NEW ZEALAND GEOTECHNICAL SOCIETY INC

JUNE 2019 issue 97

NZ GEOMECHANICS NEWS

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2019 NZGS Geomechanics Lecture

13TH ANZ GEOMECHANICS CONFERENCE

GEOTECHNICAL DESIGN PRACTICE

PHOTO & POSTER COMPETITION WINNERS

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:

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- Removable distributive tension anchors (SW-SMART)
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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.

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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
POST-TENSIONING	SEISMIC UPGRADE	DRILLING

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).

Grout Grippa bluey OVM Tighter earth anchor technology SAMWOO-AnchorTec

GEOTECHNICS

Geotechnics now offers Pressuremeter Testing for soils and soft rock using the **RocTest TEXAMe Pressuremeter**

The pressuremeter test is an in-situ testing method which is used to measure the in-situ stress-strain relationship of the soil, providing parameters such as the elastic modulus and limit pressure.

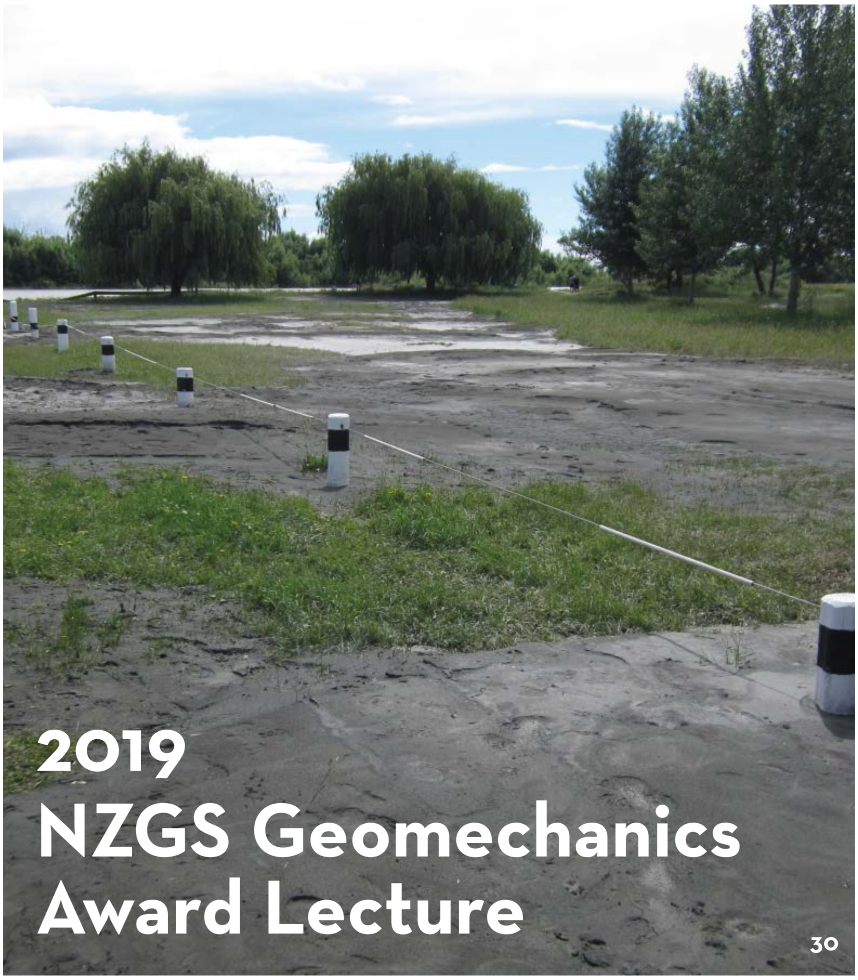
Pressuremeter features:

- Manually controlled hydraulic loading ensuring good repeatability of test
- Automatic acquisition of pressure and volume measurements
- Real time visualisation of test results for enhanced control of testing
- 1.5m minimum test spacing with tests to 45m depth

News

PHOTO COMPETITION

CONFERENCES



PROJECT NEWS

Technical

Society

OBITUARIES

COVER IMAGE: Photo Competition Winner; **Kylie Johnston**, *CGW Consulting Engineers*
Ground investigations, Tahunanui Slump, Nelson

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

INTRODUCTION

As some of you are no doubt aware, my term as the NZGS Chair comes to an end on 30 September 2019 and this will be my last "Chairs Corner" article.

I wish to thank all of the NZGS Management Committee for their hard work and support through my term as Chair, and, I look forward to helping Ross Roberts transition into this challenging but rewarding role by 01 October. I also extend my thanks to all NZGS members for entrusting me to be the Shepherd of your great organisation through the 2017 to 2019 period – it has been an honour.

The call for nominations to stand for election onto the NZGS National Management committee is scheduled to occur during June 2019. I strongly encourage all members who are passionate about their profession to submit a nomination. I am confident that, like me, they will find the experience rewarding. As always, please do not hesitate to contact the Editor of this publication, or any of the NZGS Management Committee, if you have any questions or comments regarding the articles which are published herein or the general operation of your technical society. Contact details for your management committee, the society secretary and the Editor are provided at the back of this magazine and on the NZGS website: **http://www.nzgs.org/**

MBIE PUBLIC CONSULTATION

I am sure most of you are aware that the Ministry of Building, Innovation and Employment (MBIE) launched their "Raising the bar for the building sector" public consultation program on 16 April.

Without doubt this is the most significant reform legislative program since the current Building Act was introduced in 2004. The proposed reforms will affect people, products and practices across the entire national building and construction sector. Full details of the proposed reform program can be downloaded from the MBIE website: **www.mbie.govt.nz/building-reform**

As outlined in the previous emails sent to all NZGS and Engineering New Zealand members on 18 April 2019, I respectfully and strongly recommend that all NZGS members review all of the information which is available on the above MBIE web page, discuss such information with your colleagues as appropriate, and, provide any feedback that you may have back to MBIE as quickly as possible if the deadline date has not yet passed. Even if the deadline date has passed, you should read and familiarise yourself with this information.

You can submit your feedback to MBIE via one of the following methods:

1) as an individual by completing the MBIE submission form and emailing it to "building@mbie.govt.nz", or, by filling out the online survey which is linked under the "How to make a submission" section at the bottom of the MBIE webpage (see above for the web address), and/or,
2) as part of a submission which is being compiled by a stakeholder group such as your employer.

At the time of writing this report the deadline date for all submissions was 5:00pm on Sunday 16 June 2019. However, please check the MBIE website as it is my sincere hope that MBIE will extend this deadline given the complexity and importance of the reforms which are currently proposed.

In summary, the key areas and objectives of the proposals currently outlined by MBIE are as follows:

Building products and methods:

- clarify roles and responsibilities for building products and methods.
- require manufacturers and suppliers to provide information about building products.
- strengthen the framework for product certification, and,
- make consenting easier for modern methods of construction.

Occupational regulation:

- change the licensed building practitioners scheme to raise competence standards and broaden the definition of restricted building work.

- introduce a new licensing scheme for engineers and restrict who can carry out safetycritical engineering work, and,
- remove exemptions that allow unlicensed people to carry out sanitary plumbing, gas fitting and drain laying work.

Risk and liability

- require a guarantee and insurance product for residential new builds and significant alterations, and allow homeowners to actively opt out of it, and,
- leave the liability settings for building consent authorities unchanged.

Building levy

- reduce the building levy from $2.01 including GST to $1.50 including GST (per $1,000).
- standardise the building levy threshold at $20,444 including GST, and,
- allow MBIE to spend funds raised by the building levy on broader stewardship of the building sector.

Offences, penalties and public notification

- increase the maximum financial penalties.
- set different maximum penalties for individuals and organisations.
- extend the time enforcement agencies can lay a charge from six months to 12 months, and,
- remove the requirement to publish key decisions in newspapers, information would still be published on publicly accessible websites and in the New Zealand Gazette.

Since the official launch of the public consultation period on 16 April 2019, and the release of the associated supporting documentation, the NZGS National Management Committee has worked in close collaboration with Engineering New Zealand (ENZ), the Structural Engineering Society of New Zealand Inc (SESOC) and the New Zealand Society of Seismic Engineering Inc (NZSEE) to ensure general alignment and agreement is achieved between these key sister organizations.

At the time of writing this article, ENZ had published a preliminary opinion which can be viewed at the following link: **https://www.engineeringnz.org/news-insights/mbie-proposes-new-way-regulating-engineers/**

Finally, the NZGS National Management Committee has developed a detailed submission to MBIE on the behalf of our society. A copy of this submission is available to view and download from the NZGS website http://www.nzgs.org/

ISSMGE AUSTRALASIAN VICE PRESIDENT

During May 2019 I received the following poignant and heart-rending message from Gavin Alexander, the current ISSMGE Australasian vice-Present and past Chair of NZGS:

This report is brief, as I have to withdraw from all my professional activities to deal with an unexpected and serious health challenge. It has been a great honour and pleasure to have represented our Societies on the ISSMGE Board, and I am disappointed to have to withdraw from that role part way through.

The NZGS Management Committee will maintain respectful communications with Gavin through this difficult time and will support him whenever possible. We wish Gavin a speedy recovery, and, extend our warmest thoughts and love to him and his family.

The NZGS National Management Committee is currently working with the Australian Geomechanics Society to confirm and appoint a new ISSMGE Australasian vice-Present for the current term.

ISRM AUSTRALASIAN VICE PRESIDENT

In September, Stuart Read will complete his four-year term as the ISRM Australasian Vice President. Stuart has done a fantastic job in this role, having volunteered countless hours of his time in service to the ISRM affiliates in Australia and New Zealand, and to the wider NZGS membership.

I thank Stuart for his tireless support to our Society over many years and wish him all the best in his future ('retirement') endeavours!

UPCOMING SOCIETY EVENTS

Planning for several NZGS events, which are to be held during Q3 and Q4 of 2019, is well underway.

Further detail and updates on all upcoming events is available via the NZGS fortnightly email to members and our website (**http://www.nzgs.org/**).

Guidelines and Standards Publications

The Earthquake Geotechnical Engineering Guidelines Series - Finalisation Project

ENZ, supported by NZGS, has submitted a proposal to MBIE to complete a final round of public consultation and feedback, update, and, finalise each of the six design-focussed Geotechnical Engineering Guideline modules which have been published to date (Modules 1, 2, 3, 4, 5 and 6).

Subject to acceptance of the current proposal by MBIE, this project is expected to commence during Q3 or Q4 2019 and take between twelve and eighteen months to complete. Details of the key dates for the overall project program will be published on the NZGS website. Further, confirmation of the closing dates for the submission of feedback for each module will be staggered and communicated to all members as part of our fortnightly email blasts, and, published on the

NZGS website as appropriate.

In the meantime, all NZGS members should not hesitate to submit any feedback or suggestions they may have for improvement of the modules documents to NZGS and MBIE at their convenience.

All ongoing feedback on the Geotechnical Engineering Guideline modules should be sent via email to: **modulefeedback@NZGS.org**.

SESOC / NZGS Piling Specification

After careful consideration and consultation with SESOC, I am pleased to confirm that the Panel of Experts for the review and update of the Auckland Structural Group "Piling Specification" document has been confirmed as follows:

Panel Chair:

Anthony Fairclough NZGS, Christchurch

Geotechnical Engineering Panel Members:

Andrew Langbein Tonkin & Taylor Ltd, Auckland
Nicola Ridgley Beca, Auckland
Martin Larisch Golder, Waikanae
Andy Dodds Arup, Auckland

Construction and Supplier Panel Members:

Nick Warmby March Construction, Christchurch
Malcolm McWhannell Brian Perry Civil, Auckland
James Harrison Fulton Hogan, Christchurch
Lian Ching Oh Firth Industries, Auckland

Structural Engineering Panel Members:

Michael Robinson Beca, Auckland
Rob Presland Holmes Consulting, Wellington
Ryan Clarke Dunning Thornton, Wellington
Tessa Beetham Aurecon, Wellington

As previously communicated, the above panel is to review, update as appropriate and reissue the Auckland Structural Group "Piling Specification" as a national document that is jointly published by SESOC and NZGS.

REPORT CLOSURE

Please do not hesitate to contact me via email on **chair@nzgs.org** or any other member of the NZGS Management Committee if you wish to discuss any issue which you believe is of direct relevance to our membership.

Your humble servant,

Tony Fairclough
NZGS Chair, 2017 - 2019

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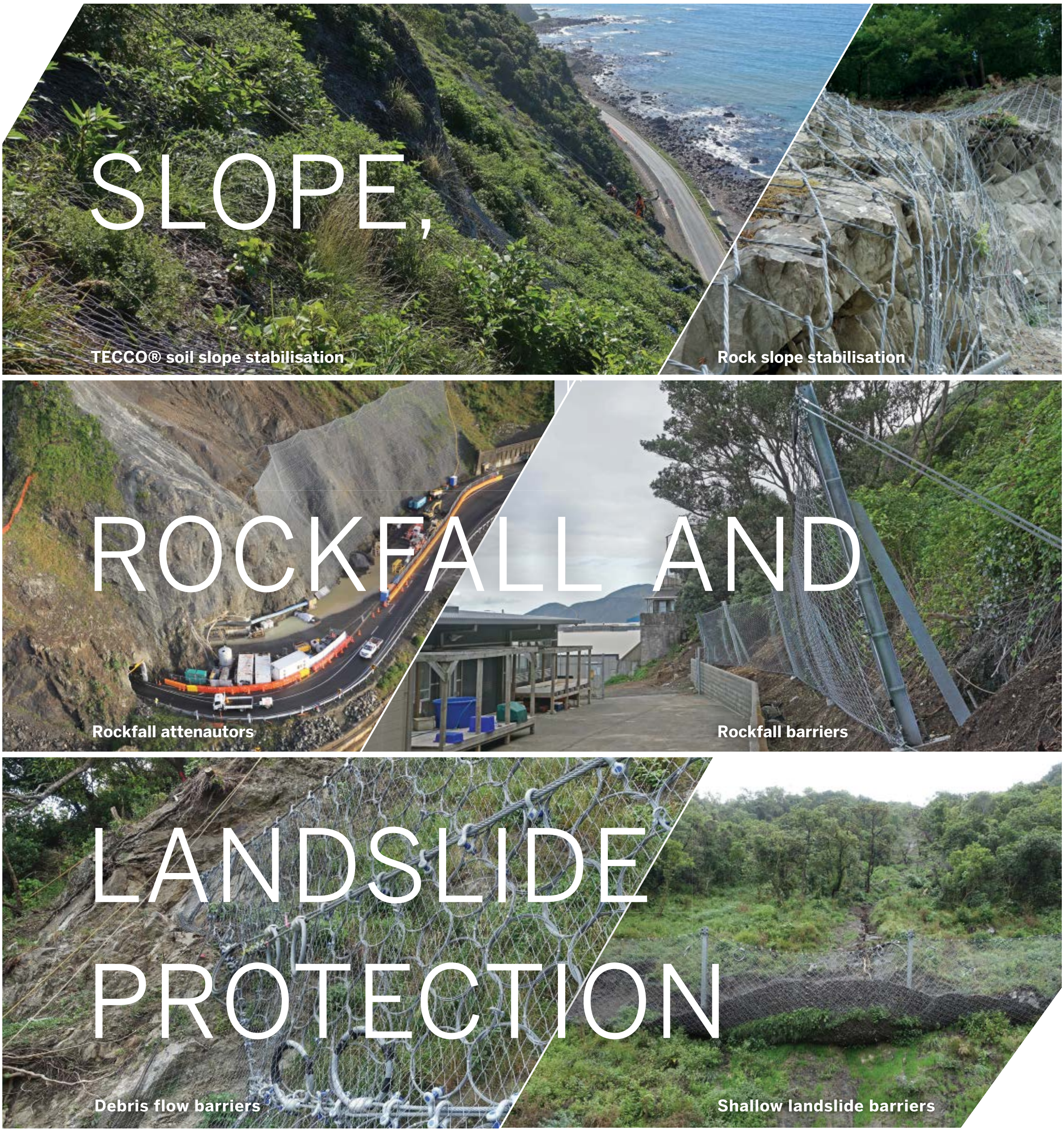

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Times they are a'changing

"NOTHING STAYS THE SAME!". In this issue we see a farewell message from the current Chair, Tony Fairclough, whom we will definitely miss, and sadly a withdrawal from all geotechnical duties by Gavin Alexander, another long-serving NZGS Committee member. And Stuart Read has also submitted his last contribution on behalf of ISRM. These fellows have all worked tirelessly for NZGS and our profession for many years, and we most gratefully thank them for that while wishing them well for their future endeavours. Tony will be lurking around as Past-Chair in the meantime...

Ross Roberts will be our new Chair, effective 1 October, and there will be a call for nominations for the new Executive Committee Members during June. "Change" is the word!

Furthermore, in this issue we have a new initiative, courtesy of George Brink. We are planning to introduce a series of articles focussing on different aspects of career development and aimed at the younger readers. The first dedicated article is planned to be in the December issue, with more to follow. You will find a bit more info tucked away in the following pages.

Technology is bringing constant change to the world, and our corner is no exception. Recent years have seen an upswing in 3D modelling, the use of UAV's and other new software and technologies. We are happy to acknowledge and support these changes, but PLEASE do not let this stop you from thinking – pen, paper, sketches and interactive discussions remain the basis of good engineering practice. The computer cannot do that for you.

Lastly, Geomechanics News is introducing a procedural change similar to that proposed by Australian Geomechanics. Beginning with our next issue, we will require all contributions of technical material to be accompanied by an Author Declaration Form (which will be available via the website). This will help us to acknowledge your contributions as original or re-published, as the case may be, without any additional hassle for the editors! As we said earlier, "Nothing stays the same!"

Don and Gabriele

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

Gabriele is a Senior Lecturer in Geotechnical Engineering at the University of Canterbury. Gabriele's research interests include earthquake geotechnical engineering and related problems; constitutive modelling for geomaterials; development of advanced laboratory and field testing devices; geo-hazard reconnaissance and mitigation; reuse and recycling of industrial granular wastes as sustainable geomaterials.
NZ Geomechanics News co-editor

PROLIFIC GEOTECHNICAL RESEARCHER RECOGNIZED BY ASCE

ASCE has honoured our very own Professor Misko Cubrinovski, Ph.D. with the 2019 Ralph B. Peck Award for outstanding contributions to the geotechnical engineering profession through the publication of several insightful field case histories, including Cubrinovski and Robinson's "Lateral Spreading: Evidence and Interpretation from the 2010-2011 Christchurch Earthquakes," *Soil Dynamics and Earthquake Engineering* (2016).

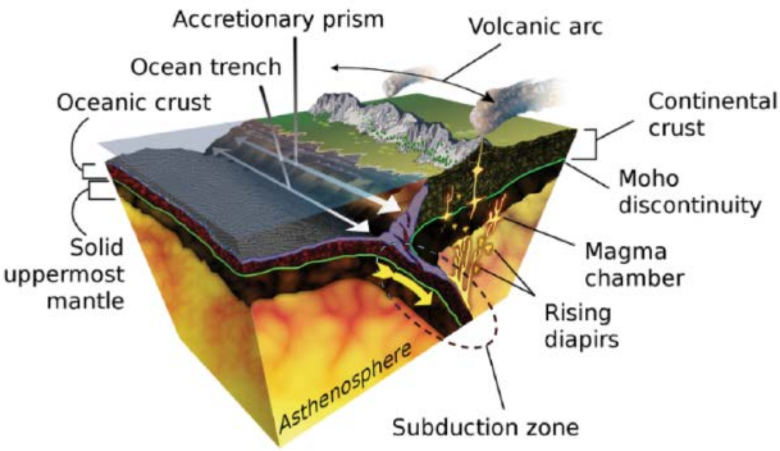
Misko has authored or co-authored over 300 technical publications and worked as a geotechnical specialist and advisor on over 50 significant engineering projects.

The Ralph B. Peck Award is presented for outstanding contributions to the geotechnical engineering profession through the publication of a thoughtful, carefully researched case history or histories, or the publication of recommended practices or design methodologies based on the evaluation of case histories.

We are proud to have Misko's 2019 NZGS Geomechanics Lecture Award lecture as the keynote paper in this issue of NZ Geomechanics News.

News – In Brief

A TECTONIC PLATE MAY HAVE SPLIT APART, PULLING EUROPE TOWARD CANADA



IN A RECENT NEWS article (published on 6 May 2019), Stephen Johnson reports that we may be seeing evidence of the birth of a subduction zone (for the non-geologists a subduction zone is where one tectonic plate is being driven under another).

Since 1969, some geologists have been puzzled by a 7.9-magnitude earthquake that occurred beneath a flat, featureless region off the coast of Portugal that's since been the location of several earthquakes. Now, a team believe that a drip-shaped mass, buried 155 miles (250 km) below the seafloor, might be responsible for the seismic activity and that the drip-shaped anomaly may represent the early stages of a new subduction zone. You can find the entire article here: **https://bigthink.com/surprising-science/subduction-zone-portugal**

Above: Phillip Flentje, Engineering Geologist and Senior Research Fellow at University of Wollongong was the winner of the NZGS Jacket for signing up as a new member of the NZGS at the ANZ2019 conference.

GEOTECHNICAL ENGINEERING in RESIDUAL SOILS COURSE

Presenter: ***Laurie Wesley***

Auckland	1st August
Wellington	5th August
Christchurch	7th August
Bay of Plenty	12th August

For the course outline and booking information visit: **nzgs.org**

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.


KIWI & PROUD
As we celebrate 30 years in New Zealand we also mark our involvement in the award winning Kaikōura Recovery project.

Adventures in Norway

Far left: Mapping a rockslide on a coastal island from a drone

Left: Skiing to a rockslide for mapping in winter (Photo S. Kjellman)

LOUISE VICK COMPLETED a PhD in rockfall modelling at the University of Canterbury in 2015, then worked in Auckland for Ormiston Associates. In 2017 she began a three-year postdoctoral fellowship in Tromsø (at UiT The Arctic University of Norway). Tromsø is in Northern Norway, high above the Arctic Circle and experiences long dark winters and summers where the sun never sets.

Norway has a lot of unstable mountains that pose a risk to communities from rock avalanche and displacement waves in lakes and fjords. The postdoctoral fellowship is in rock slope failure, and Louise is focusing on using engineering geology to understand large rock slope deformations using drones, satellite InSAR, traditional mapping and rockslide instrumentation data to examine failure mechanisms. It means a lot of interesting fieldwork in the high mountains around Tromsø!

Louise also runs a Masters course titled Rock slope failures: Geology, hazard and monitoring where she teaches students mapping and analysis techniques for large unstable rock slopes.

Louise is a member of the IAEG Executive Committee and the Chair of the YEG (Young Engineering Geologists) Committee of the IAEG. **See page 124 for more details about Louise's international role and activities.**

Above: Skiing to collect data in winter (Photo: S. Kjellman)

Above: Coring (frozen) lake sediments to age date a rock avalanche deposit (Louise in the red pants behind) (Photo: S. Kjellman)

Canterbury Earthquakes Royal Commission

John Scott, Senior Advisor – Resilience, Earthquake Commission
Gavin Alexander, NZ Geotechnical Society

The Canterbury Earthquakes Royal Commission (CERC) was formed in recognition of the importance of reviewing the performance of buildings in the Christchurch central business district (CBD) during the earthquakes. It considered the adequacy of current legal and best-practice requirements for building design, construction and maintenance. Issues considered in detail included:

a) seismicity
b) soils and the seismic design of buildings
c) the performance of CBD buildings
d) low-damage building technologies
e) earthquake-prone buildings
f) building management after earthquakes
g) roles and responsibilities within the current regulatory
h) framework.

