation methodology. Part of the cut was located at the bottom of a relatively steep slope and it extended along two boundaries where there was insufficient room to batter the slope back. Geotechnical recommendations were for the contractor to initially cut the slope at 1V:1H, install the retaining wall poles and then excavate behind the poles and install the wall in sections of about 3m lengths. However, the earthworks contractor did not follow these recommendations and excavated the cut vertically and all at once. To make matters worse they did not inform the geotechnical professional nor the structural engineer that this was taking place. Unfortunately, a large storm event occurred causing part of the cut along the steepest part to fail. They attempted to place a temporary retaining wall along the part of the cut that failed. However, the regulators placed a stop work notice on the site and ordered them to place a temporary gravel toe bund as seen in Figures 3.3. At this point the geotechnical professional and the structural engineer who initially designed the retaining wall were reengaged.

A concrete pile retaining wall was constructed to retain the slip. As a result, what was initially going to be a 3.5m high timber pole retaining wall with 5m embedment

Figure 3.1: 3m deep unsupported cut near a boundary on an Auckland Property.

Figure 3.2: Slope failure on the cut slope after heavy rain.

Figure 3.3: 3.5m high cut along the boundary of a property supported by a gravel toe bund.

became a 4m high concrete pile wall with 10m embedment to retain the slip (Figure 3.4). The cost of this wall is estimated to be at least 4 times what the original cost would have been. This situation also resulted in significant delays in the project.

Figure 3.4: Concrete Pile Wall Installed to retain the slip.

3.3 Case Example 3

Figure 3.5 shows footings for a house addition and extension. This project had been given a building consent for construction without a geotechnical investigation. The site was relatively level and located within an area that published geological maps showed was underlain by basaltic tuff. It appears that a geotechnical investigation was considered unnecessary as a result. However, the project experienced a significant delay when the builder noticed a potential problem with the ground conditions at the bottom of the excavated footings. The builder then sought the advice and assistance of a geotechnical professional to determine the size of the problem. Hand augers drilled on site indicated low strength volcanic ash soils over the upper 1.5m of the ground. As a result of this site investigation the shallow foundations had to be

Figure 3.5: Footings for a house extension where low strength soils were encountered.

redesigned taking into consideration the lower strength soils using a reduced bearing pressure.

3.4 Case Example 4

In contrast to the above examples positive outcomes can occur if the client/developer gives due consideration to carry out site investigations that have adequate scope to reduce the uncertainty in ground conditions. One site investigation for a large warehouse, that had large floor and foundation loads on alluvial soils, required hand augers and machine drilled boreholes to assess both the shallow and deeper soils. This information was sufficient to provide adequate data on ground conditions to estimate settlement. During the detailed design process, additional CPTs were requested by the developer to better understand the consistency and variability of the ground conditions for the design of piled foundations.

4 CONCLUSION

The problems encountered during construction are caused either by fast tracking during construction, not following the specific geotechnical recommendations, by reducing the scope of investigations due to pressure to save on cost and time or by not carrying out a geotechnical investigation at all.

Contractors may face pressure by clients to keep cost down so they take on more risk by either not following recommendations made by geotechnical professionals or by not communicating their intentions with the geotechnical professionals. As the rule implied by the title above often they try and carry out a project cheap and fast. However, in many cases the end-product turns out not to be good or it is significantly more expensive than was originally planned. Local authorities may also be under pressure to process consents quickly to speed up the building process and either due to lack of resources or inexperienced staff are sometimes issuing consents based on inadequate of inappropriate geotechnical reports. There are concerns that the pressure to build houses faster in the Auckland region to meet the demand of a rapidly growing population is leading to an increase in problems caused by the above issues. It is obvious that houses need to be built faster to meet demand but projects should also be carried out properly with due consideration to best practice to reduce the risk of construction delays, budget blowouts and adverse effects on neighbouring properties.

It is essential that geotechnical professionals provide a service based on good practice. It may be necessary for the geotechnical professional to educate clients in what is considered best practice and the reason behind it. They should also endeavour to maintain their professional integrity by resisting the pressure to reduce the scope of investigations to win jobs based on the lowest price. Clients and developers need to be educated that awarding jobs based on lowest price will not necessary lead to cost saving. It is also highly recommended that clients are encouraged to use the geotechnical professional used for the initial site investigation throughout the entire life of the project to ensure that best practice is achieved and continuity of service.

Local Authorities should ensure that the appropriate geotechnical reports are used in building consent applications and that proper consideration has been given by the developer in all recommendations made by the geotechnical professional.

Clients, developers, earthworks contractors and builders should also be aware of their obligations under the resource and building consents. They should be aware that communication with geotechnical professionals is essential. They should also be educated in the risks involved and implications of not following through with geotechnical recommendations. Developers should also be careful in what contractors they use for a project. The cheapest contractor will not often be the best and may prove costlier in the long run. The geotechnical profession should also communicate better with developers and builders that trying to save money in the initial site investigation stages of a project runs a significant risk of increased costs and time delays further along in the process. The fact should be promoted that significant cost savings can be made in the project if site investigations have adequate scope like in example 4 above to reduce the uncertainty in ground conditions and good practice is carried out through an entire project. In turn, it should be emphasised that a "good" work may not be cheap and a "cheap" work may not be wholly successful.

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AUSTRALIAN GEOMECHANICS SOCIETY

NEW ZEALAND GEOTECHNICAL SOCIETY INC

12TH AUSTRALIA & NEW ZEALAND YOUNG GEOTECHNICAL PROFESSIONALS CONFERENCE

7-9 NOVEMBER 2018 HOBART

CALL FOR ABSTRACTS AND NOMINATIONS

The Australian Geomechanics Society and the New Zealand Geotechnical Society invite you to attend the 12th Young Geotechnical Professionals Conference (12YGPC Hobart)

The 12YGPC is for geotechnical professionals from Australia and New Zealand 35 years old and younger. It is designed for all attendees to present a technical paper on any topic of interest / experience relating to the field of geomechanics or geotechnical engineering.

Call for abstracts and nominations

Please complete the nomination form available on AGS website to this call for abstracts. Nominations of delegates must also be supported by a senior mentor and include an abstract of 200 words on a topic that is related to geotechnical practice or research.

Successful nominations will be selected based on the quality and relevance of the abstract. Positions are limited to approximately 50 attendees and all successful nominations will be expected to present their technical paper at the conference. The Don Douglas Youth Fellowship Award and the Young Geotechnical Professional Fellowship will be awarded to the best Australian paper and the best New Zealand paper, respectively during the conference. Top papers are to be published in a special issue of *Australian Geomechanics*.

Cost

Registration cost will be confirmed soon. It is anticipated that the registration cost will include three nights' accommodation at the Woolstore Hotel near the waterfront in Hobart, one conference banquet, one dinner, an arrival drinks reception, conference venue and a field trip to discover the engineering geology of the Hobart region.