The CERC Final Report on the lessons learnt and recommendations provided was published in June 2012. The seven volumes contained 189 recommendations, of which 175 were assigned to Ministry of Business Innovation & Employment (MBIE) to action. These seven reports can be found on this link: **https://canterbury.royalcommission.govt.nz/Final-Report**---Summary-and-Recommendations

Volume 1 covered many of the issues most relevant to geoprofessionals although a number of geotechnical recommendations are scattered throughout the other volumes as well. The full list of recommendations is included in Volume 7. Table 1 below summarises the CERC recommendations.

Subsequently MBIE prepared a formal CERC response to outline what actions had been taken to address the recommendations, and this report was published in February 2017. For the MBIE responses, you need to follow this link: **https://www.mbie.govt.nz/assets/27c53c4193/responses-cerc-recommendations.pdf.**

It is strongly recommended that all geoprofessionals familiarise themselves with the background behind the CERC geotechnical recommendations – for easy reference the relevant recommendation numbers are collated in pages 1 to 12 and 70 to 81 of the MBIE response report, where most (but not all) of the geotechnical recommendations are well summarised. In reality, there is more to be learned by reading the background to each recommendation than the recommendation itself as these were necessarily economic in their wording.

The Society can take great pride in being closely involved in the response to the majority of the geotechnical related recommendations, as most obviously demonstrated by the preparation of Geotechnical Earthquake Engineering module series 1 to 6 as well as other initiatives - commonly in partnership with MBIE and EQC.

The modules provide internationally reviewed state of practice guidance on many aspects of earthquake geotechnical engineering in New Zealand, and would not have been possible without the support of MBIE and EQC. We are fortunate to have them, and the ongoing task now is to keep them in regular use and to periodically update them to keep them current.

Reading the CERC report it was clear that there was a trend over many years towards insufficient attention and investment being given to fully understanding the performance of the ground under seismic shaking and appreciating its relationship to the overall performance of buildings. Recognition of the importance of good quality geotechnical advice in structural design has risen substantially as a result of the Canterbury and subsequent Kaikoura earthquakes. It is important that we collectively as an industry don't let time and complacency dilute this position. Once the current modules are formally finalised over the coming two years, the Society will need to plan for their ongoing maintenance and periodic updating.

Table 1. Summary of CERC's geotechnical recommendations

1	Research continues into the location of active faults near Christchurch and other population centres in New Zealand, to build as complete a picture as possible for cities and major towns.
3	A thorough and detailed geotechnical investigation of each building site, leading to development of a full site model, should be recognised as a key requirement for achieving good foundation performance.
4	There should be greater focus on geotechnical investigations to reduce the risk of unsatisfactory foundation performance. The Ministry of Business, Innovation and Employment (Department of Building and Housing) should lead the development of guidelines to ensure a more uniform standard for future investigations and as an aid to engineers and owners.
5	Geotechnical site reports and foundation design details should be kept on each property file by the territorial authority and made available for neighbouring site assessments by geotechnical engineers.
6	The Christchurch City Council should develop and maintain a publicly available database of information about the subsurface conditions in the Christchurch CBD, building on the information provided in the Tonkin and Taylor report. Other territorial authorities should consider developing and maintaining similar databases of their own.
7	Greater use should be made of in situ testing of soil properties by the cone penetrometer test (CPT), standard penetration test (SPT) or other appropriate methods.
8	The Ministry of Business, Innovation and Employment (Department of Building and Housing) should work with the New Zealand Geotechnical Society to update the existing guidelines for assessing liquefaction hazard to include new information and draw on experience from the Christchurch earthquakes.
9	Further research should be conducted into the performance of building foundations in the Christchurch CBD, including subsurface investigations as necessary, to better inform future practice.
10	Where liquefaction or significant softening may occur at a site for the SLS earthquake, buildings should be founded on well-engineered deep piles or on shallow foundations after well-engineered ground improvement is carried out.
11	Conservative assumptions should be made for soil parameters when assessing settlements for the SLS.
12	Foundation deformations should be assessed for the ULS load cases and over-strength actions, not just foundation strength (capacity). Deformations should not add unduly to the ductility demand of the structure or prevent the intended structural response.
13	Guidelines for acceptable levels of foundation deformation for the ULS and over-strength load cases should be developed. The Ministry of Business, Innovation and Employment (Department of Building and Housing) should lead this process.
14	The concessional strength-reduction factors in B1/VM4 for load cases involving earthquake load combinations and over-strength actions (Φ = 0.8–0.9) should be reassessed.
15	The strength-reduction factors in B1/VM4 should be revised to reflect international best practice including considerations of risk and reliability.
16	For shallow foundations, soil yielding should be avoided under lateral loading by applying appropriate strength-reduction factors.
17	For deep pile foundations, soil yielding should be permitted under lateral loading, provided that the piles have sufficient flexibility and ductility to accommodate the resulting displacements. In such cases, strength-reduction factors need not be applied.
18	The Ministry of Business, Innovation and Employment (Department of Building and Housing) should lead the development of detailed guidelines to address the design and use of shallow foundations.
19	The Ministry of Business, Innovation and Employment (Department of Building and Housing) should lead the development of more detailed guidance for designers regarding acceptable foundation deformations for the ultimate limit state (ULS).
20	Shallow foundations should be designed to resist the maximum design base shear of the building, so as to prevent sliding. Strength- reduction factors should be used.
21	The performance of ground improvement in Christchurch should be the subject of further research to better understand the reasons for observed variability in performance.
22	Ground improvement, where used, should be considered as part of the foundation system of a building and reliability factors included in the design procedures.
23	Ground-improvement techniques used as part of the foundation system for a multi-storey building should have a proven performance in earthquake case studies.

24	The Ministry of Business, Innovation and Employment (Department of Building and Housing) should consider the desirability of preparing national guidelines specifying design procedures for ground improvement, to provide more uniformity in approach and outcomes.
25	Detailed guidelines for deep foundation design should be prepared to assist engineers and to provide more uniformity in practice. The Ministry of Business, Innovation and Employment (Department of Building and Housing) should lead this process.
26	Because driven piles have significant advantages over other pile types for reducing settlements in earthquake-resistant design, building consent authorities should allow driven piles to be used in urban settings where practical.
27	Where there is a risk of significant liquefaction, deep piles should be designed to accommodate an appropriate level of lateral movement of the surface crust even when they are far from any watercourse.
28	Base friction should not be included as a mechanism for lateral load transfer between the ground and the building when it is supported on deep piles.
29	If reliance is to be placed on passive resistance of downstand beams and other vertical building faces, a realistic appraisal of the relative stiffness of the load-displacement response of the passive resistance compared to the pile resistance should be made.
30	For buildings on deep piles, it is not essential that the calculated lateral capacity of the foundations should exceed the design base shear at the ULS, provided that the piles have sufficient flexibility and ductility to accommodate the resulting yield displacement and kinematic displacements.
31	There are major problems in the use of inclined piles where significant ground lateral movements may occur. Where the use of inclined piles is considered, the kinematic effects that may generate very large axial loads that could overload the pile and damage other parts of the structure connected to the pile should be considered.
32	The response spectral shape factor, C (T), for deep alluvial soils under Christchurch, should be revised. The likely change in spectral shape with earthquakes on more distant faults also needs to be considered.
33	The shape of response spectra for vertical ground motion should be revised.
34	The implications of vertical ground motion for seismic design actions should be considered and locations identified where high vertical accelerations may be expected in earthquakes.
37	A more rational theoretical basis should be developed for 'magnitude weighting', which is used in the development of the design response spectra for structures.

53	There should be greater co-operation and dialogue between geotechnical and structural engineers. (Links to #185)
180	The universities of Auckland and Canterbury should pursue ways of increasing the structural and geotechnical knowledge of civil engineers entering the profession.
186	Sections 6 and 7 of the Resource Management Act 1991 should be amended to ensure that regional and district plans (including the zoning of new areas for urban development) are prepared on a basis that acknowledges the potential effects of earthquakes and liquefaction, and to ensure that those risks are considered in the processing of resource and subdivision consents under the Act.
187	Regional councils and territorial authorities should ensure that they are adequately informed about the seismicity of their regions and districts. Since seismicity should be considered and understood at a regional level, regional councils should take a lead role in this respect, and provide policy guidance as to where and how liquefaction risk ought to be avoided or mitigated. In Auckland, the Auckland Council should perform these functions.
188	Applicants for resource and subdivision consents should be required to undertake such geotechnical investigations as may be appropriate to identify the potential for liquefaction risk, lateral spreading or other soil conditions that may contribute to building failure in a significant earthquake. Where appropriate, resource and subdivision consents should be subject to conditions requiring land improvement to mitigate these risks.
189	The Ministry for the Environment should give consideration to the development of guidance for regional councils and territorial authorities in relation to the matters referred to in Recommendations 186–188.

A huge thank you to all of you who entered the annual photo competition. As usual, it is hard to choose a winner but these are the top five as chosen by our judging panel. Keep the photos coming – this is an annual competition showcasing the interesting and exciting places we get to work (and get paid for).

WINNER

FIRST – **Kylie Johnston**, *CGW Consulting Engineers*
Ground investigations, Tahunanui Slump, Nelson

PHOTO COMPETITION CONTINUED

SECOND - **Mat Avery**, *Hiway GeoStabilization*
Hanging off a cliff, Ohau Point, north Kaikoura Coast
THIRD – **Harry Follas**, *WSP-Opus*
Drone photo capture of Lemons Hill Slip Remediation, April 2018
FOURTH – **Dan Sandilands**, *Aurecon*
North wall of the Globe Progress Pit at OceanaGold's Reefton gold mine, May 2013
FIFTH – **Harry Follas**, *WSP-Opus*
An unexpected slip in urban Birkenhead, Auckland

NZ Geotechnical Society

2019 PHOTO COMPETITION

The 2019 theme is

Anything Geotechnical

WIN $250

ENTRIES CLOSE

SEPTEMBER 30

The winning photo and the top runners-up will be printed in the December 2019 issue of *NZ Geomechanics News*

SEND YOUR ENTRY TO

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2. Photos must be taken by the entrant.
3. No computer generated pictures.
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5. Winning entries will be final and no correspondence will be entered into.
6. NZ Geotechnical Society members only may enter.

The 13th Australia New Zealand Conference on Geomechanics, Perth, Western Australia

1. INTRODUCTION

The 13th Australia New Zealand Conference on Geomechanics was organised by the Australian Geomechanics Society (AGS) and held in Perth, Western Australia, at the Perth Convention and Exhibition Centre between 1 and 3 April 2019. The current ANZ series of conferences on geomechanics is held under the auspices of the ISSMGE and started in 1971 (noting that five ANZ conferences on soil mechanics and foundation engineering were held between 1952 and 1967). Previous events in the current series were held in Melbourne (1971), Brisbane (1975), Wellington (1980), Perth (1984), Sydney (1988), Christchurch (1992), Adelaide (1996), Hobart (1999), Auckland (2004), Brisbane (2007), Melbourne (2012) and Wellington (2015). The cooperation of the sister New Zealand Geotechnical Society (NZGS) is acknowledged.

2. TECHNICAL PROGRAM

2.1 Abstracts and papers

A total of 369 abstracts were submitted and peer reviewed, resulting in 199 papers in the proceedings volume. The host country led the submissions as shown in Figure 1. It is interesting to note that 10.4% of final papers came from 13 countries belonging to other regions of the ISSMGE.

The distribution of papers by the main themes of the parallel sessions is presented in Figure 2.

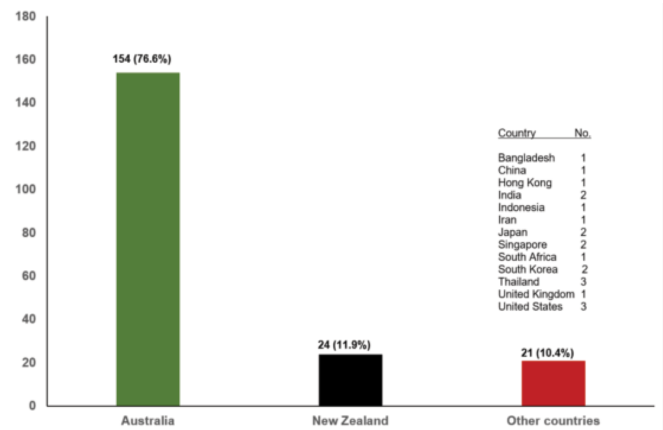

Figure 1: Distribution of papers by country of origin

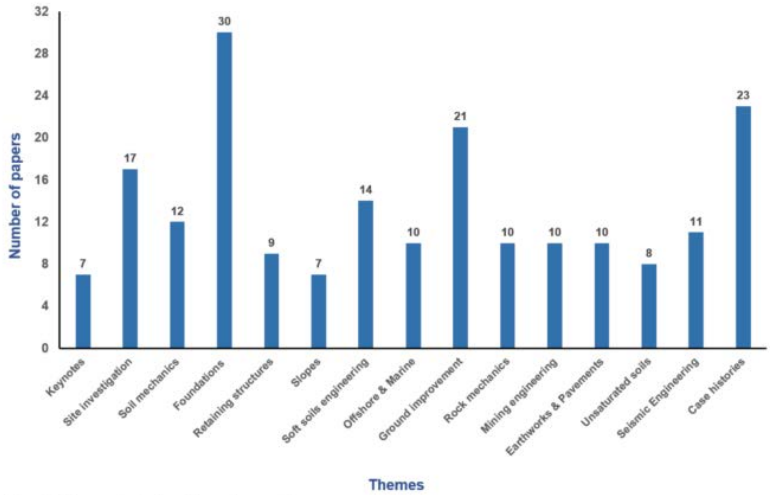

Figure 2: Paper submissions by conference themes






Figure 3: Keynote lecturers (L-R, top-bottom) - Prof Fumio Tatsuoka, Rob Day, Prof Misko Cubrinovski, Dr Doug Stewart, Dr Oskar Sigl, Mike Jefferies and Marc Woodward

2.2 Keynote lectures

Seven keynote lectures were delivered by:

- *Prof (Emeritus) Fumio Tatsuoka* (University of Tokyo and Tokyo University of Science, Japan) - Geosynthetic-reinforced soil structures for transportation – from walls to bridges
- *Rob Day* (Arup, Australia) - Trying to make a difference – and why sometimes we can't [AGS Practitioner Award Lecture (2016)]
- *Prof Misko Cubrinovski* (University of Canterbury, New Zealand) - Some important considerations in the engineering assessment of soil liquefaction [NZGS Geomechanics Award Lecture (2018)]
- *Dr Doug Stewart* (Golder Associates, Australia) - Unexpected ground movements and their impact
- *Dr Oskar Sigl* (Geoconsult Asia, Singapore) - Dealing with the challenges of underground construction
- *Mike Jefferies* (Golder Associates, UK) - The utility of critical state soil mechanics
- *Marc Woodward* (CMW Geosciences, Australia) - Effective communication – A critical component of geotechnical engineering

The full proceedings will be made available at the AGS website and the ISSMGE Online Library by the end of June 2019.

2.3 Conference awards:

An independent panel of judges from Australia and New Zealand selected the winners and highly commended finalists for the following awards that are traditionally presented at this conference series:

a) Best paper
(Joint Societies Award)
Winner: Strath Clarke, Garry Mostyn and Bernard Shen - *Collapse of the Old Pacific Highway, Piles Creek, Somersby*

Figure 4: Tony Fairclough (NZGS Chair), Sean Goodall, Ian Finnie, Su Kwong Tan and Prof Stephen Fityus (AGS Chair)

Highly commended: Ian Finnie, Rick Gillinder, Mark Richardson, Carl Erbrich, Mark Wilson, Fiona Chow, Meysam Banimahd and Steve Tyler – *Design and installation of Mobile Offshore Drilling Unit mooring piles using innovative drive-drill-drive techniques*

b) Best paper by a young professional (< 35 years old)
Winner: Sean Goodall – *Design of a reinforced soil capping beam over a soil-bentonite barrier wall*
High Commendation: James Watton and Mark Fowler - *Geotechnical management of large scale slope deformations at the Teal Gold project*

c) Best poster
Winner: Su Kwong Tan – *Geotechnical design and construction considerations for Old Mandurah traffic bridge project*
Highly Commended: Elisabeth Boczek and Marc Amtsberg – *Operational and construction impacts on GCL performance in TSFs*

Each winner received a certificate and A$1,000 and the highly commended finalists received a certificate and A$500.

3. SOCIAL PROGRAM

A pre-conference field trip showcasing Perth's regional geology highlights was organised. The tour went to the rugged Darling Range to the East of Perth and visited Canning Dam, its quarry and the surrounds. Then it returned to the centre of Perth for lunch at the iconic Old Swan Brewery. After lunch the tour head to the magnificent Indian Ocean Coastline with its coastal limestones in Fremantle at the Fremantle Port, historic Round House and Whalers Tunnel. From Fremantle the tour returned to the Perth CBD.

The conference gala dinner was held at Fraser's in Kings Park, the state's reception centre that celebrates the best Western Australia has to offer in food and wine. The event included entertainment from Domenic Zurzolo, one of Australia's premier guitarist, and Mick Collis (author, poet and rugby commentator) with a fascinating story of how it took him '42 years to play for Australia'.

4. CONFERENCE ATTENDANCE

A total of 465 delegates (including exhibitors and sponsors) attended the conference (Figure 6). The distribution by country of origin is presented in Figure 7, noting that apart from Australia and New Zealand, 20 other countries were represented.

Figure 5: Domenic Zurzolo (left) and Mick Collis during the gala dinner

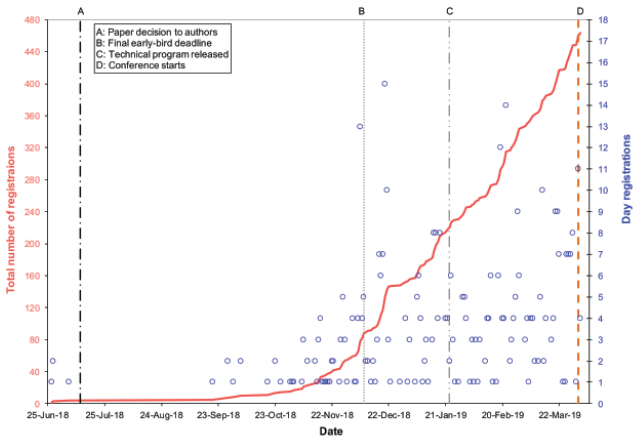

Figure 6: Day and total number of registrations by date

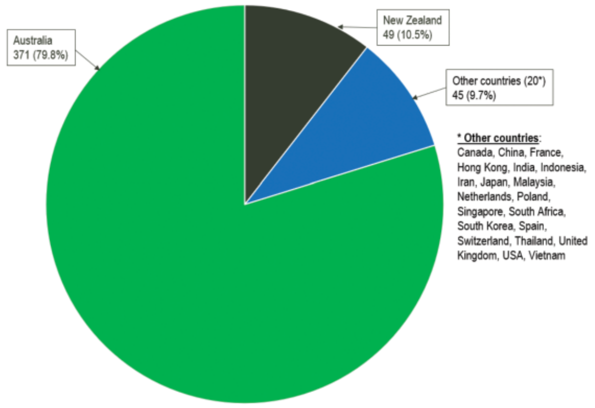

Figure 7: Registrations by country of origin

A total of eight plenary sessions were held, including seven keynotes and a panel discussion organised by the ISSMGE's Corporate Associates Presidential Group on the topic of 'Collaboration in geotechnical engineering – Impact on Research and Project Delivery'. A total of 179 papers were presented in 40 parallel sessions. The technical program was complemented with 20 electronic poster presentations.

5. EXHIBITION

The exhibition space was sold-out to accommodate 43 booths for sponsors and exhibitors. Two prizes of A$500 were drawn among attendees that completed a delegate passport by visiting several stands.

6. VENUE FOR NEXT ANZ CONFERENCE ON GEOMECHANICS

Hosting of the ANZ Conference on Geomechanics rotates 2:1 between Australia and New Zealand. The AGS will host the 14th ANZ Conference on Geomechanics in Cairns, Queensland in 2023.

7. ACKNOWLEDGEMENTS

The AGS thanks all the delegates, speakers, voluntary reviewers, sponsors and exhibitors for their contribution to the success of this event. A special mention to Arinex, Professional Conference Organiser, for their support in the organisation.

Reported by
Hugo Acosta-Martinez,
Aurecon, Australia
Michael Smith,
Water Corporation, Western Australia

Figure 8: General view of exhibition area

ISSMGE YMPG In-person Meeting Report

Left: YMPG with ISSMGE Board Members

ON THE 8TH and 9th of March, the ISSMGE Board meeting was held in Singapore. The ISSMGE YMPG took the opportunity to have their first in-person meeting since the YMPG was established back in back in 2009, a refreshing change to the ad-hoc skype meetings. The mission of the YMPG is to increase the attractiveness of the ISSMGE for the younger generations of geotechnical engineers. Meeting in-person was an opportunity to grow and strengthen connections among the board members and plan for the future.

The purpose of the in-person meeting was to discuss and organise the Bright Spark Lecture awards (locally, regionally, and internationally), provide updates and plan regional conferences for young members and in collaboration with the Corporate Associates Presidential Group (CAPG), discuss and plan engagement with young members, and review the YMPG mission statement and goals.

Eight members of the YMPG were able to attend the in-person meeting, representing each region across the world. Attendees included Lucy Wu (Chair), Jean Potgieter (Vice Chair), Ceres Chung (Secretary), Aswin Lim (Indonesia), Fabio Tradigo (Europe), Ezra Tjung (North America), Vitor Pereira Faro (South America), and Ashe Cooper (Australasia). The YMPG and ISSMGE board are both made up of a combination of academics and practitioners from across the globe.

The group recognised that news about young geotechnical engineers within the Engineering Geology and Geotechnical Engineering community needed to be more actively shared and updated, particularly when recognising those who has received awards. The YMPG is going to become more visible through web and news content highlighting what is going on in the geotechnical community, at a global level, for young geotechnical engineers.

With the 20 ICSMGE on the horizon, a plan of attack was developed for the selection of the Bright Spark award recipients. With the Sydney 2021 conference being an international conference, the YMPG recognised there should be a difference in distinction within the Bright Spark awards between regional and international. The distinction will be worked through with the ISSMGE board. Two winners for the upcoming European conference have been announced, congratulations to Federico Pisanò and Matteo Oryem Ciantia on your achievement.

On Sunday 9 March, the YMPG joined the ISSMGE board for the afternoon slot of their meeting to disseminate the weekends work, highlighting what was accomplished, what is planned for the future, and, specifically, the importance of the in-person meeting. The board supported questioned and supported the ideas and vision of the YMPG. They also provided a timely reminder that we are shaping the future of interaction for a part of the future Geotechnical community.

The succession planning to future proof the YMPG was also discussed. Board members of the YMPG are on a bi-annual term, with the Secretary, Vice Chair, and Chair on a four-yearly term. Discussion around the transfer of knowledge and continuation of momentum were identified as key things to be implemented to ensure the YMPG remains cohesive and relevant for young members. The YMPG plans to have a framework for this in place before the next term.

On Monday evening, the YMPG was invited to an event held by GeoSS young members, kindly hosted at the Arup office. It was an awesome opportunity to meet some of the young geotechnical engineers of Singapore, talk about the geotechnical community, and the projects that some of the members are working on. It was interesting to hear about the underground projects since this is not something that is commonly done here in New Zealand and is something we will need more expertise in for future projects.