IMPORTANT DATES

15 FEB 2018
Nominations & abstracts due

30 MAR 2018
Notice of acceptance

30 JUN 2018
Full paper due

14 SEP 2018
Registrations close

FURTHER INFORMATION

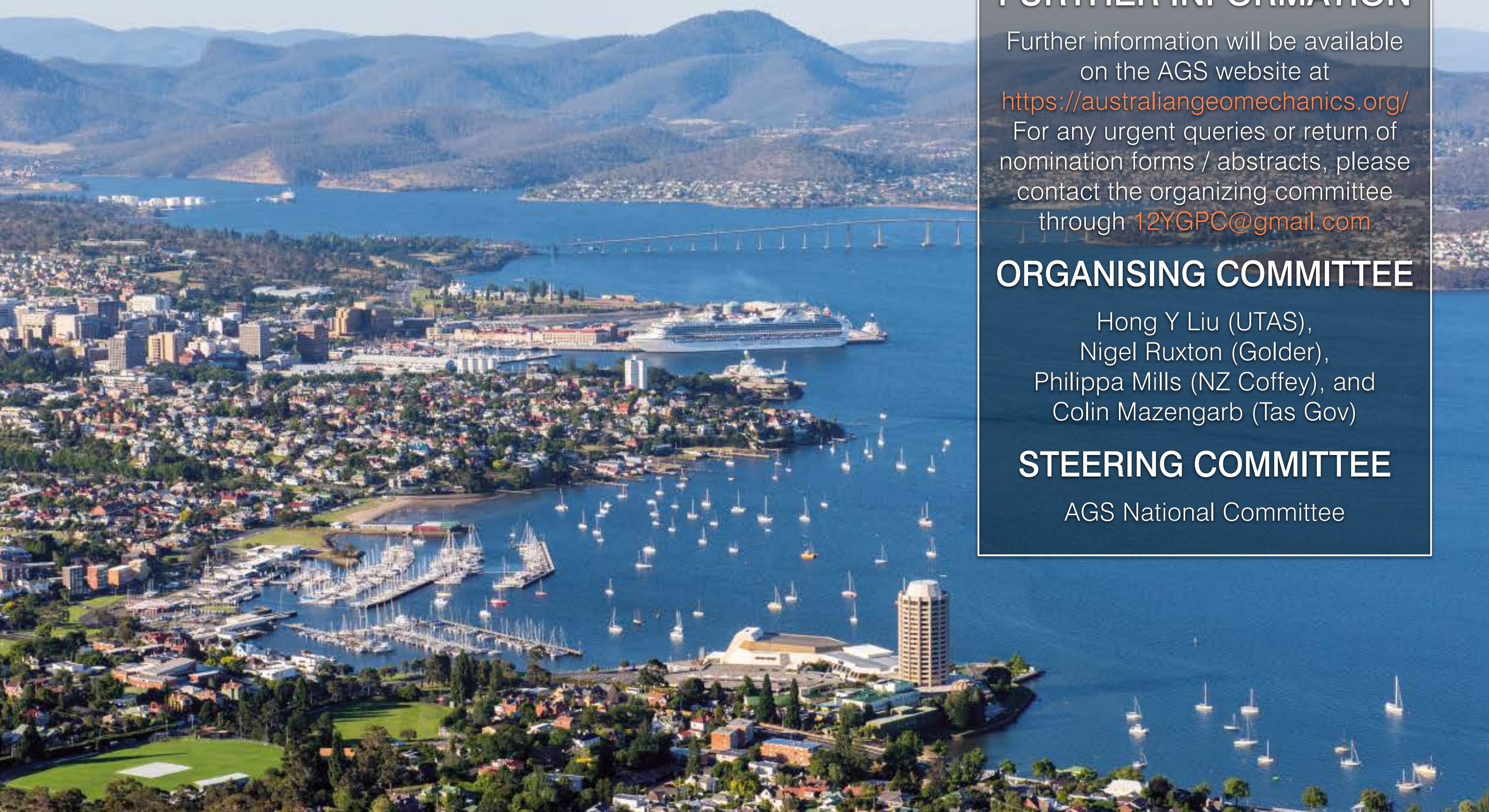
Further information will be available on the AGS website at https://australiangeomechanics.org/ For any urgent queries or return of nomination forms / abstracts, please contact the organizing committee through 12YGPC@gmail.com

ORGANISING COMMITTEE

Hong Y Liu (UTAS), Nigel Ruxton (Golder), Philippa Mills (NZ Coffey), and Colin Mazengarb (Tas Gov)

STEERING COMMITTEE

AGS National Committee



iYGEC6 Conference Report

THE 6TH INTERNATIONAL YOUNG Geotechnical Engineers' Conference was held in September, on the weekend preceding the 19ICSMGE in Seoul. It was attended by 107 delegates from 52 countries, including two delegates from New Zealand – myself and Pip Mills.

All delegates were required to deliver a 10 minute presentation, with topics ranging from "Friction tests on clayey soils for fast and bouncy cricket pitches in Sri Lanka" to "The shrinkage of invading CO_2 phase by capillary end effect in the drainage modelling for geological microfluidic chip geometry by lattice Boltzmann method". I'll admit to following the first slightly more readily.

The presentations gave an insight to the "flavour" of geotechnical engineering in the different regions. The Swedes use lots of timber poles, as we do in NZ, with their contractors buying forests directly adjacent to road embankments over soft ground and driving the poles untreated. The South Africans seem to have a pragmatic approach, not dissimilar to our own. The Ghanaians were looking at use of coconut fibres in road construction. The Danish are not so interested in landslides, their highest point having an elevation of 171m, but they are experts in offshore wind turbine foundations.

Of the two papers I found most interesting, one detailed centrifuge testing of a scale soil nail model to assess seismic performance. The testing visually demonstrated how closer spacing and smaller inclination is more effective. The second paper was on modelling of soil suction and groundwater pressure based on actual rainfall for stability analysis. This was a good reminder of the ever present, but not always considered, realm of unsaturated soil mechanics.

My highlight of the conference was, however, the chance to mix and mingle with people from around the world. The outgoing ISSMGE president strongly asserted that we were in an enjoyable profession of good people. My time at the conference reinforced this. This was equally the same at the ICSMGE where the keynote speakers and other well recognised names were friendly and willing to have a good discussion.

Pip provides her highlights as follow:

"iYGEC6 was an intense yet thoroughly enjoyable two days of discussion about our mutual love of soil and rock behaviour. As David mentioned, the scope of presentation topics varied hugely, making it easier to concentrate for the full day. Highlights for me included meeting a fellow halloysite enthusiast, delicious Korean BBQ, fangirling with the legendary engineer Idriss, and mingling with so many different cultures. Between talks, we explored the bustling metropolis of Seoul – 12 million in one city, just a slight step up for this Waikato country girl! This included visits to the Royal Palace, the North Korean Border, hiking in the local national park, many a Korean BBQ, and exploring the nightlife, where I discovered that the Belgians have a certain love for Celine Dion classics at karaoke. Many thanks to NZGS for the unforgettable experience."

Myself and Pip were selected to attend 6iYGEC during the ANZ Young Geotechnical Professionals' Conferences (YGPC) in 2014 and 2016 respectively. NZGS provided an award to contribute towards our attendance and we thank NZGS for this. We also encourage all young NZGS members to keep eye out for the next ANZ YGPC in Tasmania, Australia in 2018. Attending will be a rewarding experience and it is a great place to deliver your first technical paper.

Reported by *Dave Buxton*

19th International Conference on Soil Mechanics and Geotechnical Engineering, Seoul

THE 19TH ICSMGE was held in Seoul over four days in September 2017, and was attended by over 1800 delegates. After a rousing "welcome" by a number of Korean "big" drums, the Conference was opened by the former Secretary General of the United Nations, Ban Ki Moon. The first two days comprised plenary sessions. Most of those combined sessions involved the presentation of "Honour Lectures" covering the many different sub-fields of soil mechanics and geotechnical engineering. The particular highlights for me were the lectures delivered by Peter Day (the Terzaghi Oration, on Geotechnical Engineering Practice in Developing Countries), Jon Bray (the Ishihara Lecture, Simplified Procedure for Estimating Liquefaction-induced Building Settlements), Chris Haberfield (the Gregory Tschebotarioff Lecture, Practical Application of Soil Structure Interaction Analysis) and Buddhima Indraratna (the Louis Menard Lecture, on Recent Advances in Vertical Drains and Vacuum Preloading). There was a lot of other interesting ground covered by the honours and other lectures delivered during the first two days.

Above: Official opening by Ban Ki Moon

Formal awards were also presented during the plenary session, with "our" Outstanding Member Society award being passed to the British Geotechnical Association. We're clearly in good company! It was also pleasing to see the Kevin Nash Gold Medal awarded to Professor Antonio Gens. The Kevin Nash Gold Medal is awarded in memory of Professor Kevin Nash, Secretary General of the International Society (1965-1981). The medal is awarded to a person who, through distinction as an engineer, through international contributions to engineering practice and education, through contributions to international good will, and through service to the International Society, has made a major contribution to fostering the ideals and goals of the International Society for Soil Mechanics and Geotechnical Engineering throughout the world. Professor Gens has visited NZ several times, and thoroughly deserves this recognition.

The Australian Geomechanics Society generously hosted a dinner for the outgoing and incoming Boards on the first night of the Conference. They were rightly pleased to have secured the hosting rights for the next conference, and certainly showed their appreciation. Their hospitality was faultless. It was a great chance for me to meet some of my fellow (incoming) Board members for the first time. A Gala Dinner was held on the evening of the second day, and introduced many of us to traditional Korean music. While photos cannot do justice to the multiple dancers each playing five drums, it was certainly an interesting cultural experience.

The third and fourth days of the Conference were given over to parallel sessions, each run by the different Technical Committees. Up to thirteen parallel sessions were held, varying between paper presentations, discussion sessions and workshops. While it was easy to find something of interest and value, it was difficult to choose the best session to attend. As with the plenary sessions, a lot of ground was covered! While it's not really fair to identify highlights from the inevitably small sample of sessions I was able to attend, work that Ross Boulanger has done on dam stabilisation

Above: An Immediate Past Chair contemplating dinner, Gangnam Style

using shear walls (TC211) and a long running study by Stephen Buttling on the settlement of and load sharing beneath a high rise building in Bangkok (TC205) struck me as particularly interesting and valuable.