The value of the in-person meeting was clearly shown by the amount achieved by the group over the two day period together. It also highlighted how important it is to build connections. So the next time you are sitting down at your desk, you have a personal connection with the people on the other end of the call. Getting buy-in from one another, listening to different points of view, and working together as a team. This is a key takeaway from the meeting and one that is applicable in all facets of life.

Outside of the meetings and sessions, the evenings provided a forum to get to know one another on a personal level, sample some of the sights, and get amongst some of the hawker food. I'd strongly recommend the satay skewers to anyone visiting! The Marina Bay Gardens and rooftop bars were personal highlights of mine.

I'd like to personally thank the NZGS and Beca for funding my trip to attend the inaugural meeting, along with the GeoSS for arranging the accommodation during our stay in Singapore. If you are interested in being associated with the ISSMGE and the YMPG, head over to the ISSMGE website and sign up https://www.issmge.org. Also if you are interested in what the YMPG is up to, have a look under the new section of the ISSMGE website **https://www.issmge.org/news?from=&to=&category_id=88** or join us on LinkedIn **https://www.linkedin.com/groups/7056027/**. Alternatively, if you are interested to know more, feel free to drop me a line at **ashe.cooper@beca.com.**

Reported by
Ashe Cooper, BECA

Above: YPMG with YMGeoSS Members

The Career Development of the Geotechnical Practitioner

THIS ARTICLE IS an introduction to a new regular series that will feature in the upcoming editions of *NZ Geomechanics* News. The series of articles will largely be directed towards those undergraduate and graduate students planning to practice in the fields of engineering geology or geotechnical engineering (for the purposes of this document hereafter referred to as "Geotechnical Practitioners"). It will aim to provide them with some advice and guidance about their chosen professions, focussing on aspects that may relate to the different stages of their career, providing some new ideas, guidance or just general useful tips and tricks in managing one's professional development and long-term career goals.

21st century careers, like so many other things in this day and age, invariably seem to be following a more segmented or "modular" trend, with young professionals moving around between positions, employers and even different industries at increasingly regular intervals. This often sees young professionals transition to new roles with significantly different job descriptions to the previous positions they have experience in; either doing so out of personal choice or circumstances, opportunity or some level of economic uncertainty. Each new position can effectively be viewed as a "module" completed as part of the individuals own unique career path, similar to the completion of modules towards a university qualification. There is certainly a case to be made for such an approach, and even highly desired in some industries (think Silicon Valley, the creative arts etc.), where the conscious effort to move to a new geographic region or industry forms part of the career development of the individual. In such cases, the individual's professional development benefits as a direct result of the diversity in accrued experience.

Some professions are however characterised by significant inherent diversity, i.e. variability within the overarching field of practice. Diversity is a critical part of the appeal in science, presenting opportunities to explore, enquire and test ideas to the benefit of solving engineering problems for example. In practice however, diversity can be a complexity if not well understood and adequately managed, presenting a challenge to the profession (Eggers, 2016). The geotechnical engineering and engineering geology disciplines are prime examples where significant variability in the career paths of individual practitioners may occur due to diversity throughout the profession. This diversity could be ascribed to geographic variability in the way the disciplines evolved on a global scale, variability in conditions, standards and procedures, or due to different employers and industries having different expectations of the particular

Current Student	Graduate	Professional	Mentor
• Current degree programs offered in Australasia region... • Other helpful skills to develop during your studies... • CV, interview and networking tips and tricks...	• Transitioning from student to professional... • Different industries of practice... • Stuff they do not teach you at Uni... • Handy reference when starting your career...	• The importance of ongoing professional development... • The role of the geotechnical practitioner in... • The Geotechnical Practitioner of the future...	• Looking after those starting their careers... • What I wish I knew when I started • Lessons learned... • The importance of professional ethics...

Figure 1: Potential topics relevant to the different career stages of the Geotechnical Practitioner

profession. Regardless of the reason, this may see the individual's career path take any number of different turns in direction, particularly when combined with a "modular" career path tendency. Poorly managed or too significant levels of variability will be detrimental to the professional's development, the pursuit of professional goals and the pro-actively managed career path, ultimately risking indifference towards the direction that their careers follow.

Happily, all this "variability" and "diversity" could also be viewed as "opportunity". We hope that ultimately this feature will provide the readers with some ideas about which fields of study and employment to pursue, managing their development, setting goals and becoming a professional in their chosen careers. This feature, of course, by no means aims to exclude the more "seasoned" readers. The experienced and practicing Geotechnical Practitioners might not only also significantly benefit from the views and ideas shared here, but more importantly will be crucial in contributing to the success and development of this feature. A conceptual structure and potential topics for future discussion is summarised in the figure below. Contributions will be sourced from experienced practitioners with backgrounds in a variety of industries (academia, civil infrastructure, tunnelling, mining etc.) and types of employers (universities, small to large consulting firms, corporates etc.).

The success and relevance of this feature is naturally going to depend on the contributions from different corners of the industry. Any volunteers interested in contributing towards any of the topics outlined here, or who have thoughts and suggestions of alternative content are encouraged to get in touch with **editor@nzgs.org**

George Brink
Engineering Geologist, Tonkin and Taylor

2019 NZGS GEOMECHANICS AWARD LECTURE

Some important considerations in the engineering assessment of soil liquefaction

ABSTRACT

Three important aspects in the engineering assessment of soil liquefaction, i.e. material characterization of liquefiable soils, in-situ state characterization of soils, and system response of liquefiable deposits are the subject of this paper. These aspects in the assessment are especially important in the evaluation of liquefiable soils other than uniform clean sands, such as silts, silty sands with non-plastic or low-plasticity fines, gravel-sand-silt mixtures, and interbedded deposits composed of liquefiable and non-liquefiable soils. Background of simplified liquefaction assessment procedures is first provided, and then well-documented case histories are used to demonstrate liquefaction response characteristics of actual soil deposits, and challenges encountered in their engineering evaluation. Liquefaction evaluation of gravel-sand-silt mixtures, and system response effects in liquefiable deposits are discussed somewhat in detail.

INTRODUCTION

In the two most recent damaging earthquakes in New Zealand, soil liquefaction was a major cause of damage to land and infrastructure. In the 2010-2011 Canterbury earthquakes, widespread liquefaction occurred in residential areas of Christchurch (Cubrinovski et al. 2011) affecting 60,000 residential buildings and properties (van Ballegooy et al. 2014), multi-storey buildings in the central area of the city (Bray et al. 2014), many bridges along the Avon River (Cubrinovski et al. 2014a), and lifeline networks throughout eastern Christchurch (Cubrinovski et al. 2014b). The economic loss due to liquefaction is estimated to be as high as 15 billion NZD or nearly 40% of the total economic loss caused by the earthquakes. In addition to the physical damage, liquefaction caused considerable long-term impacts on communities, as approximately 8,000 residential properties were abandoned in areas deemed uneconomical to recover.

In the more recent 2016 Kaikoura earthquake, liquefaction caused extensive damage in reclaimed land at the port of Wellington (CentrePort), a vital facility for the regional and national economy. Gravelly reclamations and hydraulic fills of sandy soils liquefied during the earthquake shaking causing substantial damage to wharves and buildings at the port (Cubranovski et al. 2017; 2018a). These recent New Zealand earthquakes clearly demonstrated that liquefaction-induced damage far exceeds tolerable levels of impacts for a modern society in spite of the significant advances in engineering assessment of liquefaction over the past 50 years.

In current engineering practice, liquefaction evaluation

Misko Cubrinovski

Misko is a professor in Geotechnical and Earthquake Engineering at the University of Canterbury, Christchurch. His research interests and expertise are in geotechnical earthquake engineering and in particular problems associated with liquefaction, seismic response of earth structures and soil-structure interaction. Misko has published over 350 peer reviewed publications, and has worked as a geotechnical specialist and advisor on over 50 major engineering projects. His honours include the 2019 Ralph B. Peck Award (ASCE), 2016 Norman Medal (ASCE), 2014 Outstanding Paper Award (ASCE), 2014 Outstanding Paper Award (EERI), Director's Award of Taisei Corporation (Tokyo, Japan), and several awards from NZGS and NZSEE. Misko has had a leadership role in the research efforts following the 2010-2011 Christchurch earthquakes, and was the lead author for two modules of the recently published MBIE/NZGS guidelines for the Earthquake Geotechnical Engineering practice in New Zealand. Misko is on the the leadership team of QuakeCoRE, NZ Centre for Earthquake Resilience, and is the leader of its Flagship Research Programme 2.

is commonly performed using simplified liquefaction assessment procedures, with key objectives in the assessment being to assess occurrence and effects of liquefaction, quantify damage to land and structures, and subsequently mitigate intolerable effects. To achieve these goals, reasonably accurate estimates of transient and permanent ground displacements are needed for complex ground and soil-structure systems. The important focus on displacements and damage estimates has been emphasized over the past couple of decades. Furthermore, the performance-based design framework provides means to consider the seismic performance of a given site and structure for various earthquake scenarios, and allows liquefaction effects to be considered not only in terms of physical damage, but also in terms of economic losses and impacts on communities. In spite of this important focus on the performance evaluation through estimation of deformation, displacements and damage, one could argue that it would be difficult to achieve the required level of accuracy and meet the above objectives in the assessment without adequate consideration of essential issues in the engineering evaluation. In this paper three such issues in the assessment are highlighted, namely:

1) Material characterization of liquefiable soils
2) In-situ state characterization of liquefiable soils, and
3) Consideration of cross-layer interactions and system response effects in liquefying deposits.

These issues are especially important in the evaluation of liquefiable soils other than uniform clean sands, such as silts, silty sands with non-plastic or low-plasticity fines, gravel-sand-silt mixtures, and interbedded deposits composed of liquefiable and non-liquefiable soils. In the first part of the paper, a brief overview of relevant background of simplified procedures is given, and then case histories from recent New Zealand earthquakes are used to demonstrate liquefaction response characteristics of actual soil deposits, and challenges encountered in their engineering evaluation. Even though simplified procedures are used as a basis for the discussion, the highlighted issues are equally relevant for advanced methods of liquefaction assessment.

2. SIMPLIFIED LIQUEFACTION EVALUATION PROCEDURES

The semi-empirical liquefaction evaluation procedures largely evolved around three assumptions and simplifications in the assessment:

1) Clean sand is used as a reference material in the assessment
2) Relative density is used as a principal parameter describing in-situ state of the soil, and
3) Each layer is evaluated independently, and in isolation, without consideration of the response and effects of other layers within the profile or the deposit as a whole.

These simplifications are directly related to the identified three aspects in the assessment (i.e. material characterization, in-situ state characterization and system response of liquefiable deposits), and are discussed in the following subsections. Note that other important assumptions and considerations in the simplified procedures are beyond the scope of this paper.

In what follows, CPT-based triggering procedures are used as a basis for the discussion, though the presented arguments are equally valid for alternative SPT- and V_s-based liquefaction triggering methods. Figure 1a shows CPT-based liquefaction triggering correlation proposed by Boulanger & Idriss (2014) expressed in terms of corrected *equivalent clean sand* cone tip resistance q_{c1Ncs}, for earthquake magnitude M_w = 7.5 and effective overburden stress σ_v = 100 kPa. The correlation shown with the solid line can be used to estimate the liquefaction resistance CRR = $CSR_{Mw=7.5', \sigma'v}$ = 100, if q_{c1Ncs} for any given layer (depth) is estimated from CPT data.

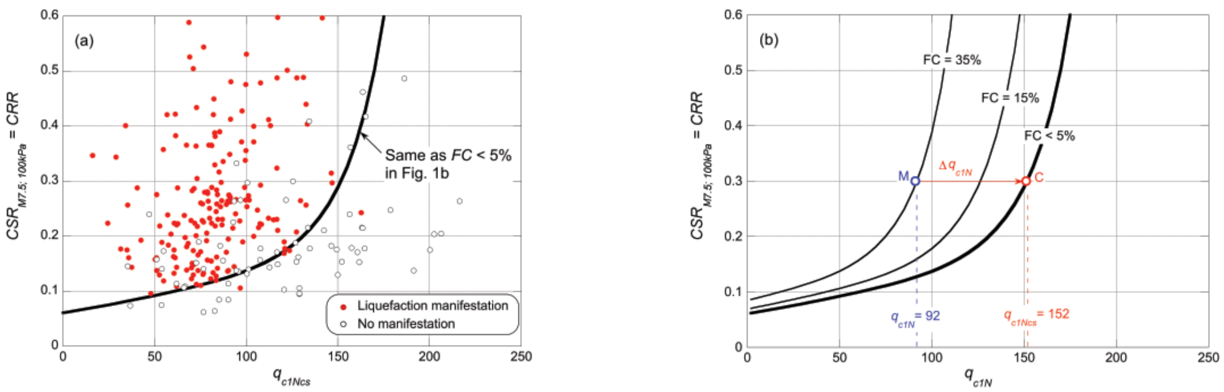

Figure 1: CPT-based liquefaction triggering correlation of Boulanger and Idriss (2014), for M_w = 7.5 and σ'_v = 100 kPa: (a) expressed in terms of corrected equivalent clean sand cone tip resistance q_{c1Ncs}; (b) expressed in terms of q_{c1N} for FC < 5% (clean sand), FC = 15% and FC = 35%; a correction of penetration resistance Δq_{c1N} for FC = 35% is illustrated with the shift from point M (measured q_{c1N} = 92) to point C (corrected q_{c1Ncs} = 152).

2.1 Clean sand as a reference material

In the triggering correlation of Boulanger & Idriss (2014), q_{c1Ncs} is calculated by correcting the measured penetration resistance for the effects of fines content using the following expression:

$$q_{c1Ncs} = q_{c1N} + \Delta q_{c1N} \quad (1)$$

Here, q_{c1N} is normalized penetration resistance obtained directly from measured cone tip resistance (q_c), and Δq_{c1N} is correction for the effects of fines content (*FC*). For clean sand, there is no correction, so q_{c1Ncs} = q_{c1N}. However, for fines-containing sand Δq_{c1N} increases with fines content, and the correction is significant. For example, as illustrated in Figure 1b, for sand with FC = 35% and q_{c1N} = 92, the adjusted q_{c1Ncs} is 1.65 times greater than the 'measured' q_{c1N} value. Such corrections for the effects of fines content can exceed a factor of 3 for low q_{c1N} values. In essence, there is a significant adjustment of the liquefaction triggering correlation based solely on the fines content, which has been adopted as a sole measure for material characteristics differentiating from clean sand.

The original correlations established from liquefaction case histories by Boulanger & Idriss (2014) are shown in Figure 1b in terms of normalized cone tip resistance q_{c1N}, for clean sand (*FC* < 5%), *FC* = 15% and *FC* = 35%. An increase in the fines content shifts the correlation upward and to the left from the reference clean sand relationship (*FC* < 5%). This shift in the correlation could be due to effects of fines on the liquefaction resistance (i.e. CRR = f(*FC*)), effects of fines on the penetration resistance (i.e. q_{c1N} = f(*FC*)), or due to combined effects of FC on q_{c1N} and CRR. It will be shown in Section 3.4 of this paper that the shift in the correlation seen in Figure 1b does not reflect the effects of fines on liquefaction resistance, but rather is predominantly due to grain-size effects on penetration resistance of soils.

2.2 Relative density as a principal state parameter

One of the key reasons for the use of clean sand as a reference material in the liquefaction assessment was that the majority of liquefaction case histories in the initial database were on clean sands and sands with small amounts of fines. Hence, the focus on clean sands was driven by the early evidence that sands have high liquefaction potential. One important corollary is that relative density (D_r) has been implicitly adopted as a reference state parameter in liquefaction assessment. Relative density is established as a parameter that works well for sands, as it represents the density state of sand, which in turn strongly influences sand behaviour under monotonic and cyclic shearing. Consequently, there is a strong correlation between liquefaction resistance (*CRR*) and relative density (D_r) of sand, as illustrated in Figure 2a. Here, conventional empirical relationships between relative density and cone tip resistance for clean sand, given in Equations 2 and 3 (Idriss & Boulanger 2008; Tatsuoka et al. 1990; Zhang et al. 2004), were used to convert q_{c1Ncs} into D_r, and then replot the CRR - q_{c1Ncs} liquefaction triggering correlation of Boulanger and Idriss (2014) in Figure 2a, in terms of CRR - D_r.

$$D_r = \{0.478(q_{C1Ncs})^{0.264} - 1.063\} \cdot 100 \quad (2)$$

$$D_r = -85 + 76 \log(q_{C1Ncs}) \quad (3)$$

The D_r - q_{c1N} relationships of Equations 2 and 3 are depicted in Figure 2b together with the relationship proposed by Robertson & Cabal (2012). The pronounced sensitivity of the penetration resistance to changes in relative density shown in Figure 2b has been one of the principal reasons for its use in the liquefaction assessment as a proxy for the in-situ density of the soil. Thus, we rely on the penetration resistance to differentiate between

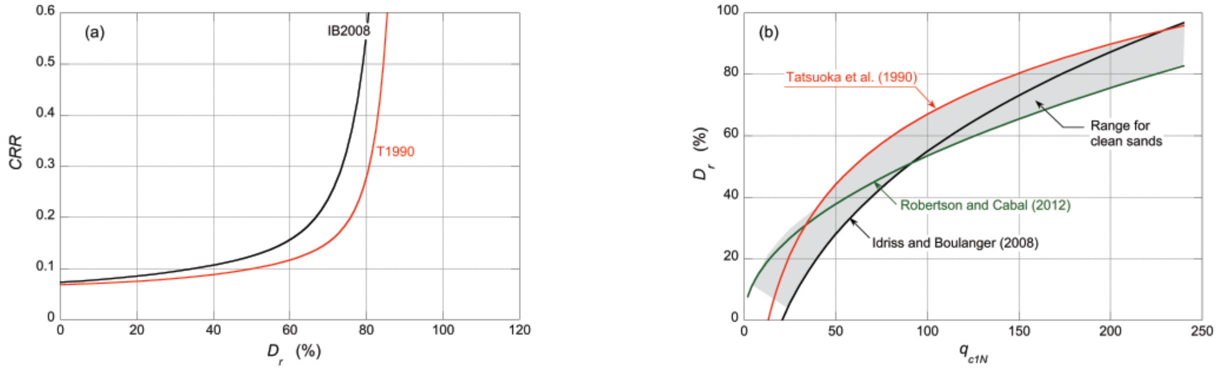

Figure 2: Relative density as a measure for in-situ state of the soil in liquefaction assessment: (a) Boulanger and Idriss (2014) liquefaction triggering correlation expressed in terms of relative density (D_r) using empirical expressions provided by Idriss & Boulanger (2008) and Tatsuoka et al. (1990); (b) relationships between D_r and q_{c1N} for clean sand used in liquefaction evaluation.

loose and dense soils or low and high liquefaction resistance respectively. However, at present, it is difficult to apply the relative density concept to soils other than clean sands, as standard procedures for evaluation of index void ratios (e_{max} and e_{min}), required in the calculation of Dr, are not available for such soils. Furthermore, there is a convincing evidence that the state concept interpretation of soil behaviour provides a more robust framework for characterization of the in-situ state of soils, as it neatly combines effects of density and confining stress on the stress-strain behaviour of soils.

The above discussion implies that, generally, material and state characterization for fines-containing soils or any liquefiable soil distinctly different from clean sand is not of the same quality and accuracy as that of clean sand.

2.3 Evaluation of liquefaction response without consideration of cross-layer interactions

The third important feature in simplified liquefaction evaluation is schematically illustrated in Figure 3 for a six-layer soil profile, in which layers 3 and 5 are liquefiable, whereas layers 1, 2, 4 and 6 are non-liquefiable. In the simplified procedure each layer is considered in isolation, and a factor of safety against liquefaction triggering (*FS*), and consequent maximum shear (γ_{max}) and volumetric strains (ε_v) are estimated separately for each layer. Thus, when calculating *FS*, γ_{max} and ε_v for any given layer, the

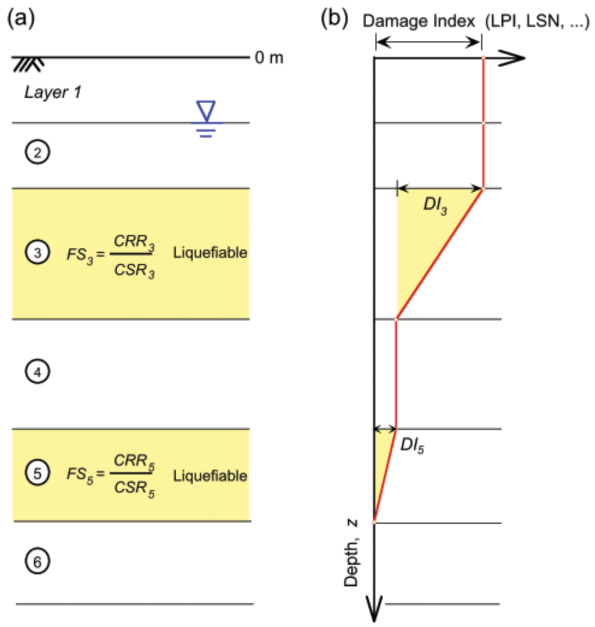

Figure 3: Schematic illustration of liquefaction assessment using simplified approach: (a) factors of safety against liquefaction triggering are calculated independently for each layer; (b) cumulative damage index is calculated for the deposit (site) by superposition of individual effects from each layer.

response and effects of other layers or interactions between layers within the deposit are ignored. In the subsequent step, liquefaction damage indices, such as LSN (van Ballegooy et al. 2014) and LPI (Iwasaki et al. 1978; Maurer et al. 2014) are calculated using specific weighting functions to quantify the damage potential of liquefying layers depending on their proximity to the ground surface. But still, when calculating the damage indices, a simple superposition of previously calculated independent effects is used, as illustrated in Figure 3b, and cross-interactions between layers through the dynamic response and liquefaction effects are simply ignored.

The key elements of the three simplifications in the assessment can be summarized, as follows:

(1) Clean sand is essentially used as a reference material, and fines content is used as a sole measure for material characteristics differentiating from clean sand; the liquefaction resistance is very sensitive to fines content in empirical CRR - qc1N relationships.

(2) Relative density is implicitly used as the principal measure for the in-situ state of the soil through penetration resistance; relative density is a well-established concept for clean sands, but its application to any soil other than clean sand is not straightforward.

(3) Finally, cross-layer interactions and overall response of the deposit are not considered in the evaluation of the liquefaction response.

In the following sections, we will further explore these issues using well-documented liquefaction case histories from recent New Zealand earthquakes.

3 MATERIAL AND STATE CHARACTERIZATION OF GRAVEL-SAND-SILT MIXTURES

3.1 Liquefaction of reclaimed land

In the 2016 M_w7.8 Kaikoura earthquake, widespread liquefaction occurred in reclamations of Wellington port (CentrePort). The liquefaction was particularly extensive and severe in the gravelly fills of Thorndon reclamation (Cubrinovski et al. 2017; 2018a). This reclamation was constructed between 1965 and 1976 by end-tipping approximately 2,900,000 m^3 of gravelly soils sourced from nearby quarries. Soft marine sediments were first removed from the seabed by dredging, and then the quarry material was dumped into the sea from truck and barge operations, thus constructing a 10 m to 20 m thick fill through a water sedimentation process. Static rollers were used to compact the top 2-3 m of the fill as soon as the fill surfaced above high-tide water level. Hence, the fill below 2-3 m depth is uncompacted. The reclamation is laterally unconfined in three directions (east, south and west) with relatively steep original slopes (1.5H:1V). The fill overlies

1-5 m thick marine sediments of interbedded sand, clay and silty clay that sit on top of the Wellington Alluvium formation, which comprises stratified dense gravels and stiff to very stiff silts.

Despite the relatively moderate peak ground accelerations of about 0.20 g at the ground surface (Bradley et al. 2017), extensive liquefaction occurred in the gravelly reclamation during the Kaikoura earthquake. The liquefaction manifestation varied from traces of ejected silts and water, to large volumes of soil ejecta with thicknesses of up to 150-200 mm. Figure 4 shows images of gravelly ejecta observed at the Thorndon Terminal which are illustrative of the worst affected areas. Large volumes of gravelly ejecta were found along cracks and fissures in the pavement, cavities and along drainage lines, which created preferred pathways for groundwater flow and soil ejecta to reach the ground surface. Visually, the ejecta appeared as a well-graded gravelly soil including some cobble-size particles, but also sand and silt.

The liquefaction resulted in large permanent ground displacements and global deformation pattern of the gravelly reclamation, as schematically depicted in Figure 5. It involved an outward

The outward displacement of the fill was accompanied by a slumping mode of deformation involving global (mass) settlement of the reclamation. As shown in Figure 5, the settlement in the central part of the reclamation was on the order of 0.2-0.3 m, whereas it increased near the reclamation edges to 0.4-0.6 m due to spreading-induced movements. The large ground displacements caused substantial damage to paved surfaces and buildings on shallow and deep foundations at the port, and damaged beyond repair two pile-supported wharves, which displaced 0.5-1.5 m laterally towards the sea. Detailed account of the observed damage to land and structures at CentrePort can be found in Cubrinovski et al. (2017).

3.2 Material characteristics of gravel-sand-silt mixtures

There are many interesting aspects of the liquefaction at CentrePort, but we will focus our attention on the identified issues around material and state characterisation in the liquefaction assessment. After the earthquake, gravelly ejecta samples were collected from 15 locations at the port for sieve analyses. Grain size distribution (GSD) curves of the collected samples are shown with solid lines in Figure 6, whereas the shaded area in the background depicts the range of GSD curves of borehole samples, which were collected several years before the earthquake.

The fill is a gravel-sand-silt mixture consisting predominantly of gravels (i.e. approximately 45-75% gravel, 15-40% sand and 10-15% fines). The good agreement between the two sets of GSD curves confirms that the ejecta samples have similar grain-size composition as the original fill, and that the gravel-sand-silt mixture indeed liquefied during the earthquake.

Liquefaction assessment of gravel-sand-silt mixtures using simplified procedures is not straightforward, as such soils effectively are not represented in the empirical

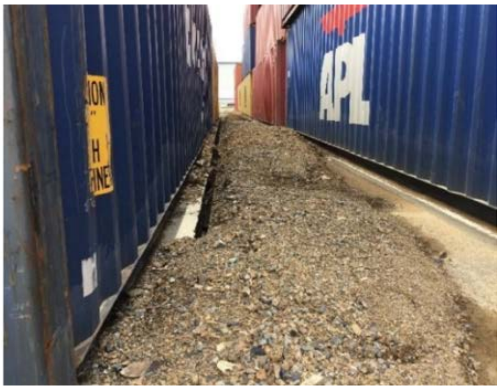

Figure 4: Liquefaction manifestation at CentrePort observed after the 2016 Kaikoura earthquake.

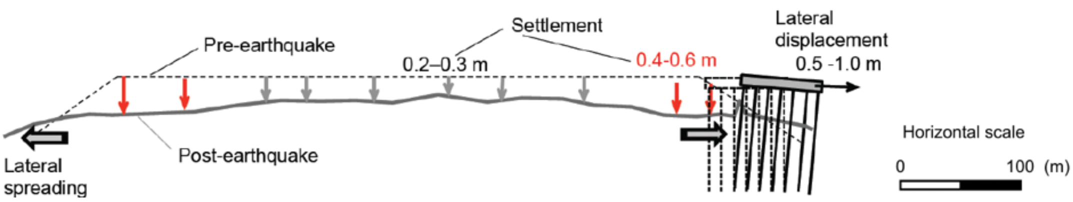

Figure 5: Global deformation pattern involving settlement (slumping) and lateral spreading of gravelly reclamation.

database. However, there is one important characteristic in the grain-size composition of the fill that one could make use of. Namely, even though the fill is dominated by gravels, there is a sufficiently large

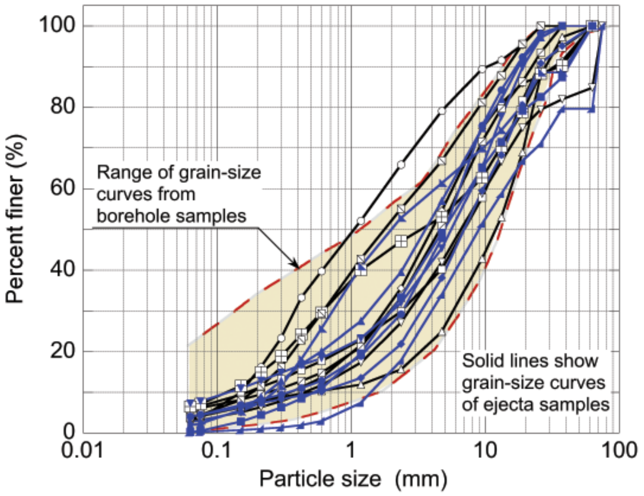

Figure 6: Grain-size distribution (GSD) curves of samples from gravelly reclamation at CentrePort; solid lines show GSD curves of ejecta samples; shaded area indicates range of GSD curves of borehole samples.

Figure 7a shows one result from such a study where index void ratios (e_{max} and e_{min}) are plotted against the fines content, for a sand-silt mixture. The plot essentially illustrates effects of fines content on the packing of sand-silt mixtures. Conceptually, when the fines content is relatively small (*FC* < 20%), the microstructure (and hence deformational behaviour) of the mixture is controlled by the sand matrix, as illustrated schematically in Figure 7b for an idealized binary packing of spherical particles. Conversely, at fines content *FC* > 40%, the microstructure is effectively controlled by the silt matrix, in which case the coarse grains (sand particles) are separated by finer grains (silt particles), as depicted in Figure 7c. As indicated in Figure 7a, there is a transition in the microstructure from sand-controlled matrix to fines-controlled matrix as the fines content increases from approximately 20% to 40%. Analogous to this interpretation for sand-silt mixtures, the 30% or more amount of sands and silts in the gravelly fill at CentrePort are considered sufficient for these finer fractions to control the soil matrix and have a critical influence on the liquefaction resistance and behaviour of the gravelly fill during earthquakes. With this background in mind, comprehensive CPT investigations were performed to characterize the gravelly reclamations at CentrePort (Cubrinovski et al. 2018a; Dhakal et al. 2019).

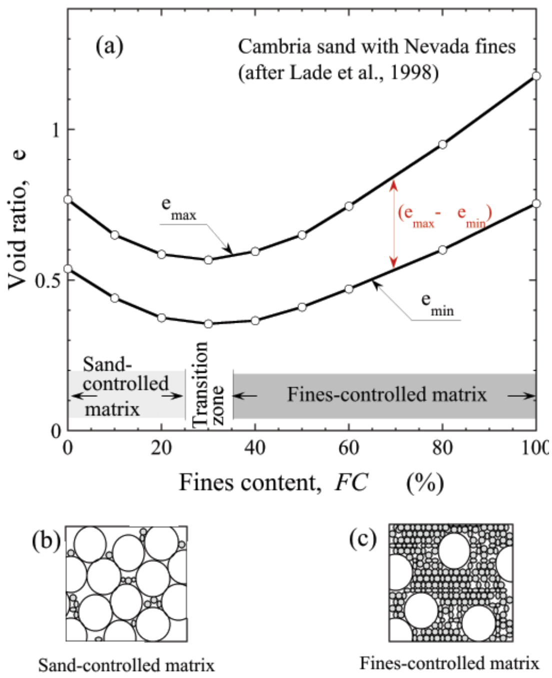

Figure 7: Influence of fines content on the packing of sand-silt mixtures: (a) variation of index void ratios with fines content for Cambria sand – Nevada silt mixtures; (b) sand-controlled matrix for *FC* < 20%; (c) fines-controlled matrix for *FC* > 40% (Cubrinovski & Ishihara 2002; after Lade et al. 1998).

3.3 In-situ state characterization of gravelly fill

About 60 CPTs were performed at CentrePort in the gravelly reclamation. Tests were performed with 10 cm² and 15 cm² cones, and field operations involved predrilling to a depth of approximately 3 m through asphalt pavement and dense compacted gravelly crust using a plugged casing with an extractable tip (Cubrinovski et al. 2018a). If early refusal was encountered during a test at depths less than approximately 10 m, the casing was pushed through the high-resistance soils beyond the depth of refusal, and then cone testing was resumed. A characteristic cross section through the gravelly reclamation derived from the CPT data is shown in Figure 8.

Careful examination of q_c traces reveals that the gravelly fill predominantly exhibited low cone tip resistance of q_c = 6.5-8.0 MPa, which represent the 25th and 75th percentile q_c values, respectively. The low penetration resistance implies low density of the fill, which is consistent with the employed construction method, sedimentation of soil particles through water, and their deposition in a relatively loose state, without any external compaction effort.

The CPT data yielded soil behaviour type index values of I_c = 2.1-2.2 for the gravel-sand-silt mixture, which imply soil behaviour consistent with sand-silt mixtures (Robertson & Wride, 1998). Indeed, the CPT data obtained in the gravelly fill are characteristic for a sand-silt mixture, and the influence of gravel is only occasionally apparent as spikes in the qc trace when gravel particles are encountered by the cone tip. This CPT-based interpretation is consistent with the anticipated governing influence of sand-silt fractions in the soil matrix implied previously based on the grain-size composition of the mixture (i.e. the presence of 30% or more sands and silts in the fill), and is also in agreement with the observed performance of the reclamation during the 2016 Kaikoura earthquake, which exhibited liquefaction severity more typical for sand-silt mixtures rather than gravels. Figure 9 shows details of two additional CPT profiles in the gravelly fill to better illustrate some of the CPT characteristics discussed above.

Tokimatsu 1988; Cubrinovski & Ishihara 1999). To estimate the density state of the fill, relative density profiles for the fill were calculated using relationships for clean sand illustrated in Figure 2b, which strictly speaking are not directly applicable for the gravel-sand-silt reclamation. The relative density of the fill was also estimated using an empirical correlation that was developed for a wide range of liquefiable soils including silty sands, clean sands and gravels (Cubrinovski & Ishihara 1999). The latter correlation was derived from a comprehensive study on the effects of grain-size characteristics of sandy soils (including clean sands, sands with fines and gravelly sands) on the packing of soils (Cubrinovski & Ishihara 2002), steady state characteristics of soils (Cubrinovski & Ishihara 2000) and

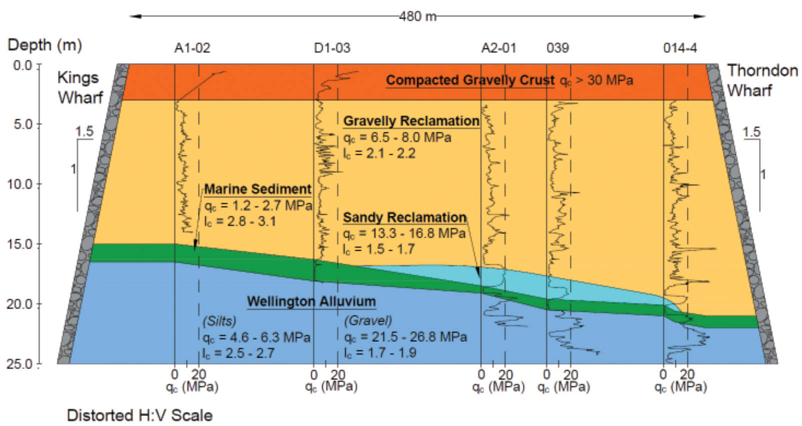

Figure 8: East-west cross-section through the gravelly fill of Thorndon reclamation showing CPT q_c traces and summary of representative q_c and I_c values (25th and 75th percentile values) for characteristic soil units (Dhakal et al. 2019).

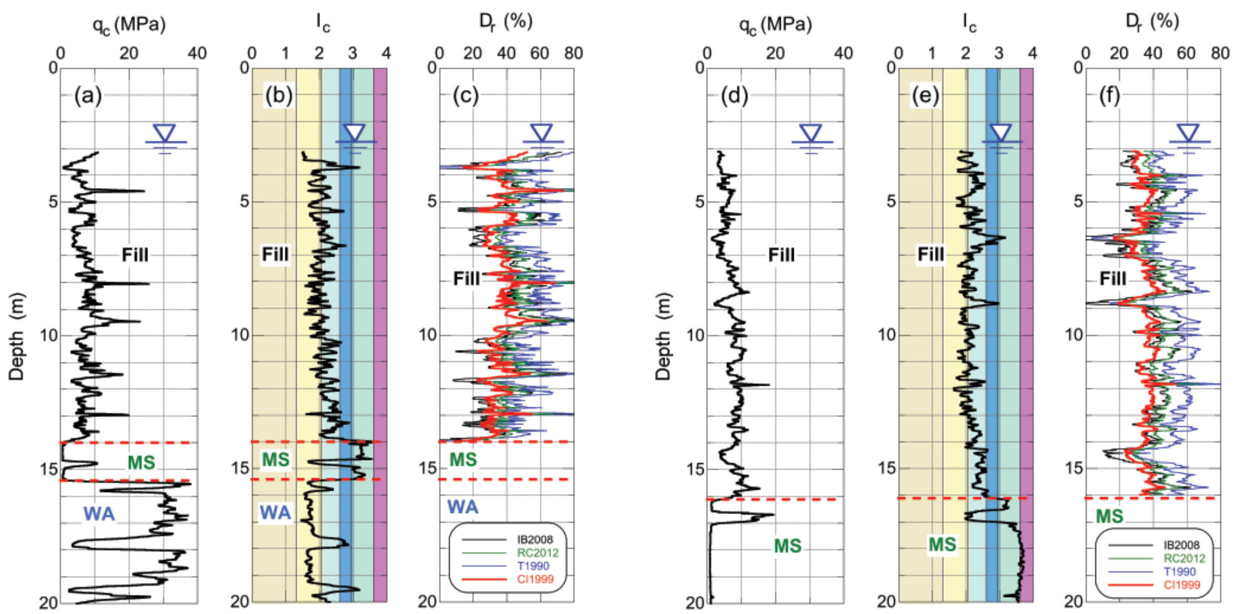

Figure 9: Measured cone tip resistance (q_c), soil behaviour type index (I_c), and estimated relative density (D_r) of the gravelly fill: (a, b, c) CPT045; (d, e, f) CPT021.

penetration resistance of soils (Cubrinovski & Ishihara 1999). Cubrinovski & Ishihara (1999) proposed penetration resistance – relative density correlation using data from high-quality samples recovered by ground freezing and SPT data with energy ratio of 78%, shown in Figure 10a. The correlation has the following original form:

$$(N_1)_{78} = D_r{}^2 \frac{9}{(e_{max}-e_{min})^{1.7}} \qquad (4)$$

which for a conventional 60% energy ratio becomes:

$$(N_1)_{60} = D_r{}^2 \frac{11.7}{(e_{max}-e_{min})^{1.7}} \qquad (5)$$

To convert $(N_1)_{60}$ into q_{c1N}, a link between the CPT-based and SPT-based triggering correlations of Boulanger & Idriss (2014) via *CRR* could be used resulting in the *QNR* = $q_{c1N}/(N_1)_{60}$ ratios shown in Figure 10b. According to the Boulanger & Idriss (2014) relationships, *QNR* ≈ 8 for q_{c1Ncs} < 100, and the ratio steadily decreases to a value of about 6 for q_{c1Ncs} = 150. Note however that Robertson et al. (1983) have shown that QNR depends on the mean grain size of soils, and that QNR increases with increasing grain size of soils. Robertson (2012) provided an updated relationship for *QNR* using I_c, in which *QNR* ranges from approximately 3 to 7, for fine-grained to coarse (gravels) liquefiable soils, respectively. To allow for different considerations of grain-size effects on *QNR*, the above correlation between the penetration resistance and relative density of Cubrinovski & Ishihara (1999) can be expressed in terms of cone tip resistance (q_{c1N}), using a generic *QNR* = $q_{c1N}/(N_1)_{60}$ term, as:

$$q_{c1N} = D_r{}^2 \frac{11.7 \cdot QNR}{(e_{max}-e_{min})^{1.7}} \qquad (6)$$

or if solved for Dr as:

$$D_r = \left\{\frac{1}{11.7 \cdot QNR} \cdot q_{c1N} \cdot (e_{max} - e_{min})^{1.7}\right\}^{0.5} \qquad (7)$$

This relationship uses the void ratio range (e_{max} – e_{min}) as a measure for the material characteristics of soils instead of conventional GSD curve parameters such as *FC* orD_{50}, as (e_{max} − e_{min}) reflects the effects of overall grain-size composition and particle characteristics (shape) of soils. Relationships (e_{max} – e_{min}) - *FC*, and (e_{max} – e_{min}) - D_{50} have been also provided (Cubrinovski & Ishihara 1999; Cubrinovski & Ishihara 2002) to facilitate the use of the relationship in practice via *FC* and D_{50}, for cases where index void ratios are not available from laboratory testing.

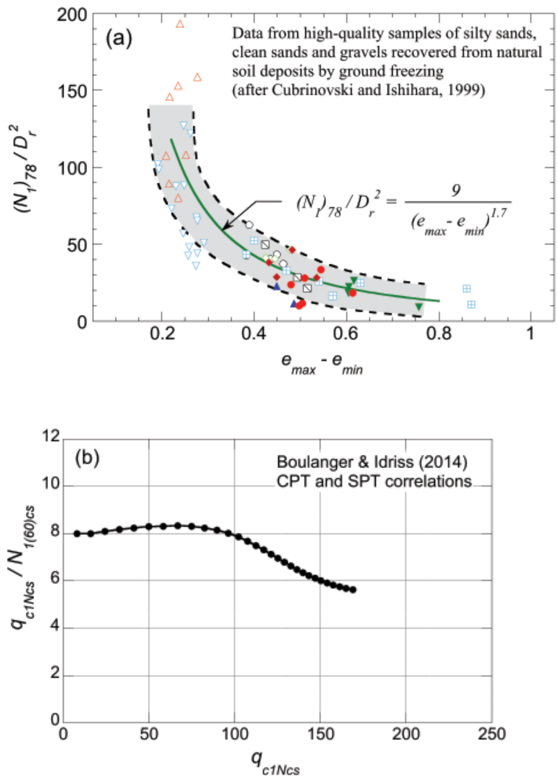

Figure 10: (a) Empirical correlation between SPT blow count and relative density of granular soils (Cubrinovski & Ishihara 1999); (b) *QNR* = $q_{c1Ncs}/(N_1)_{60cs}$ ratios derived from Boulanger & Idriss (2014) CPT and SPT triggering relationships.

Relative density estimates for the gravelly fill are shown in Figure 9c and 9f, based on the previously introduced clean sand relationships, and the relationship from Equation 7 for *QNR* = 6 and (e_{max} – e_{min}) = 0.30, which are considered representative for the gravelly fill. The latter relationship yields a low relative density of the fill of about D_r = 40%, which appears consistent with the deposition of large volume of soils through water sedimentation, and lack of compaction effort in the construction of the reclamation. It is apparent that clean sand relationships generally estimate higher relative density of the fill as they ignore grain-size effects on penetration resistance.

The material and state characterization of the gravel-sand-silt mixtures discussed above, highlight the need to carefully consider soil composition and consequently which portion of the soil matrix is controlling the behaviour during earthquakes, and then evaluate the in situ state of the soil while allowing for such compositional factors and effects in the engineering interpretation.

3.4 Empirical liquefaction triggering correlation expressed in terms of relative density

The above empirical correlation between penetration resistance and relative density can be used to scrutinize the shift in the liquefaction triggering correlation with the fines content shown in Figure 1b. Using the expression given in Equation 7, q_{c1N} – D_r relationships are plotted in Figure 11 for clean sand (*FC* = 0%), sand with *FC* = 15% and *FC* = 35%, and also gravelly sand, through the use of representative (e_{max} – e_{min}) values for these soils. In Figure 11a and 11b, *QNR* = 6 and *QNR* = 8 were used respectively. Note that QNR = 6 provides better agreement with empirical q_{c1N} – D_r relationships for clean sand, whereas *QNR* = 8 is more representative of the ratios derived from the Boulanger & Idriss (2014) triggering relationships, for q_{c1Ncs} < 100. Regardless of the adopted value for QNR, the trend in both sets of relationships consistently shows that, at a given relative density, penetration resistance of sand decreases with increased fines content. Conversely, an increase in gravel content or increase in the grain-size of soil causes an increase in the penetration resistance. These effects of grain-size composition of soils on the penetration resistance are significant.

Equations 5 and 6 can be further used to directly express empirical SPT-based and CPT-based liquefaction triggering correlations in terms of the relative density. For example, using Equation 6 together with the CRR = f(qc1Ncs) empirical expression of Boulanger & Idriss (2014), their CPT-based liquefaction triggering correlations for clean sand (*FC* = 0%), sand with *FC* = 15% and *FC* = 35% were plotted in Figure 12a against Dr. In these plots, varying *QNR* = q_{c1Ncs} / $(N_1)_{60cs}$ ratios as defined in Figure 10b were used. Similarly, Figure 12b shows empirical SPT-based triggering correlations of Youd et al. (2001) expressed in terms of D_r using Equation 5. Both plots show identical trends and effects of fines content on the liquefaction resistance:

(1) At low relative density, approximately D_r < 40%, clean sand and sands with up to 35% fines show similar liquefaction resistance.

(2) For relative densities D_r > 50%, a pronounced decrease in liquefaction resistance is seen with an increased fines content.

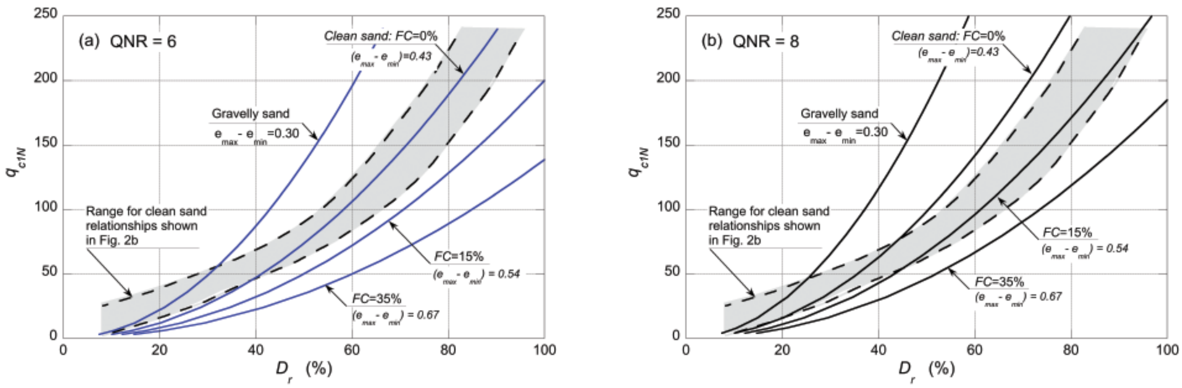

Figure 11: Relationships between penetration resistance q_{c1N} and relative density of sandy soils illustrating influence of grain size on the penetration resistance of soils (Cubrinovski & Ishihara 1999): gravelly sand with (e_{max} – e_{min}) = 0.30; clean sand with (e_{max} – e_{min}) = 0.43; sand with *FC* = 15% or (e_{max} – e_{min}) = 0.54 and *FC* = 35% or (e_{max} – e_{min}) = 0.67; (a) QNR = 6; (b) QNR = 8.