Breaks were held in the exhibition and poster area, with a wide range of suppliers showing off their products. Most interesting for me were the "industrial strength" drones being used by some of the Korean organisations. One was even set up for airborne geophysics, operating 2-3m off the ground.

I encourage you to find the conference proceedings online and have a browse. There is plenty of value in there, although it may take a bit of effort to find your way to it!

Lastly, as has been common at these events, the NZGS was poorly represented. We were able to give a significant proportion of our allocated pages to the AGS. With the next ICSMGE being held next door in 2021, we should try and do better. It really is worth the effort to find out what's going on in research and practice elsewhere in the world.

I was fortunate enough to be supported by the NZGS and Beca to attend this conference, and am truly grateful for the opportunity.

Gavin Alexander
Former immediate Past Chair,
current Australasia VP for ISSMGE

A Review of the Terzaghi Oration, and the Isihara Lecture presented to the 20th International Conference for Soil Mechanics and Geotechnical Engineering, Seoul, September 2017

Charlie Price
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, as well as extensive experience in New Zealand.

TERZAGHI ORATION
Peter Day, University of Stellenbosch, South Africa; and consultant (Jones, Wagener, Pty Ltd), South Africa:

Challenges and shortcomings in geotechnical engineering practice in the context of a developing country

Peter Day, a South African consulting engineer and academic, presented the Terzaghi Oration, an excellent paper focussing on the relationship between research and practice in geotechnical engineering, and in particular on the difficulty over transfer of information between the two.

Peter's Terzaghi Oration focussed on the gap between research and practice. Although our ability to understand and evaluate ground behaviour has improved significantly over the years, the number of publications has increased exponentially to the extent that we are unable to translate all the findings into practice. As a result geotechnical failures tend to be the result of our inability to apply available knowledge in practice, or simply of not recognising critical design situations, rather than a lack of knowledge. The consequence of this is that the profession needs to shift its focus from the creation of knowledge to facilitating its implementation.

A Critical Review of Knowledge Development and Implementation in Geotechnical Engineering

The state of practice and the uptake of research findings is lagging further and further behind the state of the art in geotechnical engineering (particularly in developing countries). This manifests itself in many ways ranging from failure of the most basic geotechnical works to ignorance of recent developments by many practitioners.

Examples were given of four projects in South Africa where failures or unexpected geotechnical issues were encountered, causing escalations in costs. These encompassed well known issues on expansive soils, a slip caused by unfavourably dipping shale beds and inadequate site investigation resulting from programme pressure. Each failure could easily have been foreseen as the knowledge was available, but for one reason or another was not put in to practice.

The proliferation of technical publications is driven by researchers who may not have worked in practice themselves, and there appears to be a preference for research into analytical procedures and laboratory testing as opposed to field experiments, construction methods or engineering practice. The result is an increase in research findings that are not being applied in practice. In addition the rate at which research is being produced far exceeds the rate at which the findings are being incorporated into geotechnical practice. The benefit of geotechnical research can only be realised when it is applied in practice, and for this to happen it needs to be converted into documents that practitioners are able to use and are sufficiently authoritative to rely on. CIRIA, BRE, FHA and GEO are given as good examples of establishments that produce authoritative guidelines extensively used in practice.

Established practitioners are likely

to stay with what is familiar rather than embrace new ideas, but the CPD system forces a degree of exposure to new knowledge that the average professional may not otherwise have experienced. However, the uptake and application of new knowledge is dependent on it providing some advantage, and in a cost driven environment it is not necessarily recognised that good engineering provides good value, although that applies less to the larger projects.

Development of Codes and Standards

In the past South Africa has adopted many British standards, but also had regard to North American and Australian standards, the latter due to similarities in climatic conditions. It is currently evaluating options for the development of a local geotechnical design standard, which has led to critical examination of the purposes of such codes. The main purpose is of course that of standardisation, but benefits also include the setting of the norms for the profession, and that they represent a distillation of existing knowledge on which there is consensus. In this sense, codes and standards lag behind the introduction of new developments in the industry.

Design methods

In many countries the method of design is prescribed by national standards, but countries like South Africa, which do not have a geotechnical design code, face a choice of which design methods to adopt. The four main geotechnical design methods that have been considered by South Africa are working stress design, limit states design, reliability based design and mobilised strength design. A comparative study of these has been undertaken as part of the assessment being carried out on the suitability of reliability based design for routine design purposes and possible incorporation into a South African geotechnical design code. This comparative study was based on the existing Eurocode-compliant solutions and assessed a very limited range of design conditions, but significant conclusions included:

(i) in limit state design reliability indices are reasonably constant for all the types of structures considered;

(ii) global factors of safety vary significantly across the range of problems and material properties considered, supporting the well-established realisation that Factor of Safety is a poor measure of the reliability of a structure;

(iii)the global factors of safety obtained using characteristic values of material properties are closer to accepted values while those obtained using mean values are significantly higher. This supports the view that there is no significant difference between characteristic values and the "responsibly-conservative" values typically used in working stress design;

(iv) a working stress design prepared to achieve the minimum acceptable Factor of Safety using mean parameter values will have an unacceptable reliability index. This situation becomes worse as f' increases;

(v) Reliability analysis is a valuable tool to be used in conjunction with limit states design procedures. The comparisons undertaken suggest that our current limit states design procedures are robust and not far off the mark.

The agreement between the results of limit states design and reliability-based design in the case of simple every-day geotechnical problems is encouraging. The development of simple methods of undertaking reliability-based design calculations opens the way for their use in practice, particularly for problems with explicit solutions. It is clear that working stress design methods are flawed and the use of such methods should be discouraged.

For countries that do not have geotechnical design codes of their own, the adoption of limit states design codes from developed countries remains a good option. Reliability based design can be introduced to complement limit states design. However, the adoption of reliability-based design as the "default" or "preferred" method could present problems with the potential for misuse because of its complexity or abuse on account of the flexibility of the method and the dependence of the outcome on the input parameters.

Geotechnical Data

One of the main challenges in developing countries is the poor quality of laboratory test results. Peter presented data from studies comparing the results of Atterberg Limit, specific gravity and PSD tests carried out in four accredited laboratories in South Africa. The differences between the results are such that they could seriously affect the classification of the material. In tests on five samples of clayey material the plasticity indices from the control tests were 29% to 75% higher than from commercial laboratories. In conclusion, there is no doubt that poor quality data, in conjunction with inadequate site investigations, and the types of test on offer is hampering the uptake on new technologies.

Conclusions

The problem of inadequate site investigation has been with us for years and is unlikely to disappear soon, and competitive bidding of geotechnical work is here to stay. The best that the profession can do is make it as simple as possible for those issuing bids to correctly specify the type and scope of investigation.

The quality of geotechnical data received from testing laboratories remains a concern. It may be necessary to create a new standard for the testing of fine-grained soils distinct from the standards used for road-building materials.

Limit states design is a step forwards from working stress design. Reliability-based design is an ideal complement to limit states design.

There is a concern that the way we teach geotechnical engineering and the way current codes are formulated tends to focus more on design situations involving rupture of the ground than its deformation.

There is a break in the cycle of new knowledge transfer back and forth between researchers/academics and practitioners. This communication is the responsibility of both parties.

The Ishihara Lecture

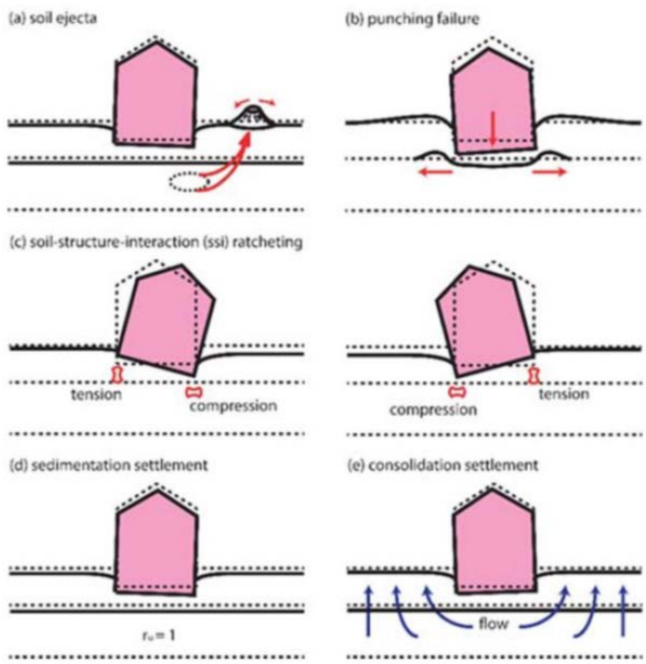
Jonathan Bray, University of California, Berkeley, California, USA

A SIMPLIFIED PROCEDURE FOR ESTIMATING LIQUEFACTION-INDUCED BUILDING SETTLEMENT

The state-of-the practice for estimating liquefaction-induced building settlement still largely involves using empirical procedures developed to calculate post-liquefaction, one-dimensional, reconsolidation settlement in the free-field away from buildings, and neglects the importance of

other mechanisms that contribute to building settlement. These free-field analyses cannot possibly capture shear-induced deformations in the soil beneath shallow foundations.