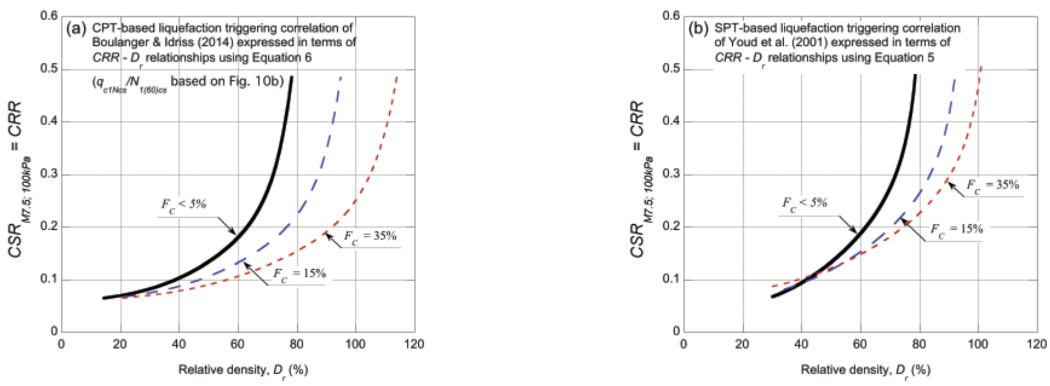

Figure 12: Empirical liquefaction triggering correlations for clean sand and sand with fines expressed in terms of relative density: (a) Boulanger & Idriss (2014) CPT-based relationships; (b) Youd et al. (2001) SPT-based relationships.

4. SYSTEM RESPONSE OF LIQUEFIABLE DEPOSITS

4.1 Liquefaction observations from the 2010-2011 Canterbury earthquakes

The final aspect to be discussed herein is the importance to consider cross-layer interactions and overall response of the deposit in the evaluation of liquefaction. Evidence and findings from the Canterbury earthquakes have shown that characteristics of the soil deposit, its overall dynamic and liquefaction response features, and effects of cross-layer interactions can significantly influence and even govern the severity of liquefaction manifestation at the ground surface.

After the 2010-2011 Canterbury earthquakes, comprehensive studies were carried out to scrutinize the accuracy of simplified liquefaction evaluation procedures in predicting liquefaction triggering (manifestation) and associated damage. These studies found that although the simplified procedures could capture general trends in the observed liquefaction damage, there were a significant number of predictions that were inconsistent with field observations. Importantly, biases in the predictions were seen with systematic mispredictions of occurrence and severity of liquefaction for specific areas and certain types of deposits. For example, simplified procedures erroneously predicted that moderate to severe liquefaction damage should have occurred over large areas in the suburbs south of the CBD during the Darfield earthquake (4 September 2010 earthquake), e.g. Beyzaei et al. (2018). However, as illustrated in Figure 13a, no such damage was observed in these parts of Christchurch after this earthquake.

To investigate the reasons for systematic mispredictions by the simplified methods, a comprehensive study was performed using 55 well-documented case history sites that showed vastly different performance during the earthquakes, from no liquefaction manifestation to extreme severity of liquefaction manifestation (Cubrinovski et al. 2018b). Based on the observed liquefaction manifestation, the 55 sites were classified into three groups: (i) sites that manifested liquefaction (soil ejecta) in both 4 September 2010 and 22 February 2011 earthquakes ('Yes-Yes' or YY-cases, shown with red symbols in Figure 13); (ii) sites that did not manifest liquefaction in the 4 September 2010 event, but manifested liquefaction in the 22 February 2011 earthquake ('No-Yes' or NY-cases; black symbols); and, (iii) sites that did not manifest liquefaction in any event during the 2010-2011 Canterbury earthquakes ('No-No' or NN-cases; green symbols).

4.2 Critical layer characteristics

To investigate the reasons for the dramatic difference in liquefaction manifestation (damage) amongst these sites, detailed field investigations were performed at each of the 55 sites using CPT, V_s and V_p cross-hole measurements. Using the CPT data, simplified soil profiles were determined for each site, in which characteristic soil layers were identified throughout the profile, and representative thickness, cone tip resistance ($\boldsymbol{q_c}$) and soil behaviour type index (I_c) were assigned to each layer. In the subsequent step, for each site the simplified soil profile and results from conventional triggering analyses were used to identify the critical layer within the profile or the layer that is the most likely to trigger and manifest liquefaction at the ground surface (Cubrinovski et al. 2018b).

Somewhat surprisingly, no significant difference between the properties of the critical layers of YY-sites (which manifested liquefaction in both earthquakes) and NN-sites (which did not manifest liquefaction in any earthquake) were found. Indeed, the YY-sites and NN-sites have identical critical layers in terms of their median

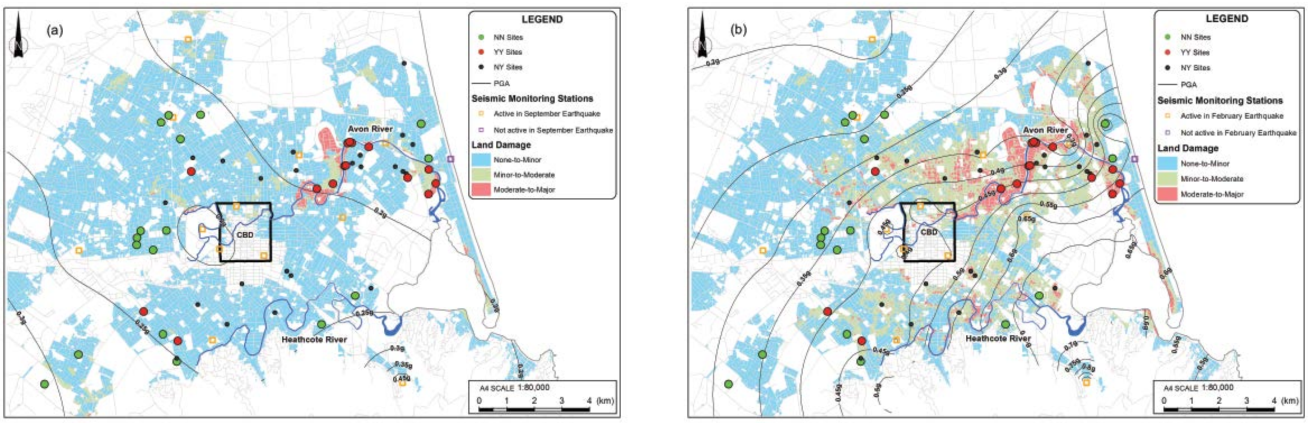

Fig 1a Fig 1b

Figure 13: Land damage maps indicating none-to-minor, minor-to-moderate, and moderate-to-major liquefaction in: (a) 4SEP2010 (Darfield) earthquake; (b) 22FEB2011 (Christchurch) earthquake; PGA contours and locations of 55 investigated sites are also shown (Cubrinovski et al. 2018b).

CPT values ($q_{c1Ncs} \approx 86$ and $I_c \approx 2.15$), and depth of the critical layer ($z_{CL} \approx 2.1$ m). Hence, the dramatic difference in observed liquefaction manifestation at the YY-sites and NN-sites cannot be explained through differences in the characteristics of their critical layers. Consequently, liquefaction manifestation predictions from simplified analyses were generally inconsistent with observations at NN-sites and YY-sites. Importantly, there was a systematic bias in the predictions by the simplified procedures. For NN-sites, liquefaction manifestation was overestimated for 91% of the cases, whereas for YY-sites, severity of liquefaction manifestation was underestimated for 37% of the cases.

4.3 Deposit characteristics

While YY-sites and NN-sites have identical characteristics of their critical layers, there are clear differences in their overall deposit characteristics. As shown in Figure 14a, sites that manifested liquefaction in both major earthquakes (YY-sites) are characterized by vertically continuous liquefiable soils in the top 10 m. These deposits are typically composed of a shallow silty sand in the top 2-3 m, overlying a vertically continuous 7 m to 8 m thick sand or fine sand layer.

The sites that did not manifest liquefaction in any of the 2010-2011 earthquakes (*NN*-sites), on the other hand, are characterized by highly stratified deposits comprising interbedded liquefiable and non-liquefiable soils, as shown in Figure 14b. A crust of non-liquefiable soil, horizontal 'grid' of non-liquefiable layers and consequent vertical discontinuity of liquefiable soils are key features of the NN-sites. At both YY- and NN-sites the water table is shallow, at about 2 m depth. However, V_p profiles indicated different degrees of saturation in the shallow part of the YY- and NN-deposits. Full saturation was implied in vertically continuous liquefiable sands at YY-sites for all soils deeper than 0.5 m below the water table, whereas the presence of non-liquefiable layers at NN-sites resulted in partial saturation for soils up to 3 m to 5 m below the water table.

To investigate more rigorously the response induced by the earthquakes for these two types of deposits, a comprehensive series of seismic effective stress analyses were performed (Cubrinovski et al. 2018b). In these response history analyses, key features of the soil response and liquefaction process such as build-up of excess pore water pressures, reduction in soil stiffness and strength, and redistribution of pore water pressures through water flow are rigorously modelled. Hence, the analyses account for interactions between layers through the dynamic response and water flow effects in liquefying deposits.

Two soil-column (1-D) models used in the analyses, representative of YY and NN deposits, are shown in Figure 15. The models have nearly identical critical layers in terms of liquefaction resistance (as represented by qc1Ncs) and location of the critical layer within the deposit. However, they have different deposit characteristics, i.e. vertically continuous liquefiable sands for the YY-deposit, and stratified deposit with liquefiable and non-liquefiable layers, for the NN-deposit. Details on the seismic effective stress analyses can be found in Cubrinovski et al. (2018b), whereas herein only key results and findings from these analyses are presented.

4.4 System response effects intensifying liquefaction manifestation

In the following, only key elements in the seismic response and evolution of liquefaction effects obtained in the analyses of the soil-column model for the YY deposit

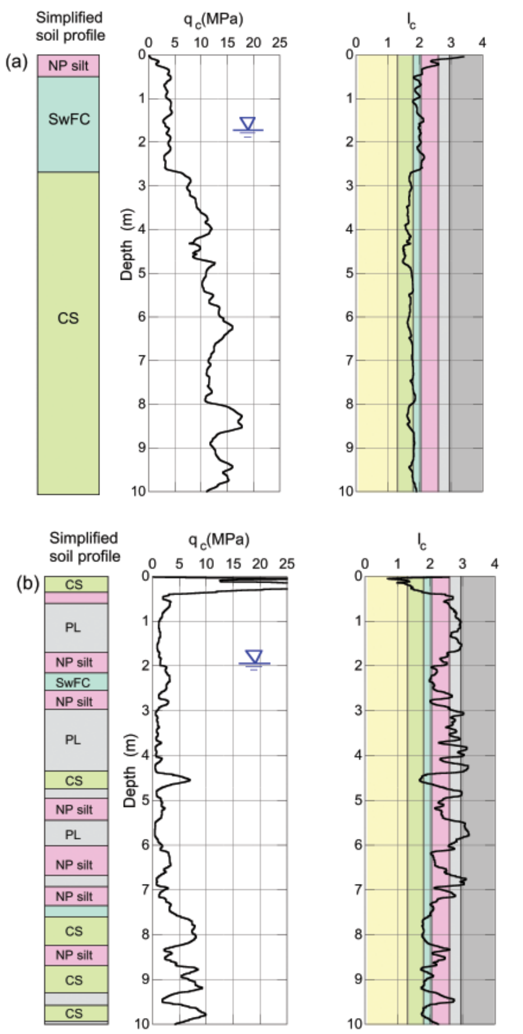

Figure 14: Characteristic CPT and soil profiles for: (a) sites that liquefied in both major earthquakes (YY); (b) sites that did not liquefy in any of the 2010-2011 earthquakes (NN); (Cubrinovski et al. 2018b).

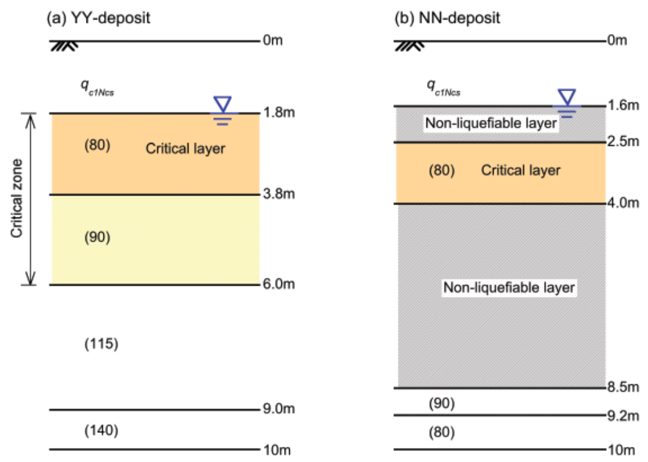

Figure 15: Soil-column models used in the seismic effective stress analyses representative of: (a) vertically continuous liquefiable soils of YY-deposits; (b) stratified liquefiable and non-liquefiable soils of NN-deposits (Cubrinovski et al. 2018b).

(Figure 15a) will be briefly examined. This deposit is characterized by a shallow and relatively thick critical layer (zone), from 1.8 m to 6 m depth, with low penetration resistance of q_{c1Nc} = 80 – 90. The critical layer is underlain by sand layers exhibiting higher and gradually increasing penetration resistance with depth. All top 10 m of the deposit are composed of liquefiable soils including the nominal crust above the water table.

Figure 16 shows computed time histories of excess pore water pressures throughout the depth of the deposit that reveal important liquefaction response features and interactions within the deposit. As indicated in Figure 16b and 16c, liquefaction was first triggered in the critical layer, at approximately t = 10 s on the computational time scale. The excess pore water pressures (u_E) build up rapidly and the critical layer quickly liquefies after only four seconds of strong shaking, approximately from t = 6 - 10 s. As illustrated in Figure 16d, the pore pressures also substantially increased in the denser sands beneath the critical layer, but these layers did not liquefy, as the excess pore water pressures did not reach the initial effective overburden stress (i.e. $u_E < \sigma'_{vo}$). However, the excess pore pressures in these deeper sand layers (u_E ~ 70 kPa) are substantially above the respective pressures in the critical layer, which range between 35 kPa and 65 kPa. This implies a significant inflow of water from the underlying deeper layers from 6 m to 10 m depth into the critical zone (layer), which will cause additional disturbance and instability through a prolonged and more severe fluidization of the already liquefied soils in the critical layer (zone). Finally, in Figure 16a a gradual increase in the excess pore water pressures is seen in the top part of the deposit, above the initial water table depth, due to upward flow of water from the liquefied critical layer towards the ground surface. These seepage-induced effects may further extend the liquefaction front towards the ground surface, and eventually result in a liquefied deposit in the top 6 m with substantial inflow of water from deeper layers from 6 m to 10 depth. These mechanisms involving vertical communication of excess pore water pressures and large volumes of water create system response effects that intensify severity of liquefaction and associated damage (manifestation) at the ground surface.

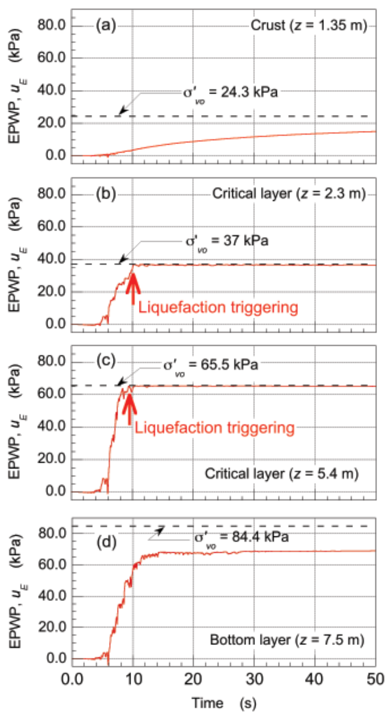

Figure 16: Excess pore water pressure time histories computed throughout the depth of YY-deposit that liquefied during the Canterbury earthquakes: (a) crust above water table; (b), (c) critical layer (zone); (d) deeper sand layers beneath the critical zone.

Figure 17 schematically illustrates the principal mechanisms that lead to severe liquefaction manifestation at the ground surface of YY sites. They involve: (1) early and rapid liquefaction of the shallow critical layer; (2) additional disturbance of the liquefied critical layer due to substantial inflow of water from the underlying layers that didn't liquefy but generated high excess pore water pressures; and, (3) seepage-induced liquefaction in shallow soils above the water table. These system response effects result in a strong and damaging discharge of excess pore water pressures in which liquefiable soils from the entire deposit contribute to, and intensify, the severity of liquefaction manifestation. These insights from seismic effective stress analyses may explain the severe liquefaction manifestation observed at the YY-sites after the earthquakes.

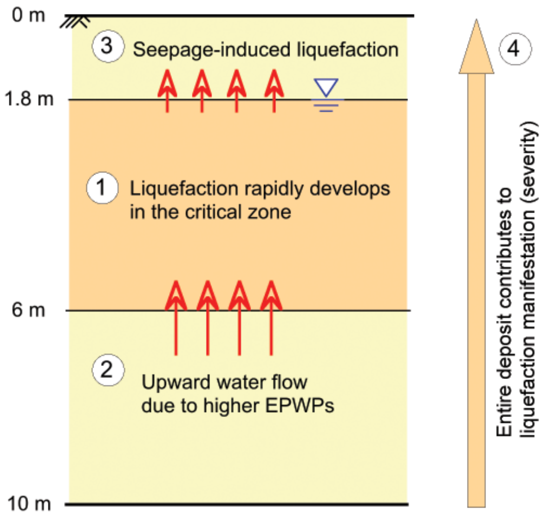

Figure 17: Schematic illustration of system-response mechanisms intensifying liquefaction manifestation (YY-sites) (Cubrinovski et al. 2018b).

4.5 System response effects mitigating liquefaction manifestation

The NN-deposit (Figure 15b) is also characterized by a shallow critical layer, from 2.5 m to 4 m depth, with equally low penetration resistance of q_{c1Ncs} = 80 as that of the YY-deposit. The key difference of this model is the presence of non-liquefiable layers and consequent vertical discontinuity of liquefiable layers. Another important feature of the NN-model is that a liquefiable layer of low penetration resistance is also encountered at larger depth, from 8.5 m to 10 m depth. Even though the water table is shallow at 1.6 m depth, there is a crust of non-liquefiable soil above the critical layer that prevents occurrence of seepage-induced liquefaction in near-surface soils. Hence, it is immediately obvious that mechanisms 2 and 3 intensifying liquefaction severity, identified for the YY-deposit (Fig. 17), cannot develop in the NN-deposit.

Figure 18 shows maximum shear strains and horizontal accelerations computed throughout the NN-model. Liquefaction occurred in both the shallow critical layer (2.5-4.0 m depth) and the deep low resistance layer (8.5-10 m depth), with liquefaction triggering occurring only slightly faster in the deeper layer. The higher shear strains for the deeper layer reflect this nuance in the liquefaction response of the NN model. A significant effect of the liquefaction in the deep layers is seen in Figure 18b, where sharp reduction in accelerations occurs from 10 m to 8.5 m depth due to liquefaction-induced softening of the deep loose layer. This 'base-isolation' effect has a profound influence on the overall deposit response, as it substantially reduces the inertial load for all soils above 8.5 m depth.

As mentioned previously, in the NN-deposits of interbedded liquefiable and non-liquefiable soils, partial saturation of soils was observed at depths within 3 m to 5 m below the water table, which implies that the shallow critical layer in the NN-deposit could be within the partially saturated zone. If an increased liquefaction resistance is assumed for the shallow critical layer due to partial saturation, then as illustrated in Figure 19, liquefaction will not occur in the shallow critical layer. Thus, combined effects of a reduced demand due to liquefaction in deep layers and an increased liquefaction resistance in shallow soils due to partial saturation may result in no liquefaction developing in the top 8.5 m of the deposit, and consequent absence of liquefaction manifestation at the ground surface.

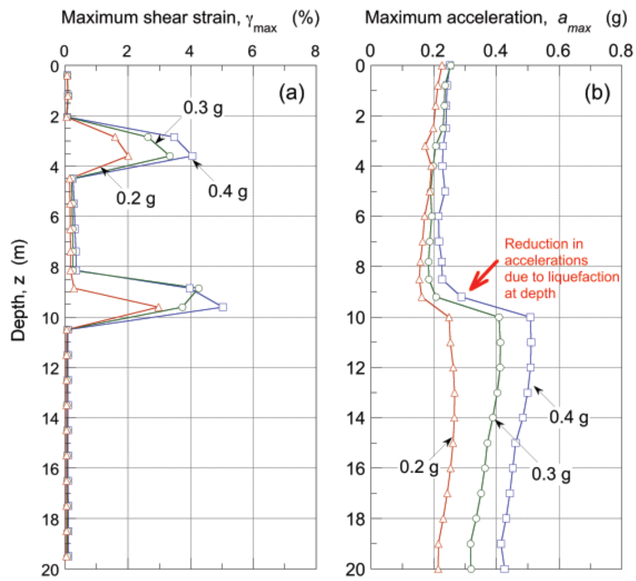

Figure 18: Computed response of the NN-model: (a) max. shear strains; (b) max. horizontal accelerations.

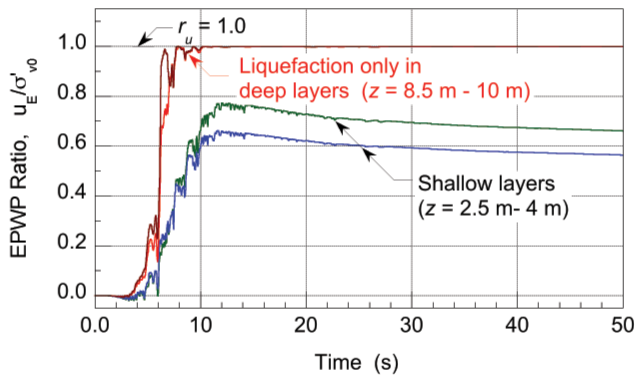

Figure 19: Excess pore water pressure time histories for shallow and deep liquefiable layers computed in analyses considering effects of partial saturation in shallow layers.

Again, as in the case of the YY-deposit, cross-interactions between layers significantly modify the deposit response and influence liquefaction manifestation at the ground surface. However,

unlike the mechanisms that intensify the severity of liquefaction for the YY-deposit, different mechanisms and interactions are at work for the NN-deposit, which mitigate development of liquefaction and its manifestation at the ground surface. Figure 20 illustrates key characteristics of the response and relevant mechanisms for the NN-sites. At these sites, liquefaction of the deep layer occurs first and produces the 'base-isolation' effect that substantially reduces the accelerations (and hence, the shear stresses) for all layers at shallower depths. This reduction in the seismic demand together with effects of partial saturation in the shallow parts of the highly-stratified deposit prevents occurrence of liquefaction in the shallow critical layer. As depicted schematically in Figure 20, this sequence of mitigating mechanisms effectively results in a non-liquefied 'crust' from the ground surface to 8.5 m depth, which prevents liquefaction manifestation at the ground surface, and is consistent with the absence of evidence of liquefaction at the ground surface of these sites after the Canterbury earthquakes.