It is useful to categorize movements as ejecta-induced, shear-induced, or volumetric-induced deformations. Observations clearly show that buildings supported by shallow foundations displace downward more than the 1D volumetric reconsolidation liquefaction-induced settlement for the free-field, level ground case. Shear-induced mechanisms should be considered in procedures estimating liquefaction-induced building settlement. When it occurs, ejecta-induced deformation can govern building settlement. The resulting impacts can be devastating and lead to large settlement. The calculated post-liquefaction bearing capacity factor of safety (FS) is also an important index of seismic performance for buildings with shallow foundations situated on liquefiable soils.



The seismic performance of a site should be more influenced by the characteristics of the ground motions at the ground surface than those for the outcropping "rock" site condition. Therefore, the ground motion intensity parameters corresponding to the surface ground motions are preferred in the proposed simplified procedure for estimating liquefaction-induced building settlement.

Some 1308 non-linear effective stress soil-structure interaction dynamic analyses were performed to identify key parameters controlling settlement for this project.

The proposed simplified procedure for estimating liquefaction-induced building settlement involves these steps:

1. Perform a liquefaction triggering assessment, and calculate the safety factor against liquefaction triggering (FSL) for each potentially liquefiable soil layer.
2. Calculate the post-liquefaction bearing capacity factor of safety (FS) using the simplified two-layer solution of Meyerhof and Hanna (1978). If the post-liquefaction bearing capacity FS is less than 1.0 for light or low buildings or less than 1.5 for heavy or tall buildings, large movements are possible, and the potential seismic building performance can generally be judged to be unsatisfactory.
3. Estimate the likelihood of sediment ejecta developing at the site by using ground failure indices such as LSN, LPI, or the Ishihara (1985) ground failure design chart. If the amount of sediment ejecta is significant, estimate the amount of building settlement as a direct result of loss of ground due to the formation of sediment ejecta.
4. Estimate the amount of volumetric-induced building settlement.
5. Estimate the shear-induced building settlement (Ds) due to liquefaction below the building according to:

$$Ln(Ds) = c1 + 4.59*\mathrm{Ln}(Q) - 0.42*\mathrm{Ln}(Q)^2 + c2*\mathrm{LBS} + 0.58*\mathrm{Ln}(\mathrm{Tanh}(HL/6)) - 0.02*B + 0.84*\mathrm{Ln}(\mathrm{CAVdp}) + 0.41*\mathrm{Ln}(Sa) + \varepsilon$$

See the full paper for definition of terms in the equation and further explanation.

6. Estimate the total liquefaction-induced building settlement as:

$$D_{total} = D_{ejecta} + D_{volumetric\text{-}induced} + D_{shear\ induced}$$

7. Use engineering judgment.

Some important limitations of the procedure which should be considered are:

- The equation in 5 above was developed using a subset of potential building configurations and earthquake ground motions. The structures considered are regular (e.g., uniformly loaded) and have heights no greater than 24 m.
- The non-liquefiable crust does not have defects (e.g., utility trenches that could provide preferential paths for ejecta).
- Some volumetric induced liquefaction building settlement occurs during strong shaking, but this procedure categorizes all of this settlement as being due to the shear-induced mechanism, which is conservative.
- Case histories and previous experience are important to consider in developing the final engineering assessment of this complex problem.
- For important projects, perform nonlinear dynamic SSI effective stress analyses.

The performance of this methodology was tested for several well-documented case histories from the Kocaeli and Canterbury earthquakes.

Report on NZGS "Principles and Practice of Engineering Geology" Courses 2017

NZGS ORGANISED AND carried out in late August – early September 2017 a successful two-day short course on "Principles and Practice of Engineering Geology". The course was held in the three main centres, Auckland, Wellington and Christchurch and was very well attended. It was the first time an engineering geology focused course was organized by NZGS.

Above: Class based Day 1 in GNS Science in Lower Hutt, Wellington

COURSE OBJECTIVES

The course aimed to be practical and relevant to the day to day activities of engineering geology professionals. The objectives of the course included:

To provide an overview of the basic principles underpinning the practical application of engineering geology, including the geoscientific methodologies and practices required for successful interpretation of the ground conditions.

To achieve a greater understanding of "how to get the geology right" in the context of various ground engineering projects and of what constitutes appropriate engineering geology deliverables in the investigation, design and construction phases of those projects.

To develop communication skills of engineering geologists to other designer disciplines and industry clients, regarding engineering geology issues and their influence on projects.

The course was ideally intended for engineering geologists with three to six years' experience, in particular those preparing for Professional Engineering Geologist (PEngGeol) registration, in the sense of understanding the skills required to be demonstrated by candidates when applying for assessment. Practitioners with different experience levels would also benefit from the course, less experienced by getting good guidance and developing confidence on how to apply theoretical knowledge into real life projects, and more experienced as a refresher.

PRESENTERS

The best of the best from the local and international industry joined their efforts to organise and deliver this course. The main presenters, Fred Baynes, Stuart Read and Ann Williams, distinguished engineering geologists based in Australia and New Zealand, brought to the course more than 100 years' professional experience altogether, in the local and international industry.

Fred in particular, conveyed his experience and expertise in organising and delivering similar courses for the Australian Geomechanics Society, which was valuable, given that there was no previous experience within NZGS in the organisation of similar courses for professionals.

The main presenters were assisted by renowned experts in each centre who organised and carried out the field exercises. These were Warwick Prebble, Susan Tisley and Ross Roberts in Auckland, Ian Brown and Nick Perrin in Wellington and Don Macfarlane, Barry McDowell, Rori Green and Scott Barnard in Christchurch.

COURSE CONTENT

The first day of the course included class based lectures and practical exercises, consistent for all three centres. The lectures covered a wide range of engineering geology topics, starting from refresh of basic knowledge, such as principles of geological data collection, development of geological maps and sections, use of structure contours, structural geology and stereographic projection.

Other subjects covered during the first day were ground observations and interpretation in projects, ground models, site characterisation and landslide recognition. The practical exercises asked the attendees to practice in basic and advanced geoscience skills. The attendees worked in groups to interpret the structural features of a geological map and develop geological cross

Top left: Auckland course Day 2 field exercise at Leigh – Pakiri Beach. **Above:** Mapping of rock fall hazard at Sumnervale, Christchurch. **Left:** Fred Baynes and Stuart Read teaching structural geology at Owhiro Bay

sections and to develop complicated ground models by studying a range of available background information.

The second day consisted of field exercises to enhance class based knowledge and practice observational skills and engineering geology field techniques, which included a more local geology component in each centre. The field day in Auckland was at Leigh – Pakiri and Warkworth, and covered study of landforms, mapping of geomorphic and coastal erosion features, logging of rock exposures, structural geology mapping and observation of landslides, in rocks of Pakiri and Waipapa group formations.

In Wellington the group visited Owhiro Bay, where they carried out logging of quarry slopes in Wellington Greywacke rocks and Seatoun, for landslide recognition in the field based on geological and geomorphological features.

In Christchurch, the field day was at Sumnervale and Avonside, for mapping of cliff exposures, study and assessment of rock fall hazard in Lyttelton volcanics and study of liquefaction land forms in liquefiable Quaternary alluvial deposits.

INDUSTRY RESPONSE AND FEEDBACK

The course attracted the interest of professionals following its announcement, with the spaces filling up very quickly, especially in Auckland. We had to exceed our initial maximum accepted number of 20 attendees in all centres, to 25 in Auckland, 23 in Wellington and 22 in Christchurch, to satisfy the high demand.

Most of the attendees spent time to provide detailed feedback following the course, which indicates their level of engagement and interest. The course received positive feedback comments in general, but also constructive criticism and suggestions for additions and improvement, which will help us optimise future courses.

Most of the positive feedback was about the content of the course, the value and relevance to practitioners' work, the practical exercises both in the class and field, the expertise and knowledge of presenters. Areas of improvement consist our registration processes, course notes and handouts and some technical issues, mainly with respect to the field exercises. The duration of the course with respect to the material covered was also discussed by attendees.