The cascading mechanisms or system response effects work in opposite directions for the YY and NN sites with regard to liquefaction manifestation. For YY-sites, the system response effects increase the severity and consequences of liquefaction, whereas conversely, for the NN-sites, the interaction mechanisms mitigate liquefaction manifestation at the ground surface. In both cases, there are important cross-interactions between layers and different parts of the deposit through the dynamic response and water flow that substantially influence the development of liquefaction and even govern its manifestation at the ground surface. This clearly illustrates the need to incorporate system response effects in the assessment of liquefaction and associated damage.

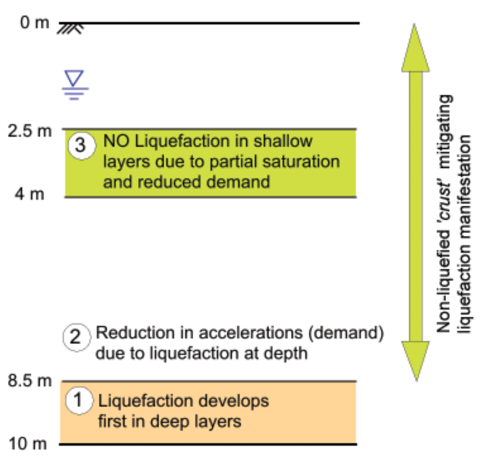

Figure 20: Schematic illustration of system response effects of liquefiable deposits that mitigate liquefaction manifestation (NN-sites).

4.6 Demand-dependent system response effects

To further elucidate system response effects of liquefying deposits, results from seismic effective stress analyses are briefly discussed for two NY-sites, i.e. sites that did not manifest liquefaction in the 4 September 2010 (SEP10) event, but manifested liquefaction in the 22 February 2011 (FEB11) earthquake.

Simplified soil profile, CPT data and analyses results for the NY-1 site are shown in Figure 21. The NY-1 site has general deposit characteristics similar to those of the YY-sites, with vertically continuous liquefiable soils in the top 10 m of the deposit. However, the NY-1 site exhibits some subtle but important differences in details as compared to the YY soil profile (shown in Figure 15a). The critical layer of the NY-1 site is relatively thin and deeper (z_{CL} = 3.7 m), and it has slightly higher cone tip resistance of q_{c1Ncs} = 100. Also, the q_{c1Ncs} values are generally higher throughout the entire NY-1 profile.

Computed maximum accelerations, excess pore water pressures at different time sections during the strong shaking, and maximum shear strains are shown in Figures 21d to 21g, for the SEP10 and FEB11 earthquakes. Analysis results show that the demand imposed by the SEP10 earthquake was not sufficient to trigger liquefaction in the deposit, with excess pore water pressures remaining well below the initial effective vertical stress (Figure 21e), which is consistent with the absence of evidence of liquefaction at this site, after this event.

In contrast, for the FEB11 earthquake, analysis results indicate that liquefaction occurs in the critical layer at an early stage of the strong shaking (t = 16.2 s, Figure 21f). The computed excess pore water pressures in the deeper layers below the critical layer are at slightly lower level than those of the critical layer. This implies that mechanism 2 which is associated with strong inflow of water into the critical layer (depicted in Figure 17) could not develop in this case. Instead, these pressures will dissipate through a gradual and steady upward water flow towards the ground surface, which is evident in the gradual increase in the excess pore water pressures in the layers immediately above the critical layer. These system response effects are consistent with the moderate liquefaction manifestation observed at this site after the FEB11 earthquake.

Soil profile characteristics and results from effective stress analyses for the NY-2 site are shown in Figure 22. The NY-2 site has general characteristics similar to the NN deposits. It comprises interbedded liquefiable and non-liquefiable soils, with a thin non-liquefiable crust

above the water table. It has a critical layer (zone) from 1.4 m to 3.0 m depth, but also liquefiable layers of low liquefaction resistance at greater depths, at 4.5 m and 7.5 m approximately.

Results from the seismic effective stress analyses for the SEP10 earthquake show that liquefaction developed in the layer from 4.4 m to 5.6 m depth at $t \approx 20$ s. At that time, the excess pore water pressures in the shallower layer at 2.1 m depth reached about 50% or less of the initial vertical effective stress, whereas no excess pore water pressures have developed in the

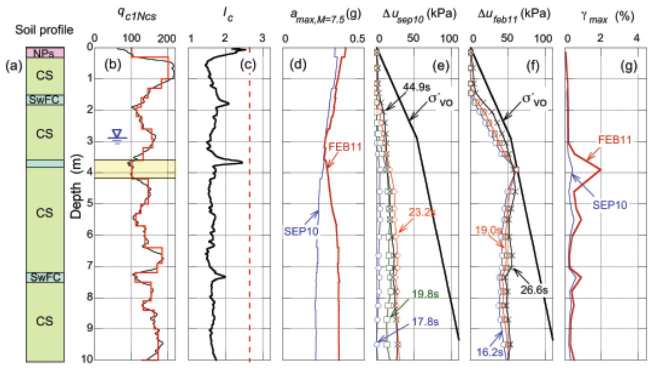

Figure 21: Characteristics of NY-1 deposit and computed response for SEP10 and FEB11 earthquakes: (a) simplified soil profile; (b) q_{c1Ncs} traces; (c) soil behavior type index, I_c; (d) maximum horizontal accelerations, $a_{max,M=7.5}$; (e-f) excess pore water pressures at different time sections; (g) maximum shear strains.

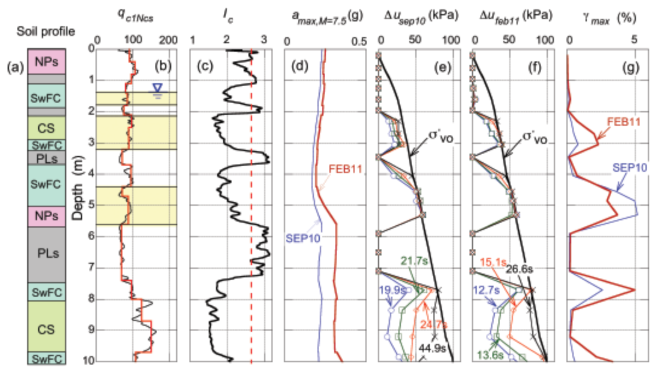

Figure 22: Characteristics of NY-2 deposit and computed response for SEP10 and FEB11 earthquakes: (a) simplified soil profile; (b) q_{c1Ncs} traces; (c) soil behavior type index, I_c; (d) maximum horizontal accelerations, $a_{max,M=7.5}$; (e-f) excess pore water pressures at different time sections; (g) maximum shear strains.

A key difference in the response induced by the FEB11 earthquake simulation is that liquefaction also develops in the shallower layer from 2.1 m to 3.2 m depth due to the higher seismic demand imposed by this event. Consequently, in this case, the non-liquefied crust is only 2 m thick and could not prevent liquefaction manifestation under the severe amplitudes of shaking imposed by the FEB11 earthquake. Again, these results are in agreement with the moderate liquefaction manifestation observed at the NY-2 site after the 22 February 2011 earthquake.

A close scrutiny of the computed maximum shear strains shown in Figure 22g provides additional insights into the important differences between the response characteristics of the NY-2 deposit for the two events. A large concentration (i.e. localization) of shear strains is seen in the liquefied layer at approximately 5 m depth for the SEP10 earthquake, whereas the strains are substantially smaller in the shallower part of the deposit. Conversely, relatively large shear strains are seen throughout the entire deposit, at four different depths, in the FEB11 earthquake simulation. This implies that while in both cases liquefaction is predicted to occur, the overall response of the deposit would be significantly different for the two events with regard to liquefaction manifestation and associated damage in near-surface soils.

The liquefaction response features exemplified by the above NY-sites analyses, further emphasize the need to consider cross-layer interaction and system response effects in the assessment of liquefaction and its consequences. They illustrate that some of the mechanisms intensifying liquefaction manifestation, shown in Figure 17, may not develop if the demand is not sufficient to activate those mechanisms. Conversely, if the demand is very high, then some of the mechanisms mitigating liquefaction manifestation, shown in Figure 20, may not be effective, as the relatively thin non-liquefiable layers and crust would be insufficient to prevent liquefied soils connecting vertically throughout the deposit, and eventually reaching the ground surface in the form of soil ejecta. These system response effects and demand-dependent mechanisms of interaction are currently the subject of a rigorous scrutiny through comprehensive series of seismic effective stress analyses.

5. CONCLUDING REMARKS

Soil liquefaction during earthquakes is a complex problem imposing numerous challenges in the engineering assessment. A large number of influencing factors are always in play that make a unique combination of contributions, and result in a particular set of response mechanisms, for given soil characteristics, ground conditions and earthquake excitation. Unweaving this complexity and identifying key factors and mechanisms that govern the liquefaction response and associated damage should be therefore one of the principal targets in the engineering assessment of liquefaction.

In order to evaluate effects of liquefaction and quantify damage to land and structures within performance-based design requirements, reasonably accurate estimates of transient and permanent ground displacements are needed for complex ground and soil-structure systems. The authors believe that these ambitious objectives in the assessment of liquefaction cannot be properly addressed without adequate consideration of material characterization, in-situ state characterization and system response effects of liquefiable deposits. Further advancements in this regard may bring a particular quality in the assessment through the ability to: 1) accurately discriminate between performances of different soils and ground conditions for a given earthquake excitation, and 2) accurately identify differences in performances of a given site for different earthquake excitations.

ACKNOWLEDGEMENTS

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Seismic Response and Soil-Pile Interaction Analyses for a Proposed Wharf Structure at the Lyttelton Port of Christchurch

ABSTRACT

A proposed expansion at the Lyttelton Port of Christchurch in New Zealand consists of the reclamation of approximately 34 hectares of new land and the construction of a new wharf supported on piles. Seismic response of the reclamation and soil-pile interaction have been key considerations throughout the development of the conceptual and the detailed design due to the close proximity of the project site to a number of active seismic source zones. The proposed wharf structure will be supported on 1.0 m to 1.5 m diameter steel pipe piles driven through the reclamation soils and the underlying natural soil deposits.

To address the above geotechnical seismic design considerations, one-dimensional (1D) equivalent-linear and nonlinear soil amplification (site response) analyses were performed using SHAKE2000 and D-MOD2000. This was followed by two-dimensional (2D) finite difference analyses using FLAC. The 1D analyses results were used as a benchmark for calibration of the 2D models. The results of the 2D analyses were used to assess the seismic deformations of the proposed reclamation and the seismic demand on the pile-supported wharf structure. This paper discusses aspects of input motion selection for the above analyses, and deconvolution and scaling of earthquake records. Shear strain and peak ground acceleration profiles obtained from the benchmark 1D analyses are compared to those from the 2D analyses. Computed deformations of the reclamation soils and the seismic demands (moments, shear forces and deformations) on the piles are also presented.

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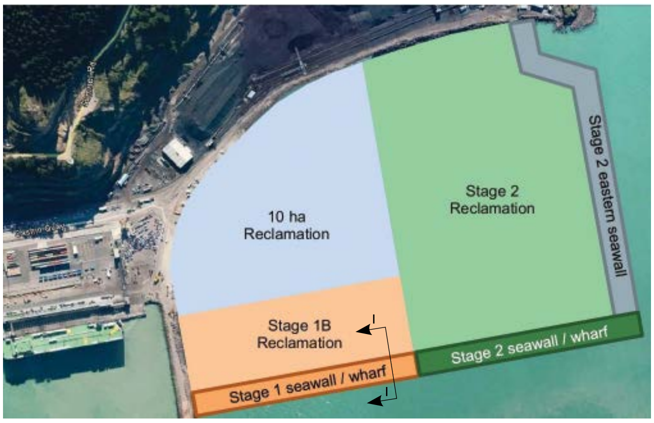

Figure 1: Layout of the reclamation stages and proposed wharves at the Lyttleton Port of Christchurch

1. INTRODUCTION

Lyttelton Port of Christchurch (LPC) is expanding the existing port at Te Awaparahi Bay. The expansion will include 34 hectares of new land including a new pile-supported wharf. To date, approximately 10 hectares have been reclaimed with the remaining reclamation to be completed in two stages. The rubble from Christchurch earthquakes was used to construct the existing reclamation. The new reclamation material will most likely be sourced from the nearby LPC owned quarry. Figure 1 shows the proposed reclamations and wharf at the port. The first stage will include the construction of a 350 m long traditional wharf with revetment, at a later time, supported on 1.0 m to 1.5 m diameter piles driven to about 100 m depth. Figure 2 is a cross-section through the reclamation at the wharf. Coffey Services (NZ) Limited (Coffey) has been involved in the project in a specialist geotechnical consulting role since early 2008. In 2014 Coffey was commissioned to provide a concept design (Antonopoulos, etc. 2017). In 2017 Coffey was engaged to carry out the detailed Stage 1 geotechnical design of the land reclamation, and seismic soil-structure interaction for the wharf piles.

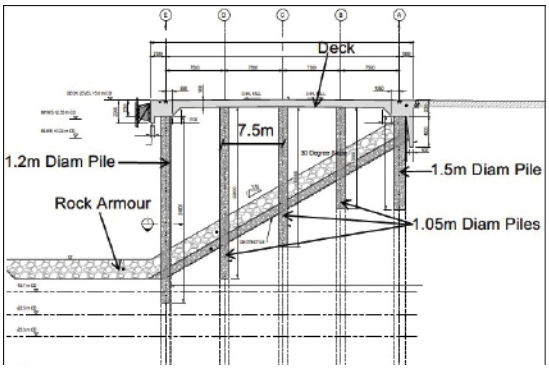

Figure 2: Cross-section through the reclamation at the location of the wharf and concept details of the future works

Seismic response of the reclamation and soil-pile interaction have been key considerations throughout the development of the conceptual design due to the close proximity of the project site to a number of active seismic source zones. For the detailed design phase, 1D benchmark soil amplification and 2D soil-foundation-structure-interaction (SFSI) analyses were performed to assess the reclamation deformations and wharf seismic performance. This paper discusses the approach followed, and the results of the seismic assessment as follows:

- Input motions from different seismic sources in consideration of the tectonic and seismic settings at the LPC;
- Deconvolution and scaling of the motions to obtain rock outcrop records;
- 1D benchmark soil amplification analyses for comparison with and calibration of the 2D time-history finite difference analysis; and
- 2D time-history finite difference analyses of the reclamation and the wharf structure to obtain the seismically induced deformations and bending moments along the wharf piles.

2. GROUND CONDITIONS AND DESIGN PARAMETERS

Soil conditions below the proposed reclamation generally comprise fine-grained relatively young marine sediments consisting of silt/clay and interbedded fine sand layers, underlain by weathered volcanic rock at about 150 m depth at the location of the proposed wharf. Coffey in the detailed design report (2017) presented the geotechnical model and basic geotechnical design parameters (also reproduced by Dai et al. (2018)). Figure 3 below is a simplified typical North-South section through the reclamation showing the soil units. Table 1 summarizes the relevant parameters of soil layers below the crest of the proposed Stage 1 reclamation slope considered for the 1D soil amplification analyses, as will be discussed in a later section. Soil units 2 to 6 consist of soft to firm and compact soils with shear wave velocities between 220 m/s and 270 m/s, noting that soil Unit 1 will be dredged prior to proposed Stage 1 fill placement. Soil Unit 8 consists of over-consolidated and very dense marine/colluvium deposits and talus with shear wave velocities between 380 m/s and 777 m/s.

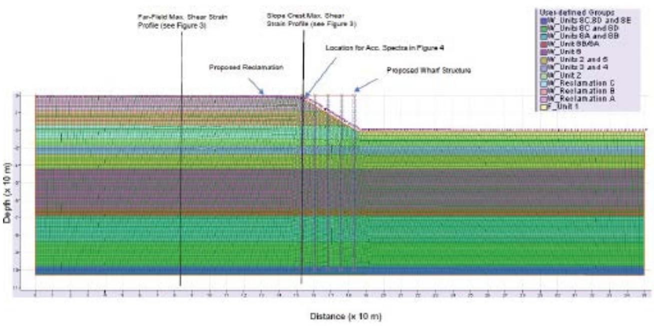

Figure 3: 2D analysis model of the North-South cross-section through the reclamation

3. METHODOLOGY

The methodology applied to the seismic design can be outlined in Figure 4. It comprises a series of ten steps that involve selection of earthquake events, calibration of the hysteretic material models, 1D soil amplification benchmark using both the equivalent linear and nonlinear methods, 1D soil amplification using FLAC, calibration of constitutive models and boundary conditions of the FLAC model using the 1D results, and finally 2D SFSI time history analysis with FLAC and further refinement/calibration.

The pseudo-spectral acceleration (PSA) with period, maximum horizontal ground acceleration (α_{max}), and maximum shear strain (V_s), propagating through the soil column from the corresponding 1D analyses are compared with each other and re used as criteria for selecting the preferred constitutive model and hysteretic damping for use in FLAC 1D and 2D analyses.

Soil Unit	Layer Thickness (m)	Bottom of Layer Depth below Ground Surface (m)	Unit Weight (kN/m³)	Shear Wave Velocity (m/s)	Shear Modulus Degradation and Damping Curve
Reclamation Fill	25	25	22.0	195-352	Darendeli (2001): PI = 0, 1.0 atm
Unit 2 – Marine Transition Layers	4	29	18.6	270	Darendeli (2001): PI = 15, 4.0 atm
Unit 3 – SAND	3	32	17.9	240	
Unit 4 – Silty CLAY	2	34	17.2	230	
Unit 2 – Marine Transition Layers	3	37	18.6	270	
Unit 5 – SAND and SILT	5	42	18.8	270	
Unit 6 – Silty CLAY	23	65	18.2	220	
Unit 8A/8B – SAND, Sandy Silt and Silty Clay	4	69	20.0	380	
Unit 8B – Silty Clay	9	78	20.0	526	
Unit 8A – SAND and Sandy Silt	6	84	20.0	485	
Unit 8C – GRAVEL and Sandy GRAVEL	11	95	20.0	526	Darendeli (2001): PI = 0, 4.0 atm
Unit 8D – SAND and SILT	3	98	20.0	485	Darendeli (2001): PI = 15, 4.0 atm
Unit 8C – GRAVEL and Sandy GRAVEL	5	103	20.0	777	Darendeli (2001): PI = 0, 16.0 atm
Unit 8D – SAND and SILT	18	121	21.0	777	Darendeli (2001): PI = 15, 16.0 atm
Unit 8E– GRAVEL	31	152	22.0	777	Darendeli (2001): PI = 0, 16.0 atm
Bedrock	-	-	22.0	800	EPRI (1993) for Rock 251 ft to 500 ft

Table 1: Typical soil profile below the crest of the proposed reclamation slope.

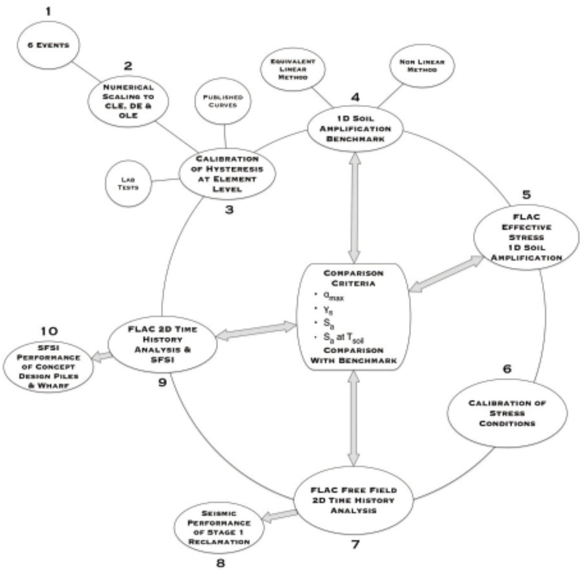

Figure 4: Outline of the seismic design methodology

4. 1D BENCHMARK SOIL AMPLIFICATION ANALYSIS

4.1 Analysis Methodology

Equivalent-linear (EL) and nonlinear (NL) 1D soil amplification analyses were performed for benchmark comparison with the 2D model to be presented in a following section. The 1 analyses were performed using the software SHAKE2000 (Ordóñez, 2012). Darendeli (2001) shear modulus degradation and damping curves were used for soil units. EPRI (1993) shear modulus degradation and damping curves were used for rock (see Table 1).

The NL analyses were performed using the software D-MOD2000 (Matasović and Ordóñez, 2011). D-MOD2000 uses the MKZ non-linear stress-strain model (Matasovic and Vucetic, 1993). The MKZ model input parameters were obtained by curve fitting to the Darendeli (2001) curves and using the Automatic Linear Data Optimization routine that is embedded in D-MOD2000. In addition to the hysteretic damping accounted for by the MKZ model, D-MOD2000 uses Rayleigh damping to account for energy dissipation at very low shear strains. Simplified Rayleigh damping was used with target damping ratios of 1% and 2% (Matasović and Ordóñez, 2011). D-MOD2000 analyses performed using damping ratio of 1% resulted in unrealistically high maximum accelerations (>10 g) at few depths and the program prompted an error message, a likely indication of numerical instability. These errors were not encountered for the analyses performed using damping ratio of 2%, and therefore, damping ratio of 2% was used in the NL model.

4.2 Input Motions

Four types of earthquake events were considered for the analysis as shown in Table 2: near-field, regional, fault forward directivity, and Alpine Fault with basin effects. Deconvolution was performed to obtain firm ground, defined as Site Class B, rock outcrop motions as input for the soil amplification analysis model. Details related to processing the input motions are as follows:

- The Christchurch February 2011 and the Darfield 2010 Site Class B (Rock) records were used as rock outcrop motions (i.e., without deconvolution);
- The Christchurch June 2011 time histories used were recorded on a reclamation site approximately 2.5 km west of the proposed Te Awaparahi Bay Stage 1 reclamation. Deconvolution of these records was performed to obtain the corresponding rock outcrop motions, which were used then used as input for this analysis at bedrock;
- The Lixouri 2014 record was recorded at soil surface for a Site Class D soil profile. The deconvolved record at the base of the soil column was used as input motion for the analyses;
- The Kaikōura 2016 TFSS records were recorded on an old reclamation at the Port of Wellington. Deconvolution was performed based on soil profile and properties interpreted according to Cubrinovski et al. (2017). Anticipated soil conditions consist of reclamation fill, underlain by less than 4 m of marine sediments, underlain by very dense/stiff (SPT N > 50 blows/ft) Wellington Alluvium, which was taken as the bottom layer for deconvolution soil column. The depth to bedrock at the Port is not known but is likely more than 200 m. In consideration of the similarity in interpreted shear wave velocity of the Wellington Alluvium and the LPC Unit 8B – Silty Clay, the deconvolved Kaikōura 2016 records were later used for LPC 1D response analysis as input motions at the top of Unit 8B – Silty Clay.

The rock outcrop motions, except for Group D, were linearly scaled to the Class B 475-year peak ground acceleration (PGA) of 0.56g based on the GNS 2015 report "Seismic hazard assessment of Lyttelton Port incorporating additional site classes" (GNS, 2015). As discussed above, the Kaikōura 2016 (Group D) records were used as input motions at the Unit 8B – Silty Clay. The input records were scaled to a horizontal acceleration of α_h=0.43g which corresponds to average peak horizontal acceleration of the records for the other groups, calculated at the top of Unit 8B.