Prevailing suggestions by attendees include the addition of more practical exercises and case studies and of more time in the field.

Bespoke Innovation with Leveloggers

- The Solinst Levelogger is a water level and temperature recording device.
- Uses include ground water monitoring from pumping and slug tests; stream gauging; level monitoring to tidal fluctuation; and long term water level monitoring.
- High accuracy at 0.05% net full scale.
- 10 years battery life based on one reading/min.

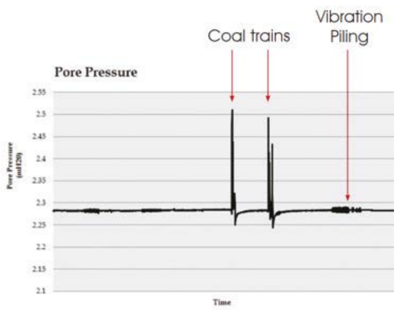
Liquefaction Monitoring

Monitoring liquefiable material in the Waikato River during vibration piling

- Construction of a new railway bridge.
- Leveloggers installed in sand pockets in the Waikato River and adjacent railway embankment.
- Preferred to vibrating wire piezometers because of required frequency of 0.5 second readings.
- Involved live, continuous monitoring of pore pressure during vibration piling embankment and train events, with software plotting every 0.5 seconds.

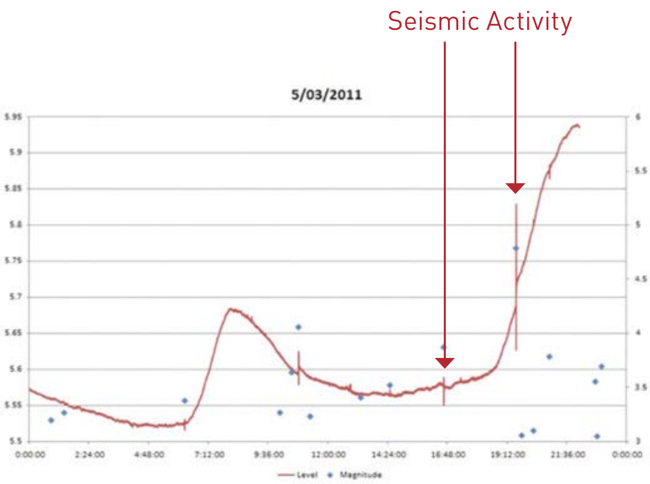
Interpretation of data:

- Results from monitoring showed a small amount of energy from the piling technique and a large amount of energy in the embankment from the coal train!



Seismic Activity Monitoring

Response of material to earthquake events in the Port Hills, Christchurch



- Located in a small estuarine area at the foot of the Port Hills, with a geology of coarse sands and gravels.
- Leveloggers installed at a depth of 6m below ground level, in a sand cell with bentonite seal in column.
- Ground water level at 2-4m below ground level.
- Leveloggers measured the effects of seismic activity on pore water pressure.
- Data points collected every 0.5 seconds.

Interpretation of data:

- The higher the magnitude of the aftershock, the higher the spike on the plot.
- Tidal and rainfall effects were also observed due to pressure increase.

Above: Don Macfarlane explaining Day 2 exercise on the way to Sumnervale field exercise in Christchurch

The inclusion of more challenging and advanced topics was also requested by some attendees, as well as of new technologies currently used in the engineering geology practice, such as UAV, modelling related software and applications.

FUTURE COURSES

Due to the fact that a number of practitioners in Auckland could not attend the Engineering Geology course this year, because of the high demand, NZGS intends to repeat it in 2018 to satisfy those people. We will soon call for registration of interest through the weekly newsletter and NZGS website, to examine the number of people interested for a repetition of the course next year in other centres as well, apart from Auckland.

The positive response to this course from the industry encourages NZGS to continue its efforts to organise more similar courses in the future. This particular course is intended to be repeated at regular two to three years' intervals, depending on ongoing interest, while more specialised engineering geology courses are examined, such as on soil & rock logging and landslides.

ACKNOWLEDGEMENTS

NZGS and the author personally would like to thank the presenters for devoting their time and efforts to make this course happen, the attendees for their enthusiastic response, participation and feedback and the course sponsors, BECA, GNS Science and Stantec for their contribution in Auckland, Wellington and Christchurch respectively.

Prepared by
Eleni Gkeli

AUSTRALIAN GEOMECHANICS SOCIETY

NEW ZEALAND GEOTECHNICAL SOCIETY INC

12TH AUSTRALIA & NEW ZEALAND YOUNG GEOTECHNICAL PROFESSIONALS CONFERENCE

7-9 NOVEMBER 2018 HOBART

CALL FOR ABSTRACTS AND NOMINATIONS

The Australian Geomechanics Society and the New Zealand Geotechnical Society invite you to attend the 12th Young Geotechnical Professionals Conference (12YGPC Hobart)

The 12YGPC is for geotechnical professionals from Australia and New Zealand 35 years old and younger. It is designed for all attendees to present a technical paper on any topic of interest / experience relating to the field of geomechanics or geotechnical engineering.

Call for abstracts and nominations

Please complete the nomination form available on AGS website to this call for abstracts. Nominations of delegates must also be supported by a senior mentor and include an abstract of 200 words on a topic that is related to geotechnical practice or research.

Successful nominations will be selected based on the quality and relevance of the abstract. Positions are limited to approximately 50 attendees and all successful nominations will be expected to present their technical paper at the conference. The Don Douglas Youth Fellowship Award and the Young Geotechnical Professional Fellowship will be awarded to the best Australian paper and the best New Zealand paper, respectively during the conference. Top papers are to be published in a special issue of *Australian Geomechanics*.

Cost

Registration cost will be confirmed soon. It is anticipated that the registration cost will include three nights' accommodation at the Woolstore Hotel near the waterfront in Hobart, one conference banquet, one dinner, an arrival drinks reception, conference venue and a field trip to discover the engineering geology of the Hobart region.

IMPORTANT DATES

15 FEB 2018
Nominations & abstracts due

30 MAR 2018
Notice of acceptance

30 JUN 2018
Full paper due

14 SEP 2018
Registrations close

FURTHER INFORMATION

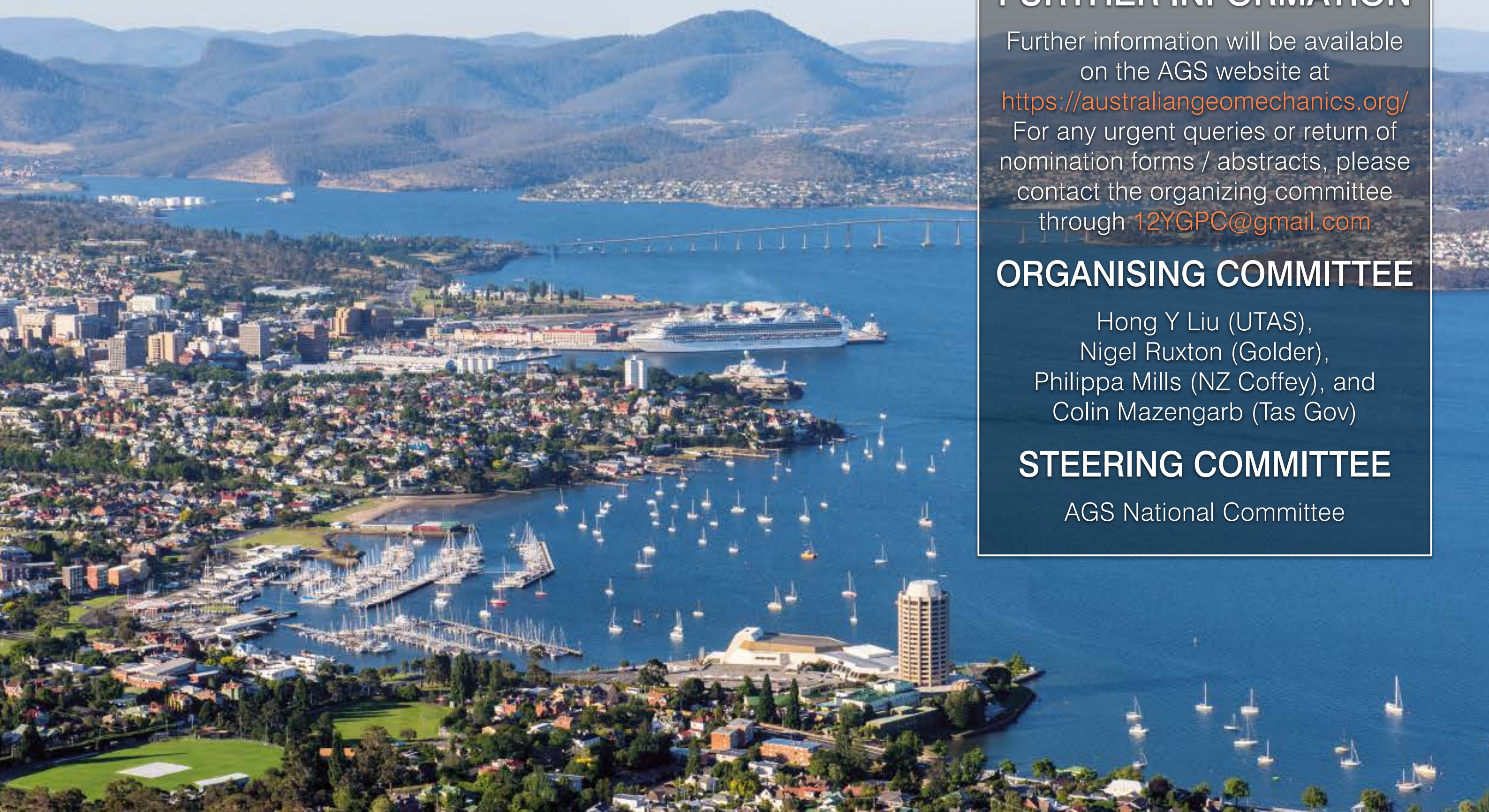
Further information will be available on the AGS website at https://australiangeomechanics.org/
For any urgent queries or return of nomination forms / abstracts, please contact the organizing committee through 12YGPC@gmail.com

ORGANISING COMMITTEE

Hong Y Liu (UTAS),
Nigel Ruxton (Golder),
Philippa Mills (NZ Coffey), and
Colin Mazengarb (Tas Gov)

STEERING COMMITTEE

AGS National Committee



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.

GEOMECHANICS AWARD 2018

The award is made to the Society member or members producing the adjudged "best" published paper during the three years ending 31 July preceding the date of the Award, in any publication at the discretion of the Management Committee. The winning paper will be that considered to be distinguished in its contribution to the development of geotechnics in New Zealand.

All Society members who are authors of any paper published within the previous three years shall be eligible, provided that at least one author is a member and a member nominates the paper in writing during the year of the award. The award is a sum of money to be determined by the Management Committee for the purchase of books, plus a certificate.

Previous winners were:

- 2015 Tam Larkin and Chris Van Houtte for their paper entitled “Determination of site period for NZS1170.5:2004” (published by NZ Society for Earthquake Engineering in March 2014)
- 2008 Misko Cubrinovski for his paper entitled “Pseudo-static analysis of piles subjected to lateral spreading”

Please send any nominations to **Sally Hargraves** on sally@tfel.co.nz

UPCOMING DATES

Student Poster Award

Registrations closed on 17th November 2017; Completed posters are due by the end of January 2018. The award will be determined in the following months with the top three posters published in the June issue of Geomechanics News and put on display at at least 2 or 3 local NZGS Branch Meetings.

Rocha medal 2019

Nominations for the 2019 award are currently open and will close on 31 December 2017 (for evaluation in 2018). The award recognises the most meritorious PhD thesis in rock mechanics, with further details on the ISRM website. Nominations should be sent directly to Secretary General (remembering a 10,000 word summary of the thesis needs to be prepared and ISRM membership demonstrated).

YGP Conference 2018

Abstracts due 15 February 2018 (see call for nominations in this issue and online). 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.

International Society for Soil Mechanics and Geotechnical Engineering

I took up my new role as Vice President for Australasia in September, at the closing ceremony of the 19th ICSMGE. I took over from Professor Mark Jaksa, and am grateful for his work and for the support that Professor Mick Pender provided as NZGS Local Liaison over the past four years. This is an exciting time for the ISSMGE in our region as we get ready to host the 20th ICSMGE in Sydney in 2021. I'm looking forward to working with the Australian Geomechanics Society and the Organising Committee to make this a huge success. Graham Scholey has led the pursuit of this conference for many years, and is the new AGS Local Liaison.

I'm also looking forward to increasing the profile of the ISSMGE amongst our members. The Society facilitates a lot of great work, which is delivered in a number of different ways. It's well worth looking into.

COUNCIL MEETING 17 SEPTEMBER SEOUL

The biennial ISSMGE Council Meeting was held in Seoul immediately before the 19thConference, with Charlie Price and I representing the NZGS. This sees many of the Member Societies represented, with the AGS particularly conspicuous! Key outcomes and messages from the Council Meeting were:

- The election of Professor Charles Ng as President for the period 2017 -2021
- Unanimous selection of Australia to host the 20th ICSMGE 2021. This will be in Sydney, and represents a great opportunity for NZGS members to attend a world class event
- Corporate Associate membership presents great networking and opportunities to influence the direction of the Society. Expect to hear more about this in the coming months
- Technical Committees require an increased focus on active participation – again something for NZ in particular to work on
- Make the most of the International Journal of Geo-Engineering Case Histories, both by referring to it and by submitting papers. It is open access, and freely available from the ISSMGE website
- The updated ISSMGE website now permits easier access to content. It includes 35 recorded webinars, and all past conference proceedings. In all, there are 10,000 papers available through the on-line library
- There is now an ISSMGE mobile platform – so there's no excuse not to keep up to date!

THE NEW BOARD

The incoming Board comprises:

President – Prof Charles Ng

Immediate Past President – Prof Roger Frank

Secretary General – Prof Neil Taylor

Vice President Africa – Dr Etienne Kana (Cameroon)

Vice President Asia – Prof Eun Chul Shin (Korea)

Vice President Australasia – Gavin Alexander (NZ)

Vice President Europe – Prof Mario Manassero (Italy)

Vice President North America – Prof Tim Newson (Canada)

Vice President South America – Prof Alejo Sfriso (Argentina)

Prof Mounir Boussida, Dr Kok Kwang Phoon and Prof Pedro Pinto have been appointed to the Board to assist in specific areas.

Leadership of many of the Board Level Committees has for the most part remained the same, with plans for some to change part way through the current four year cycle.

Gavin Alexander

Gavin has over 30 years' international experience in geotechnical engineering, with wide ranging involvement across the infrastructure, buildings and industrial sectors in New Zealand, Australia, much of Asia and the UK. He is a Senior Technical Director in Beca's Geotechnical Engineering group, where he has led single and multi-disciplinary teams on many large-scale, high profile and complex projects. He regularly undertakes independent peer reviews for other organisations. Like many NZ trained geotechnical engineers, Gavin's primary interest lies in earthquake geotechnical engineering. Piled foundations and heavy retaining walls are another area of interest, sparked by his time in the UK in the late 1980's and early 1990's. Gavin is a Fellow of Engineering New Zealand, and was Chair of the NZGS from 2013 to 2017. He took up the ISSMGE VP role on completion of his term as Immediate Past Chair of the NZGS.

VISION FOR THE COMING FOUR YEARS

Charles Ng has set out his overarching ten point plan to further improve the Society, and his message can be found in the October 2017 edition of the Bulletin (on the website, of course!). Charles' plan revolves around three primary platforms – education, innovation and diversity. Of particular interest to many of us will be the establishment of an online learning platform, improving the functionality and performance of the Technical Committees and increasing the recognition of young members. I'm certainly looking forward to helping achieve Charles' goals. In the meantime, though, I encourage all of you to make the most of what's already available.

NEW REPRESENTATIVES

I'm pleased to announce several new representatives from our region on ISSMGE groups. David Buxton (NZ) and Truong Hoang Minh and Daniel King (Australia) have joined the Young Members' Presidential Group chaired by Lucy Wu. Ioannis Antonopolous has been nominated to represent the NZGS on the Editorial Board of the Bulletin, and we are hopeful he will take up that role for the December edition. We are grateful for the interest received in these positions.

That's it for this month – don't forget to check out the ISSMGE website (and App!) and look out for more opportunities to get involved in your international society.

Gavin Alexander

International Society for Rock Mechanics

This report mainly covers ISRM-related information from the Board and Council meetings held on 30 September /01 October and 02 October 2017 respectively in association with the Afrirock 2017 conference in Cape Town, South Africa. 54 of the 62 ISRM National Groups attended the Council meeting, of which 51 are paid up and were able to vote.