Earthquake Group	Earthquake Name	Station/Location/ ID	Record ID for Analysis	Component	Site Class
Group A Near Field	Christchurch Feb. 2011	235142_LPCC_V2A	GA1	N10W	B
			GA2	S80W	
	Christchurch June 2011	010100_LPOC_V2A	GA3	S83W	D
			GA4	S07E	
		022049_LPOC_V2A	GA5	S83W	
			GA6	S07E	
Group B Regional	Darfield 2010	163541_LPCC_V2A	GB1	N10W	B
			GB2	S80W	
Group C Fault Forward Directivity	Lixouri 2014	Greece	GC1	EW	D
Group D Alpine Fault with Basin Effects	Kaikōura 2016	110330_TFSS_20_V2A	GD1	N00E	C
			GD2	N90W	

Table 2: Earthquake input records.

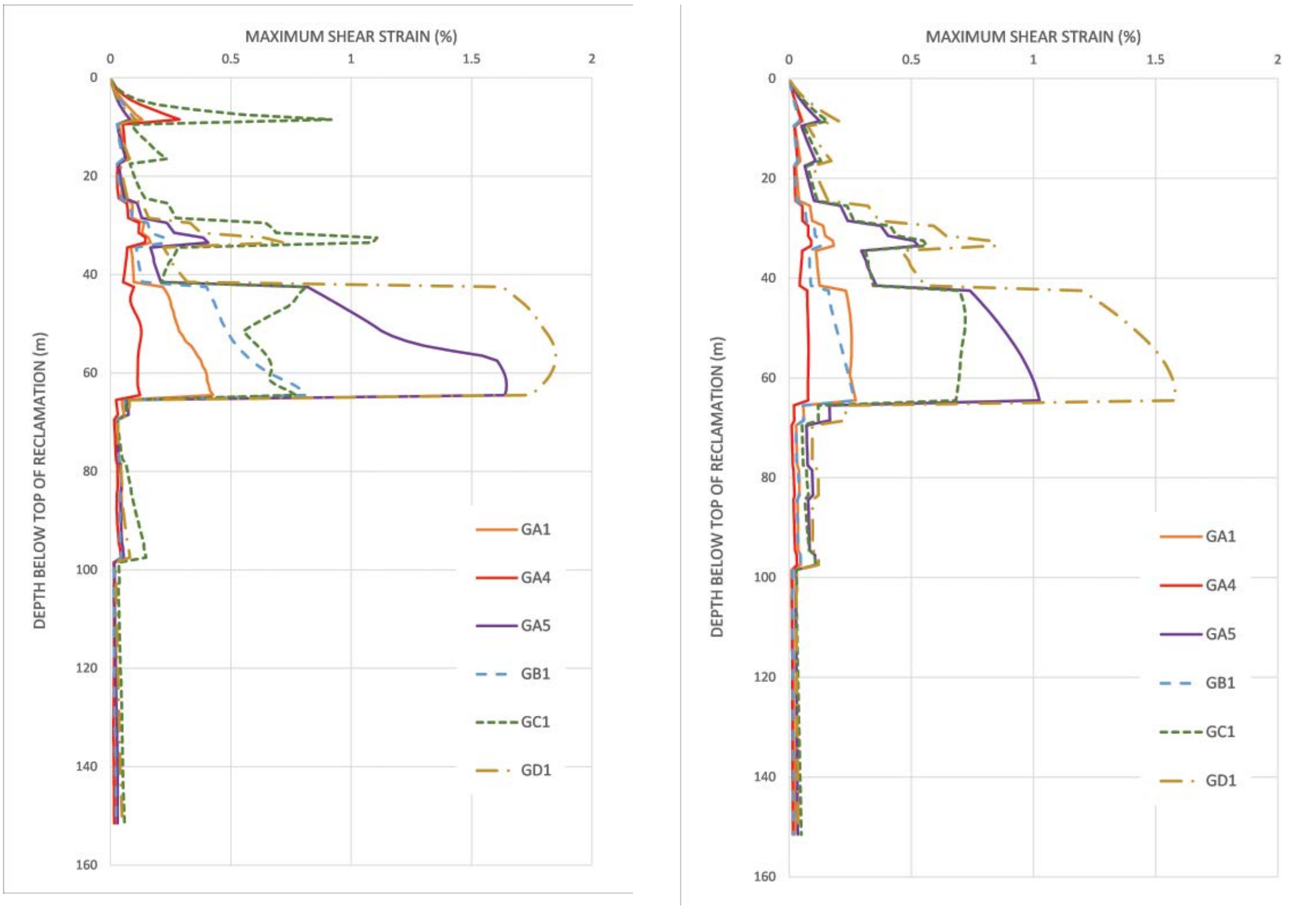

Figure 5: Maximum shear strain, (γ_s), profiles from SHAKE2000 (Left) and D-MOD2000 (Right)

4.3 1D Analysis Results

Maximum shear strain and horizontal acceleration profiles from SHAKE2000 and D-MOD2000 are shown in Figures 5 and 6 respectively, for the most critical records (GA1, GA4, GA5, GB1, GC1 and GD1) with the highest peak accelerations and shear strain responses for their respective earthquake groups. Maximum shear strains were very similar for both EL and NL analyses. The highest shear strains (up to 2%) are within soil Unit 6 (silty clay) with relatively low shear wave velocity of 220 m/s. The α_{max} values at ground surface obtained from NL analyses ranged from 0.13g to 0.37g compared to 0.16g to 0.33g for EL analyses. The Lixouri 2014 (GC1) and Kaikōura 2016 (GD1) records resulted in the highest surface α_{max}. Overall, the results indicate de-amplification of the bedrock outcrop input motions.

5. 2D TIME-HISTORY SEISMIC ANALYSIS

5.1 Analysis Methodology

Dynamic time-history analyses were performed using the finite difference software FLAC (version 7.0, Itasca). The model captures the interaction between the wharf and the soil. A north-south cross-section shown in Figure 3 was considered. Soil stratigraphy was simplified by considering horizontal layers and by modeling soil layers with similar shear wave velocities as single layers. To reduce computational time, the FLAC model was truncated within soil Unit 8 with shear wave velocity of 777 m/s. This is not expected to impact the results due to the high shear wave velocity that is similar bedrock (800 m/s). Details of the FLAC modelling are as follows:

- The Mohr-Coulomb constitutive model was used for the soil units to simulate the shear strength (excess pore water pressure generation was not explicitly modelled);
- Hysteretic damping for all soil elements was applied and was primarily captured using the FLAC sigmoidal model, sig4. The selected sig4 model parameters were obtained from FLAC dynamic manual based on a numerical fit to the upper range Seed and Idriss (1970) for sand and upper range Sun et al. (1988) for clay;
- For the analyses carried out, a Rayleigh damping in FLAC with ε_{min} of 2.2% at frequency f_{min} of 3.0 Hz was applied to all soil elements, which resulted in a close match between FLAC and 1D benchmark analysis results. Sensitivity analyses were also

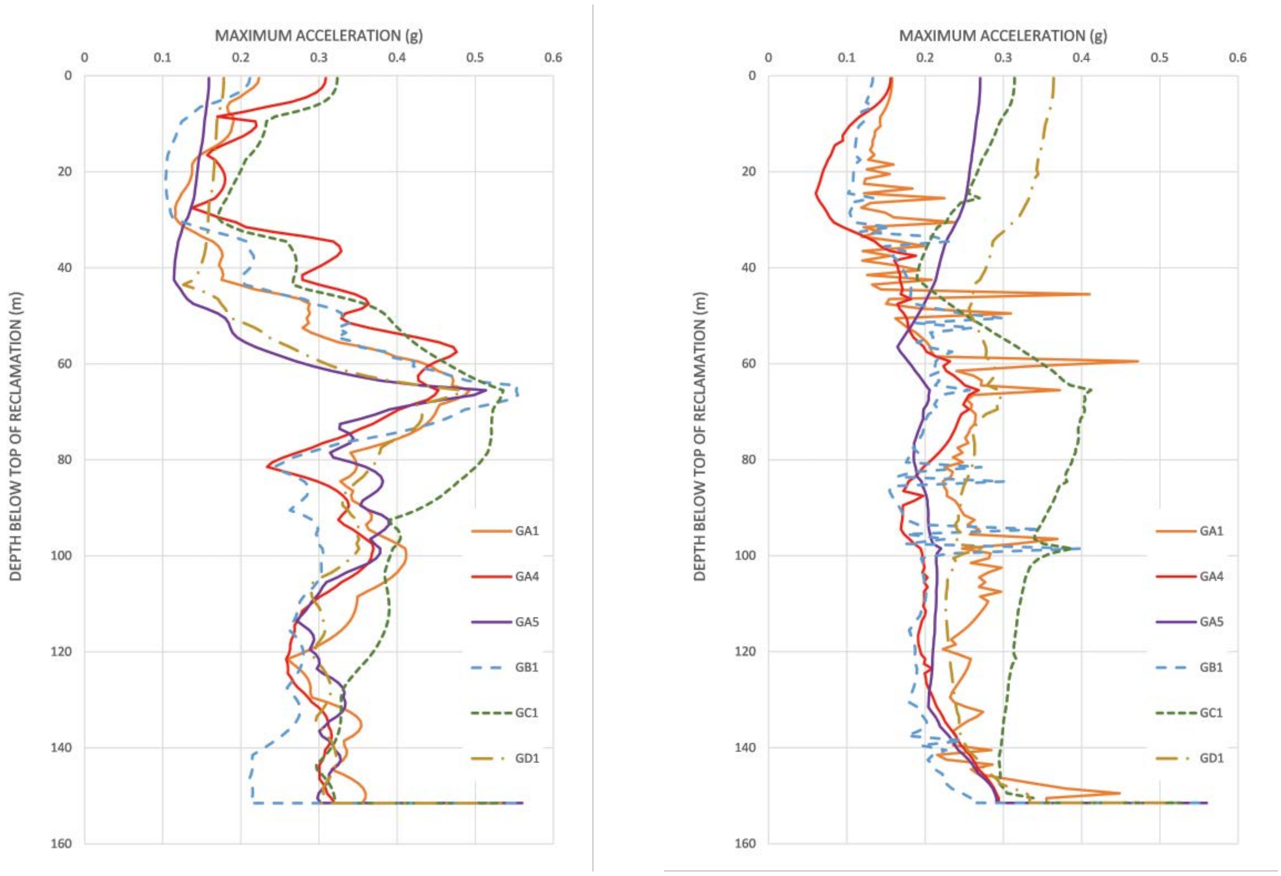

Figure 6: Peak horizontal ground acceleration, α_{max}, profiles from SHAKE2000 (Left) and D-MOD2000 (Right)

performed to assess the impact of Rayleigh damping by conservatively not including Rayleigh damping. The deformations and moments for the case without Rayleigh damping were higher than the base analysis case (i.e., with Rayleigh damping);

- Coupled fluid-mechanical effective stress analyses were performed;
- 1D FLAC analysis was also performed for comparison with the benchmark SHAKE2000 and D-MOD2000 analyses discussed in the previous section; and
- The most critical earthquake records were found to be the Kaikōura 2016 and the Lixouri 2014. They both exhibit similar acceleration, shear strain, and spectral acceleration results, however, the Kaikōura 2016 event is over 60s long whereas the Lixouri 2014 lasts for only 20s. The characteristics and especially the duration of the latter was a primary criterion for selecting it to proceed with the final 2D SFSI analyses, and its results are presented below.

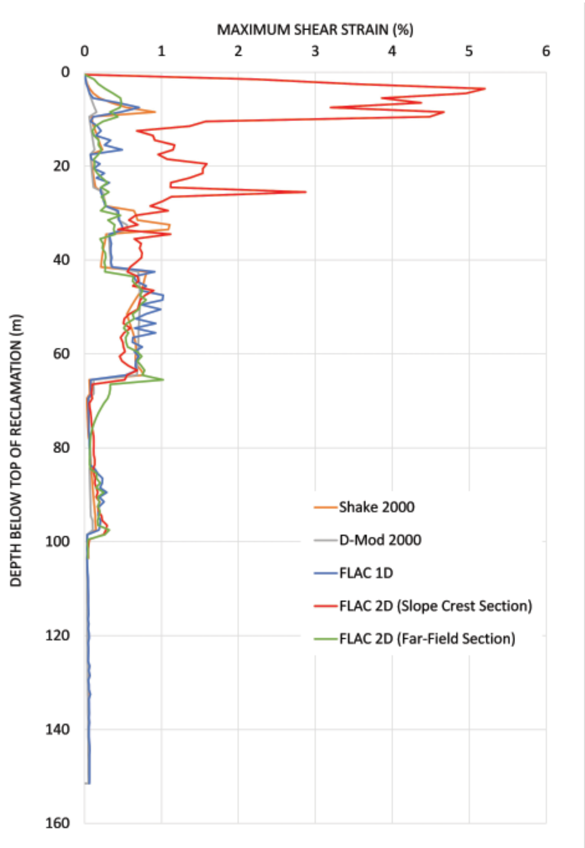

Figure 8: Maximum shear strain, (Y_s), profiles from FLAC 1D and 2D analyses

5.2 FLAC Analysis Results

FLAC large strain deformations and horizontal displacements are shown in Figure 7. Figure 8 shows the maximum shear strain profiles obtained from SHAKE2000, D-DMOD2000, FLAC 1D and 2D analyses. A reasonable match is noted for depths below approximately 30 m. Shear strains obtained from the FLAC 2D analyses (slope crest section) for depths shallower than approximately 30 m correspond to the reclamation slope deformations.

Computed lateral deformations and bending moments for the wharf piles are presented in Figure 9. Pile head deformation of about 160 mm were computed. The highest bending moments are for the landward pile with the largest diameter of 1.5 m.

6. CONCLUSIONS

1D and 2D soil amplification analyses were performed for the proposed reclamation and wharf at LPC to assess the seismically-induced deformations of the reclamation and seismic demands on the wharf piles. The 1D soil amplification analyses were undertaken using SHAKE2000 (equivalent linear) and D-MOD2000 (nonlinear). The 2D seismic analyses that incorporate the wharf structure were performed using FLAC.

Near-field, regional, fault forward directivity, and Alpine Fault with basin effects earthquake records were selected as excitation for the above modelling. Deconvolution was performed to obtain the bedrock outcrop motions to use as input for all the analysis.

The results of the 1D analyses indicate de-amplification of the rock outcrop accelerations. The far-field (i.e., away from the slope crest) FLAC 2D maximum shear strains are

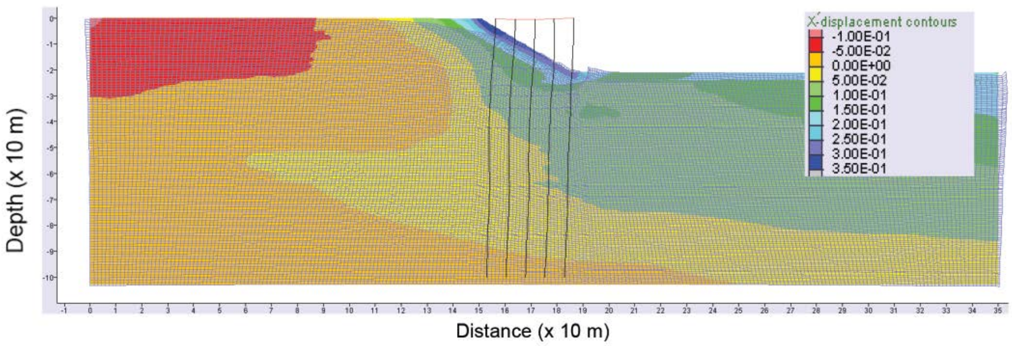

Figure 7: FLAC 2D large strain deformations and horizontal displacements (in m)

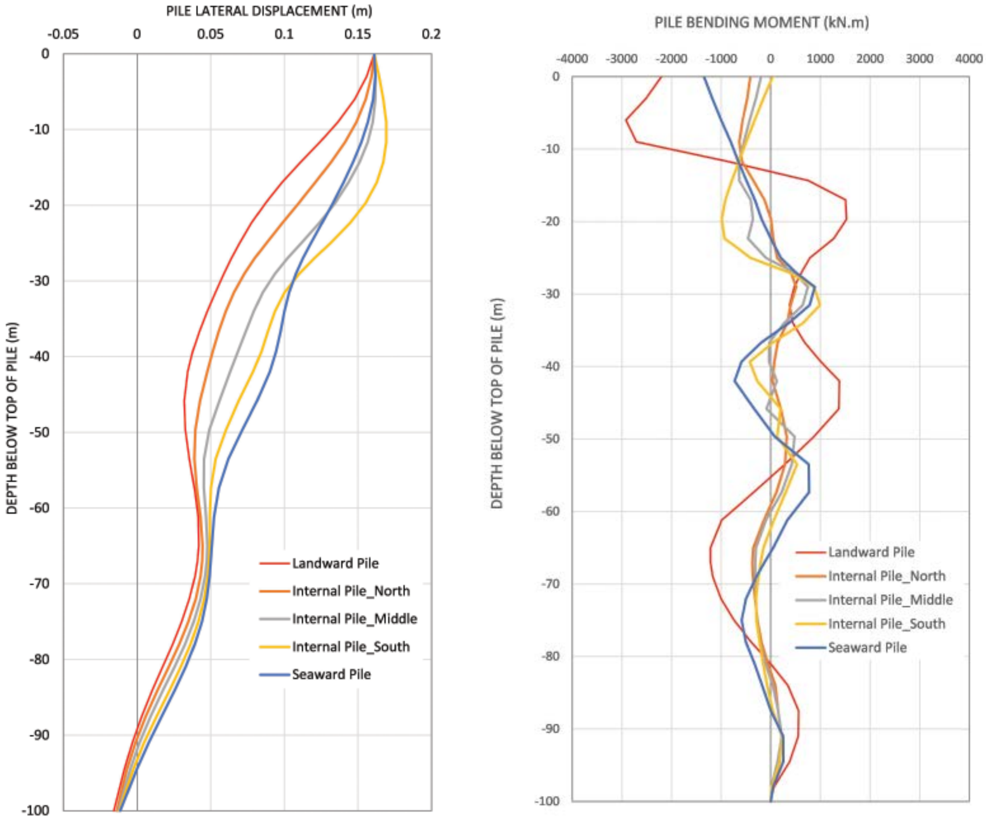

Figure 9: Horizontal (lateral) displacements (left), and bending moments of the Wharf piles from FLAC 2D analyses (right)

very similar to those from SHAKE2000 and D-MOD2000. The noted close agreement between the 1D and 2D results indicate the adequacy of the modelling approach followed.

Shear strains near the slope crest obtained from FLAC 2D for depths shallower than approximately 30 m were higher than those from for the far-field section as they correspond to the reclamation slope deformations. Furthermore, the results showed that highest wharf pile bending moments are for the landward pile with the largest diameter of 1.5 m.

7. ACKNOWLEDGEMENT

A number of Coffey staff have worked cooperatively on this project, contributing to the successful completion of the work, including Andrew Hutchinson and Chris Thompson. The authors are grateful to the staff who contributed to this project. The authors also wish to thank Lyttelton Port of Christchurch for their approval to publish this paper.

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Seismic Assessment of Existing Buildings Geotechnical Considerations

INTRODUCTION

The document, "*The Seismic Assessment of Existing Buildings*" July 2017 (The Guidelines) provides a technical basis for engineers to carry out seismic assessments of existing buildings in New Zealand. *The Guidelines* support seismic assessments for a range of purposes, including whether or not a building is earthquake prone in terms of the Building Act 2004. Section C4 "*Geotechnical Considerations*" of The Guidelines stresses the need for foundation (geotechnical) aspects to be considered as part of the seismic assessment. This paper summarises the main points presented in Section C4 and particularly emphasises the need for structural and geotechnical engineers to work collaboratively in undertaking a seismic assessment. The intent is to provide an overview and to emphasise key points, but not to replace the need to read *The Guidelines* and attend relevant courses.

Does this sound familiar?

The structural engineer; "*All I want is a report confirming its subsoil class C*".

And the geotechnical engineer; "Liquefaction is triggered at 30%ULS shaking. The building therefore must be 30%NBS."

The objective of Section C4 is to get structural engineers to work in conjunction with geotechnical engineers to identify and assess geotechnical aspects which could influence the behaviour of the building; i.e. *it's more than just subsoil class.*

And for geotechnical engineers to work in conjunction with the structural engineers to jointly develop an understanding of the interaction between the soil and the structure; i.e. it's not all about the ground (eg triggering of liquefaction), it's about how the structure behaves as a consequence.

The objective is for us to work together so that geotechnical aspects are considered and an understanding is developed of how, or if, the geotechnical aspects effect the overall behaviour of the building.

KEY PRINCIPLES

Key principles of a seismic assessment include; that the focus of the assessment is on stability of the structure and life safety. As part of that we need to consider the behaviour of the foundations and assess whether or not that foundation behaviour could influence structural stability and life safety.

In ultimate limit state design it is normal to apply a load and resistance factored design approach. The foundation capacity is reduced by a strength reduction factor to provide some reliability and to limit deformations. The assessment approach is quite different. We are interested in the probable behaviour of the foundations and the building. Parameters are selected accordingly, and strength reduction factors are not applied. A displacement based approach is likely to be applied. The Guideline suggests simplified elastic plastic models be applied.

Geohazards to be considered in the seismic assessment of a building include;

- lateral and vertical load/displacement behaviour of the foundations,
- the possibility of liquefaction and its associated effects,
- slips or retaining wall movement removing support to foundations.

These geohazards occur beneath the building footprint and are to be considered in assessing the %NBS rating of the building. Retaining wall displacements, slips and rockfall originating from beyond the building with the potential to impact the building are also to be considered, but do not influence the %NBS rating of the building. This is in the same way that the stability or %NBS rating of a neighbouring building does not influence the %NBS rating of the building being assessed.

Stuart Palmer

Stuart Palmer is a Technical Director-Foundation Earthquake Engineering with Tonkin & Taylor Ltd. He has more than 30 years' experience in foundation engineering with a specialist interest in earthquake engineering and soil structure interaction. Stuart is actively involved in the seismic design and assessment of buildings in collaboration with structural engineers. He was an author of the Seismic Assessment Guidelines Part C4- Geotechnical Considerations.

STEP CHANGE

The principle of step change is another key aspect of an assessment, and particularly in relation to geotechnical considerations.

Soils can be subject to an abrupt and large reduction in strength with an increase in loading, deformation or earthquake shaking. This is a geotechnical step change. Examples of this include liquefaction, brittle failure of a rock anchor or a brittle underslip. An important aspect of an assessment is to identify potential geotechnical step changes and whether or not they could result in instability of the building and significant life safety hazard ie, whether or not they could result in a structural step change. The geotechnical engineer needs to assess;

- the capacity of the foundation before the step change,
- the %ULS shaking or deformation which triggers that step change, and
- the residual capacity of the foundation after the step change.

In the assessment the structural engineer is to allow for the geotechnical step change occurring at a %ULS shaking or deformation of ½ of that of the trigger. This is to provide some resilience in the %NBS rating. Assessment of the buildings behaviour post the geotechnical step change could become critical to %NBS rating of the building. The structural and geotechnical engineers must jointly assess the stability of the building allowing for the residual foundation capacity. For example if liquefaction is assessed to be triggered at 60%ULS shaking AND it is assessed that with the residual behaviour of the foundations the building would be unstable, potentially endangering life, then the associated score would be 30%NBS.

COLLABORATION AND ITERATION.

Figure 1 below is taken from Section C4 of *The Guidelines*. It outlines the process recommended for building assessment including geotechnical considerations. The circled portion of the flow diagram is highlighting an iterative process of assessment and review to develop an understanding of the behaviour of the building including its foundations. Each iteration includes conversations between the structural and geotechnical engineers. The objective is to focus any geotechnical work on what matters. That is what matters for the stability of the structure and life safety. The following subsections illustrate this further.