ELECTION OF PRESIDENT FOR TERM 2019 -2023

The ISRM Board (President, Regional Vice-Presidents, Vice Presidents at Large, Secretary General) is elected for four year terms – currently 2015 to 2019. The ISRM has a policy of electing the President for the following term (2019 – 2023) two years ahead so that the elected candidate becomes a member of the current Board as President-elect. The intention of such an overlap is that the next term President becomes fully familiar with Society needs and Board protocols and leadership before formally taking up the Presidential role.

Resat Ulusay, currently Professor at Hacettepe University, Ankara, Turkey, was elected as president-elect.

SOCIETY NAME

The proposal from the Board for a name change to the "International Society for Rock Mechanics and Rock Engineering" was voted on by the Council and was successful.

With the successful vote, a second vote was held for the acrynom to become ISRMRE and was unsuccessful, remaining as ISRM.

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.

ROCHA MEDAL 2018:

The winner for 2018 was announced at the Council Meeting:

Michael du Plessis (South Africa) with the thesis "Design and Performance of Crush Pillars" (University of Pretoria)

See the awards briefing in this issue for information regarding the 2019 Rocha medal.

YOUNG PROFESSIONALS:

At Afrirock the Board, mainly via the Education Fund, promoted an Early Career Forum. Eight young professionals from South Africa and Zimbabwe were funded to present their work at a special session during the conference. After the session there was an open discussion on the experience and how best to continue such events and communication.

ISRM ON-LINE LECTURES

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

- 18th by Dr Marc Panet on 13th July 2017 with the title " The deformations in the vicinity of the face of a deep tunnel"
- 19th by Professor Xia-Ting Feng on 26 September 2017 with the title "Rockbursts at Deep Tunnels"

The lectures are available on the ISRM website along with earlier lectures.

COMMUNICATION:

For NZGS members affiliated to ISRM there is a members area on the ISRM website (www.isrm.net) with access to further products. The ISRM Digital Library is part of OnePetro (https://www.onepetro.org), a large online library managed by the Society of Petroleum Engineers. ISRM individual members are allowed to download, at no cost, up to 100 papers per year from the ISRM conferences.

Other recent items on the website include:

a) Video of point load strength test (under testing methods, ex Korea university) (https://www.isrm.net/gca/?id=1233).

b) Video course on "Key Principles in Rock Mechanics" by Prof Jian Zhao – 6 modules (https://www.isrm.net/gca/?id=912)

The ISRM website heading "Products and Publications" carries a range of ISRM-related items not already mentioned including:

- **Suggested Methods** – Apart from compilations in the Blue (1974 to 2006) and Orange (2007 to 2014) books, all the procedures have been published in the journal "Rock Mechanics and Rock Engineering" and the "International Journal for Rock Mechanics and Mining Sciences".
- **Slide Collection** – 280 slides organised into 14 themes (each with 20 slides) available ISRM members registered on the website.
- **Videos and Courses** – copies of selected past Mueller lectures and rock mechanics principles video courses by Prof. Zhao Jian (2016 - Monash University, Melbourne), Prof. John Hudson (2014 - Imperial College, London), Dr Erik Eberhardt (2011 - University of British Columbia, Vancouver) and Prof. Maurice Dusseault (2008 - University of Waterloo) available to members through the website.
- **The five-volume set** "Rock Mechanics and Engineering", edited by Prof. Xia-Ting Feng, with the editorial advice of Professor John A. Hudson, is now available updating on the five-volume set "Comprehensive Rock Engineering" (1993). It is published by CRC Press with a 30% discount to ISRM members.

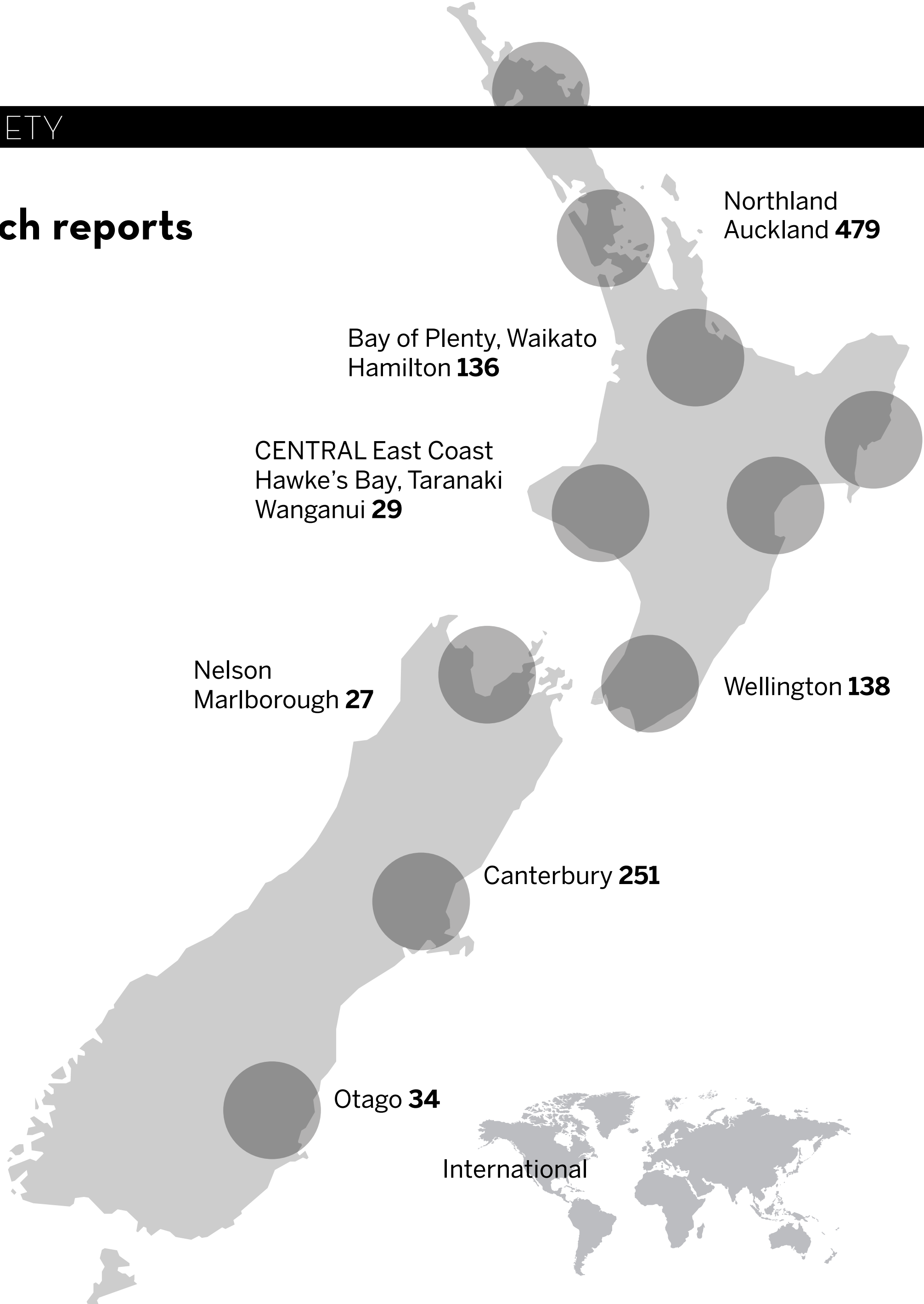
COMMISSIONS

There are 17 ISRM Commissions in the 2015 – 2019 term (not listed here but included in a previous Geomechanics News). Commission purposes and anticipated products, along with membership, are included on the ISRM website (links on https://www.isrm.net/gca/?id=153).

Stuart Read

02 November 2017

Branch reports



AUCKLAND

Auckland has been treated to a continued series of exciting talks this year. John Begg, a geologist from GNS, spoke on the urban mapping project which will see more detailed geological maps produced for the area. An exciting prospect which drew great interest. Bruce Hayward, both a geologist and a marine ecologist, presented on the topic of his new book Out of the Ocean into the Fire: Geology of Auckland, Northland and Coromandel Peninsula. If you would like a copy it is now out in book stores. Professor Nicolas Sitar from the University of California, currently in New Zealand on sabbatical, kindly spoke on the seismic performance of slopes and design of retaining structures. A very clear and useful talk for practitioners. Nick Wharmby of March Construction treated a lucky group to see the diaphragm walling going on in the Chief Post Office building down at Britomart. This work is part of the City Rail Link project. Thanks to Nick for organising access to this site. In New Zealand for the NZGS symposium, Ruth Allington, joint senior partner of GWP Consultants LLP, spoke of the importance of engineering geology and geotechnics in the design, operation and rehabilitation of quarries. Also in New Zealand for the symposium, Professor John Atkinson presented his Burland Lecture on basic geotechnical engineering skills - what can graduates do?

The Auckland Branch has a new face, Chris Wright, a geotechnical engineer at Riley Consultants on the North Shore. Welcome Chris.

Many thanks to Geotechnics, Tonkin + Taylor, Geofabrics, Ground Investigations and Beca for supporting our talks this year.

★ 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**

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

Above: Great turn out in Wellington for Russ van Dissen's talk on the complex fault rupture mechanism for the 2016 Kaikōura earthquake, hosted by Opus

WELLINGTON

The Wellington branch has been pleased to host two talks in **August** and **September**. The first by Russ van Dissen from GNS focussing on the complex multi-fault rupture system of the Kaikōura earthquake. This event was well attend by around 50 people. Russ' engaging presentation style made the talk accessible to all. Thanks to Opus for hosting and providing refreshments.

In September we had the honour of hosting Dr. Connor Hayden from the University of Auckland for a joint NZGS, NZSEE and SESOC presentation. His talk looked at liquefaction induced building performance, focusing on a series of centrifuge tests on both isolated and adjacent structures. His findings have implications for both geotechnical and structural engineers and generated some good discussion among the participants. Thanks to Griffiths Drilling and Geotechnics for sponsoring the catering, and thanks to Victoria University School of Architecture for hosting us.

In November Ruth Allington will include Wellington in her tour of talks on Quarry design. Coming up we are also planning a panel discussion and field trip for early next year.

It was also a privilege for Tonkin + Taylor to host Prof. Ishihara, Prof. Hamada, Prof. Nishimura and Prof. Cubrinovski at their Wellington office on 22 August 2017.

Pro. Ishihara and Prof. Hamada kindly offered a lunch time presentation to NZGS Wellington members. Approximately 50 members attended this event.

CANTERBURY

Things have been really quite on NZGS Canterbury front this last half of 2017. We have been fortunate enough to host Ruth Allington's tour **The Importance of Engineering Geology and Geotechnics in the Design, Operation and Rehabilitation of Quarries.**

Unfortunately, we only had a small turn out of approximately 16 people for this Presentation.

If you have any Presentations/ site visits you would like to see in the Canterbury area please contact your Branch Reps Sam or Jennifer.

NORTHLAND

Philip Cook
I am a Chartered Professional Engineer. I have an interest in risk assessment, landslides, Northland Allochthon geology, liquefaction, and seismic assessment for earthquake resistant foundations, foundation settlement.
Look forward to improving the geotechnical features of soils in Northland. Enjoy the coastal lifestyle of Northland
phil@coco.co.nz

AUCKLAND

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
James 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

Christopher Wright
Chris is a geotechnical engineer at Riley Consultants Ltd. He has bachelor degrees in civil engineering (University of Southern Queensland) and finance (Massey University) and is currently undertaking post-graduate studies in geotechnical engineering at the University of Auckland. He began in civil engineering and infrastructure asset management, and progressed to geotechnical engineering.
cwright@riley.co.nz

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 de Graaf
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.degraaf@beca.com

HAWKE'S BAY

Tom Grace
Tom is a geologist who has worked for consulting companies on a large range of projects - predominately mineral exploration, mining feasibility & development and geotechnical projects in Southeast Asia, Canada, Australia and New Zealand. Tom has a strong interest in ground testing (CPT, surface and downhole geophysics, downhole testing).
tgrace@rdcl.co.nz

Tom Bunny
Tom is a Senior Engineering Geologist with MWH, part of Stantec and is the geotechnical discipline in New Zealand. His specialities include geotechnical investigation, site hazard assessment, ground modelling, risk management, earthworks and stability assessments for central and local government, SOE's, CCO's, commercial and panel partners throughout NZ, Pacific Region, and New Zealand.
Tom.Bunny@stantec.com

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

WELLINGTON

Nima Taghipouran
*Nima is a geotechnical engineer at Beca in Wellington, with four years of experience following graduation from the University of Auckland. Nima has been involved in a wide range of projects in the North Island. His areas of interest include design of deep foundations and retaining structures in highly seismic areas and earthquake hazard assessments.***nima.taghipouran@beca.com**

Aimee Rhodes
Aimee is a graduate geotechnical engineer with Opus. She recently completed her Masters degree in Earthquake Engineering with the University of Canterbury. Aimee has experience with liquefaction analysis and soil characterisation having worked on modelling liquefaction in stratified soils for her Masters research.
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Shirley Wang
Shirley is a Geotechnical Engineer with 8 years of experience working at Tonkin & Taylor Wellington Office. She graduated from Canterbury University with a BE(Hons) in 2009. She has experience in seismic assessment, geotechnical and environmental investigation, slope stability, foundation design and construction monitoring.
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Jerry Spinks
Jerry is a chartered professional engineer who has worked in New Zealand and the UK. Returning to New Zealand in 2011, he has worked on a variety of building projects. Jerry recently joined Jacobs Engineering, where he has been undertaking a number of landslide assessments and is working on the Pinehaven Flood Protection Scheme.
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NELSON

Paul Wopereis
Paul is Principal Engineering Geologist with MWH, part of Stantec 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.
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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.
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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/

Teresa Roetman
It is now over 2 years since taking on the role of Management Secretary and I have thoroughly enjoyed talking to and meeting many of our members as well as advertisers that support this magazine. Our Branch Representatives continue to inspire me, putting in many hours on a volunteer basis to bring our members Presentations that are both informative and enjoyable and we are always appreciative for the companies that sponsor these presentations. Please support your branch Rep's as they are working for you. If you have an idea for a Presentation/site visit please do not hesitate to contact your local branch or send an email to secretary@nzgs.org
I wish you all a safe, happy, holiday season!

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 2018

POSITION	NAME	EMAIL
Chair	Tony Fairclough	chair@nzgs.org
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ISRM Australian Vice President	Stuart Read	S.Read@gns.cri.nz

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

NZGS Membership SUBSCRIPTIONS

Annual subscriptions cost $105 per member. First time members will receive a 50% discount for their first year of membership; and student membership is free. Membership application forms can be found on the website http://www.nzgs.org/membership.htm or contact the NZGS Secretary on secretary@nzgs.org for more information.

Advertisers' Directory

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
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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

2018

22-26 MAY 2018
St Petersburg – Russia
European Rock Mechanics Symposium EUROCK 2018 "Geomechanics and Geodynamics of Rock Masses

27-30 MAY 2018
Shanghai, China
4th GeoShanghai International Conference

4-6 JUNE 2018
Seville, Spain
15th International Conference on Structures under Shock and Impact

6-8 JUNE 2018
Seville, Spain
11th International Conference on Risk Analysis and Hazard Mitigation

10-13 JUNE 2018
Austin, Texas, USA
Geotechnical Earthquake Engineering and Soil Dynamics V

17-20 JULY 2018
London, UK
ISSMGE TC104 (Physical Modelling in Geotechnics)

13-16 AUGUST 2018
Vienna, Austria
The China-Europe Conference on Geotechnical Engineering

3-5 AUGUST 2018
Hong Kong
7th International Conference on Unsaturated Soils (UNSAT2018)

16-21 SEPTEMBER 2018
Seoul, Korea
11th international Conference of Geosynthetics

17-21 SEPTEMBER 2018
San Francisco, USA
Engineering Geology for a Sustainable World

24-27 OCTOBER 2018
Anaheim, CA, USA
43rd Annual Conference on Deep Foundations

28-1ST NOVEMBER 2018
Hangzhou, China
The 8th International Congress on Environmental Geotechnics

29-3RD NOVEMBER 2018
Singapore
ARMS 10 "Rock Mechanics in Infrastructure and Resource Development".

24-29 NOVEMBER 2018
Cairo, Egypt
GeoMEast 2018 International Conference

2019

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

1 JULY 2019
Rykjavik, Iceland
ECSMGE 2019 – XVII European Conference on Soil Mechanics and Geotechnical Engineering

7-11 SEPTEMBER 2019
Budapest, Hungary
6th International Conference on Geotechnical and Geophysical Site Characterization

13-18 SEPTEMBER 2019
Iguassu Falls – Brazil
ISRM 14th International Congress of Rock Mechanics

7-10 OCTOBER 2019
Cape Town – South Africa
XVII African Regional Conference on Soil Mechanics and Geotechnical Engineering

14-18 OCTOBER 2019
Chicago, USA
44th Annual Conference on Deep Foundations

14-17 OCTOBER 2019
Taipei, China
XVI Asian Regional Conference on Soil Mechanics and Geotechnical Engineering

18-22 NOVEMBER 2019
Cancun, Mexico
XVI Panamerican Conference of Soil Mechanics and Geotechnical Engineering

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


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