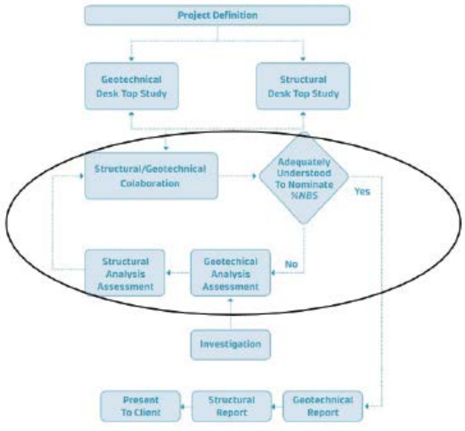
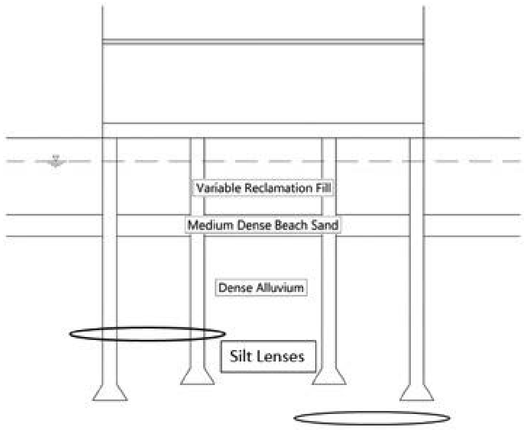


Figure 1: Collaboration and iterations during a seismic assessment.



Geotechnical Issues

- **Subsoil Class**
- **Liquefaction of reclamation fill**
- **Vertical Load/Displacement Behaviour**
- **Lateral Load / Displacement Behaviour**

Figure 2: Ground Model

GROUND MODEL

The first step is for the structural engineer to develop a general understanding of the structures form, load paths and potential structural weaknesses. And for the geotechnical engineer to develop an understanding of the ground model and identify potential geotechnical issues, on the basis of a desk top study. Certainly not to produce a geotechnical report at this stage. A simple sketch of the likely ground model and a list of potential geotechnical issues is all that is required. Figure 2 above presents an example.

The structural and geotechnical engineers then meet and compare notes. They work through each of the geotechnical issues and identify which of them could be material to the assessment of the structure. And for those which could be material what additional information is necessary to assess them further. The output of the meeting would be agreement as to what further geotechnical input is required for the next iteration. A useful question at the end of the meeting is how would we categorise the assessment; dominated by the structure, by interaction between the soil and the structure, or dominated by geotechnical considerations. This will help focus the level of further geotechnical input required. A majority of assessments are likely to be structurally dominated such that little or no further geotechnical input is required.

FOUNDATION BEHAVIOUR

A possible outcome from that first meeting is that the load displacement behaviour of the foundations could be critical to the behaviour of the structure and deserves further consideration. Figure 3 describes how that further consideration could be undertaken by the geotechnical engineer and communicated to the structural engineer.

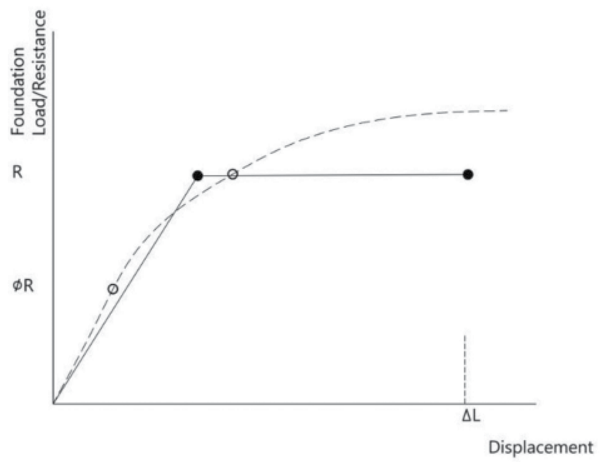


Figure 3: Modelling Foundation Behaviour

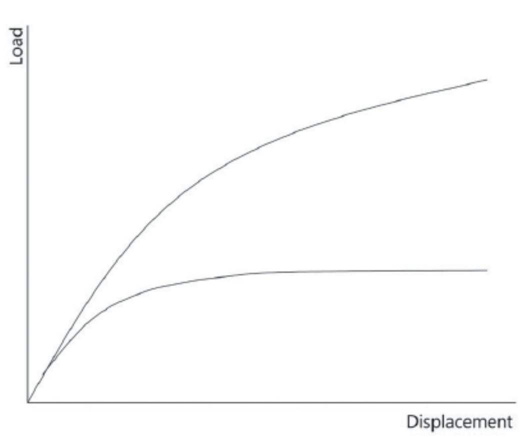


Figure 4: Managing Uncertainty

The first thing the geotechnical engineer needs to consider is whether the load displacement behaviour is likely to be ductile i.e. resistance increases with displacement; like indicated by the dashed line in Figure 3. If non-ductile or brittle behaviour is possible then a step change needs to be considered.

The next thing the geotechnical engineer needs to consider is how to model the foundation behaviour for application to a structural assessment. Keep it simple. The Guideline suggests a simple elastic plastic model as indicated by the solid line in Figure 3. The geotechnical engineer needs to nominate the limiting resistance R. This would be assessed as it would be for design but a strength reduction factor is not applied. Section C4 of The Guideline and NZGS Module 4 provides further guidance.

MANAGING UNCERTAINTY

The uncertainties we need to deal with in geotechnical engineering are large. In assessment the uncertainties are greater than in design. The presence of the existing building constrains access for investigations or makes them very expensive. Often the depth and size of foundations are unknown. Consequently as part of the iterative and collaborative assessment process the structural and geotechnical engineers need to undertake sensitivity checks to identify any critical issues in relation to stability of the structure and life safety. The geotechnical engineer needs to provide some "what if" scenarios as indicated on Figure 4 and the structural engineer needs to test these in terms of impact on the structure. If a possible foundation behaviour scenario is found to be critical then further assessment or investigation may be required. The message to the geotechnical engineer is that an expensive investigation programme may not be necessary.

CONCLUSION

Don't focus on producing a geotechnical report. Encourage structural and geotechnical engineers to work together to identify geohazards which could influence structural stability and life safety. This is going to require us to talk to each other right from the very beginning of a project. Use sketches of the structure, the ground model and of load displacement behaviours to aid our communication. Once the critical issues have been identified and assessed the conclusions can be presented in a report.

Application of Site-Specific Earthquake Geotechnical Engineering and Soil Foundation-Structure Interaction in Christchurch

ABSTRACT

A site-specific geotechnical engineering approach was proposed as part of a seismic retrofit design of a multi-storey building in Christchurch CBD (Central Business District). Various analytical techniques were utilized including Probabilistic Seismic Hazard Analysis (PSHA), development of time histories, Site Specific Response Analysis (SSRA), and Soil Foundation-Structure Interaction (SFSI). Miyamoto's New Zealand seismic hazard model was used as the basis for computing PSHA and deaggregation. The hazard model is developed using open-source Seismic Hazard Analysis (SHA) software (OpenQuake 2.1.0). This model includes active faults, area sources, and time-dependent point source model based on the modified-Oomori Law. It was compared with the New Zealand National Seismic Hazard Model (NZSHM) which form the basis for NZS 1170.5:2004. A suite of eleven spectrally-matched, single-component horizontal earthquake acceleration time histories, was developed for each of the 25-year and 500-year return period loading conditions. Then, the time histories were propagated from Riccarton Gravel to ground surface using non-linear, one dimensional (1D) total stress analysis in DEEPSOIL 6.1. A medium-rise multi-storey building with a 3.0m deep basement was modelled and analysed in ETABS software using non-linear elements. Dynamic soil properties were derived from the analysis of geotechnical data acquired from the site, this included shear wave velocity (Vs) profiles from downhole tests. The result from the geotechnical analysis can be used as the input for models incorporating soil foundation-structure interaction with further refinement and verification. This approach requires more effort but does provide an improved and realistic result of the building's behaviour including considerations of the influence of soil foundation-structure interaction.

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INTRODUCTION

For the majority of practices in New Zealand, the most widely adopted methodology to calculate earthquake demand to building structures involves using the hazard factor, Z in combination with site classification as per NZS 1170.5:2004. The elastic site spectra for horizontal loading as per NZS 1170.5 is simplified using spectral shape coefficient based on site classification of Soils A, B, C, D, and E without considering site specific effect, i.e. non-linear behaviour of the soil and liquefaction. The Z factor map in NZS 1170.5 is generalised in an effort to cover all New Zealand but limited to an earthquake database up to the year 2000 while several mid to high-rise buildings in Christchurch were damaged from earthquake shaking and/or soil liquefaction during the 2010-2011 Canterbury Earthquake Sequence (CES). In a lot of designs, the geotechnical and the structural engineer fail to work in a collaborative manner, and ultimately, the intricacies in soil foundation-structural-interaction are not captured and/or explored.

An example of the current application of site-specific geotechnical earthquake engineering is illustrated in Figure 1. The Probabilistic Seismic Hazard Analysis (PSHA), development of time histories, seismic 1D Site Specific Response Analysis (SSRA), and Soil Foundation-Structure Interaction (SFSI) are typically used in important/critical infrastructure projects, i.e. nuclear powerplants, tall buildings, large dams, and long-span bridges. However, with the development of increasingly powerful and affordable personal computers in modern times, combined with the availability of reasonably priced and open-source software alternatives, advanced analytical methods have never been more accessible and cost effective even with small to medium sized projects.

This paper presents an example to demonstrate how the method can be applied to small-medium engineering design projects for New Zealand engineering practice. Most of the analysis is not included in this paper due to space constraint.

BUILDING AND SITE DESCRIPTION

The example case is an eight-storey reinforced concrete-framed commercial building with the upper two storeys being smaller in plan and the final storey built as a lightweight penthouse. The building extends into the ground to 1 basement level, covering footprint of approximately 300m·. The foundation is a reinforced concrete mat foundation. The concrete structure was built around 1958. The building consists of six concrete frames in the east-west (transverse) direction and four concrete frames in the north-south (longitudinal) direction, Each floor has a 115mm thick suspended concrete slab and beams. Beam sizes are consistent throughout the building, and column sizes gradually reduce from ground to the upper floors.

The west elevation contains masonry infill walls, and there is a full-height opening on the west side of the building, where a lift and concrete stairs are located. At the first-floor level, a 2.4 m concrete canopy overhangs the street on the east and south sides and is supported by a deep beam. Low height walls up to sill level and brick infill walls are located between the perimeter columns at each level. The building foundation, north, and east elevation are shown in Figure 2.

Two well-boreholes (M1 and M2), and two machine boreholes to a maximum depth of 33 m below ground level (bgl) were downloaded from Environmental Canterbury and New Zealand Geotechnical Database website. The three Piezocone Penetration Test (CPT) was undertaken by ENGEO to a maximum depth of 15.0 m, and the raw data was provided to Miyamoto International NZ (MINZ). Further to the above MINZ undertook (1) a 30m borehole

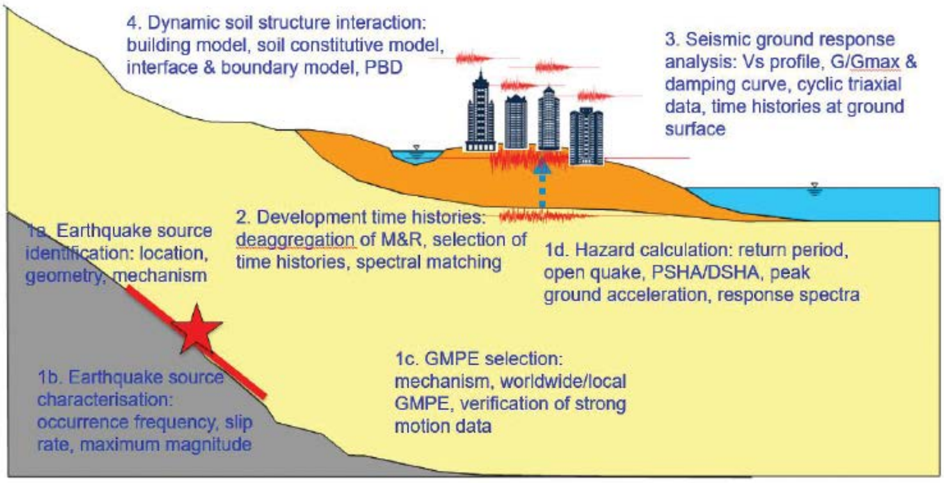
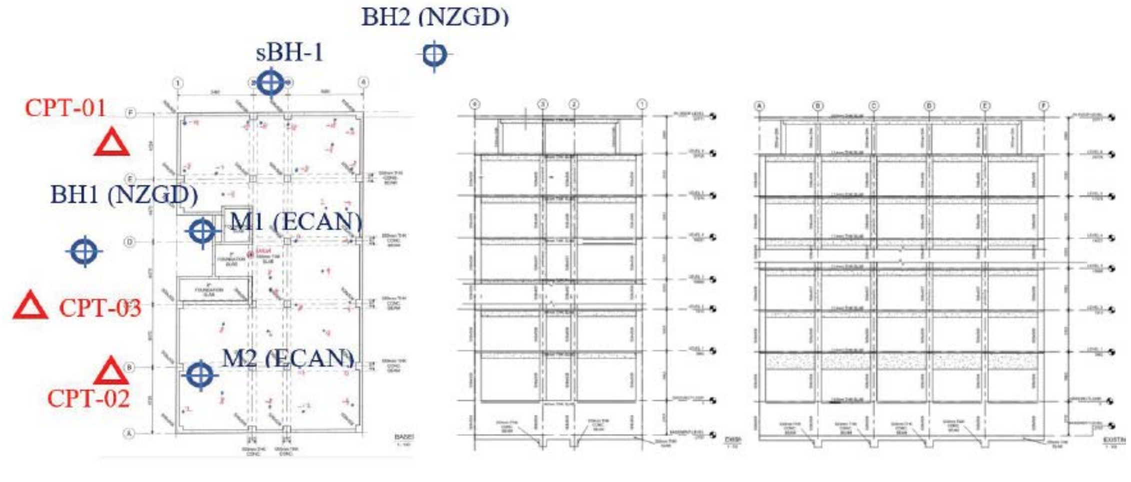
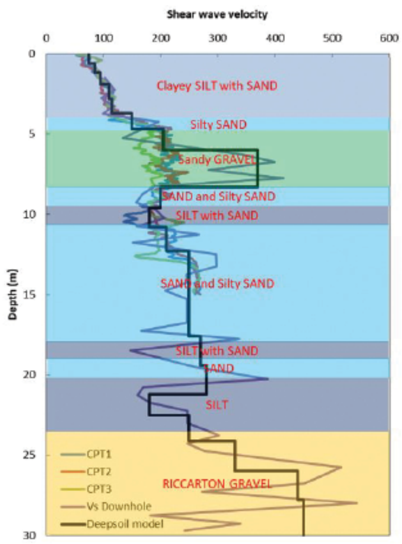


Figure 1: Advance geotechnical earthquake engineering: (1) Probabilistic seismic hazard analysis; (2) time histories development; (3) seismic ground response analysis; (4) soil foundation-structure interaction



(1) Plan view (2) North elevation (3) East elevation

Figure 2: Foundation plan view (with ground investigation locations) and elevation view of its north and east oriented frame



(1) Shear wave velocity profile

Depth (m)	2.5	3.5	9	16	18.5	20.7	22
Passing 75µm sieve (%)	81	76	5	2	88	77	96
Passing 63µm sieve	77	65	4	2	79	73	91
Water content (%)	24.4	31.3	23.6	24.1	28.3	29	26.9
Liquid limit,	27	-	-	-	-	28	-
Cone Penetration limit, CPL	-	32	-	-	-	-	-
Plastic limit, PL	24	28	-	-	-	24	-
Plasticity Index, PI	3	4	-	-	-	4	-

(2) Laboratory testing results

Figure 3: Ground profile at the site with measured, calculated, and SSRA model of Vs, and laboratory test result

for the installation of a grouted PVC pipe; (2) laboratory testing on retrieved disturbed soil samples (fines content and plasticity indexes); and (3) downhole shear (Vs) and compressional (Vp) wave velocity surveys down to 30 m and 5 m respectively. The ground profile and geotechnical properties have been developed following a detailed evaluation of existing and new geotechnical data, the resulting ground model and parameters are shown in Figure 3. Small strain shear modulus, Gmax, is a fundamental parameter for SSRA and DSFSI. Gmax for the soils underlying the site was determined from shear wave velocity, Vs, as

$$G_{max} = \rho V_s^2$$

where r is the total density of the soil.

Vs were derived from the downhole shear test and CPT data using a Christchurch-specific correlation proposed by McGann et al. (2014). The measured and calculated Vs for each of the CPTs to a depth of 15.0m are included in Figure 3. The calculated Vs from CPT of the sandy and silty soils correlate well with measured Vs from the downhole test (correlation coefficient = 0.7 – 0.9). The base of the SSRA model (Riccarton gravels) is set at a depth of 30m bgl and was assigned an average Vs of 450m/s. The Vs profile for seismic ground response analysis is also shown in Figure 3. The estimated site period of the ground is between 0.55s to 0.65s from Riccarton Gravel.

SEISMIC SOURCE MODEL

The seismic source model has been developed from the available published geological, tectonic and seismological information. The sources model includes active faults, area sources, and time-dependent point source model based on modified Oomori Law, as describe below:

1. Existing active fault and area seismic sources

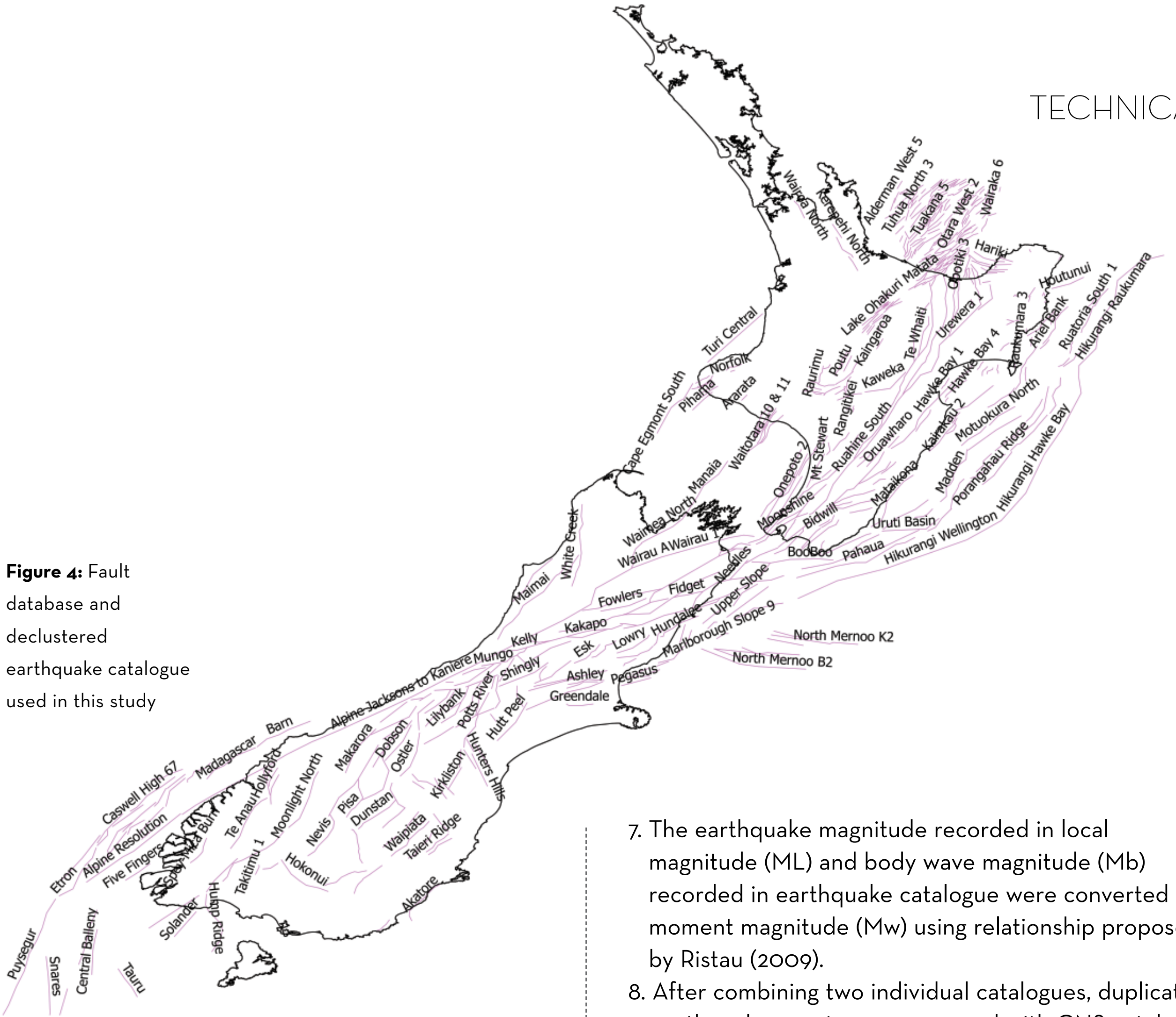


Figure 4: Fault database and declustered earthquake catalogue used in this study

throughout New Zealand have been characterised and incorporated into the 2010 New Zealand National Seismic Hazard Model (NZSHM). Details of the data for the NZSHM can be found in Stirling et al. (2012) and can be downloaded through GNS website.

2. The fault database has been updated based on the data compiled by Langridge et al. (2016) as shown in the New Zealand active faults database (http://data.gns.cri.nz/af/).
3. The fault trace has been re-traced using the Light Detection and Ranging (LIDAR). The data was downloaded from Land Information New Zealand (LINZ) website (https://data.linz.govt.nz/layers/) and incorporated to open-source software called QGIS version 2.18 as shown in Figure 4.
4. Seismic hazard parameters presented in this paper are based on the NZSHM seismic source models uploaded to open-source SHA software (OpenQuake 2.1.0) and updated to the latest study.
5. The maximum magnitudes of faults were updated based on scaling relations suggested by Stirling et al. (2013).
6. The area sources using gridded (smoothed) seismicity model was updated using an earthquake catalogue before 2016. The earthquake catalogue is downloaded from GeoNet and ISC-GEM (Storchak et al., 2013) website.
7. The earthquake magnitude recorded in local magnitude (ML) and body wave magnitude (Mb) recorded in earthquake catalogue were converted to moment magnitude (Mw) using relationship proposed by Ristau (2009).
8. After combining two individual catalogues, duplicate earthquake events were removed with GNS catalogue as a primary source.
9. The full catalogue was declustered using the algorithm proposed by Gardner and Knopoff (1974).
10. The earthquake records within the declustered catalogue were evaluated for completeness.
11. A free Python and R programming were used during the process above.
12. A 50km Gaussian kernel as proposed by Frankel (1995) was used. A Fortran programming script developed by Frankel (1995) for 1996, 2002 and 2008 USGS seismic hazard map was used in this study. The region was divided into cells of 0.1° in latitude and 0.1° in longitude.
13. The Ground Motion Prediction Equation (GMPEs) presents earthquake ground motion attenuations where different from NZSHM and shown in Figure 5. New Zealand site-specific Ground Motion Prediction Equation (GMPE) by Bradley (2013) and McVerry et al. (2006) have been used.
14. Logic tree analyses were used to account for epistemic uncertainty in the magnitude and GMPE as shown in Figure 5.

TIME-DEPENDENT MODEL USING MODIFIED OOMORI LAW

A key assumption of the standard PSHA model is that

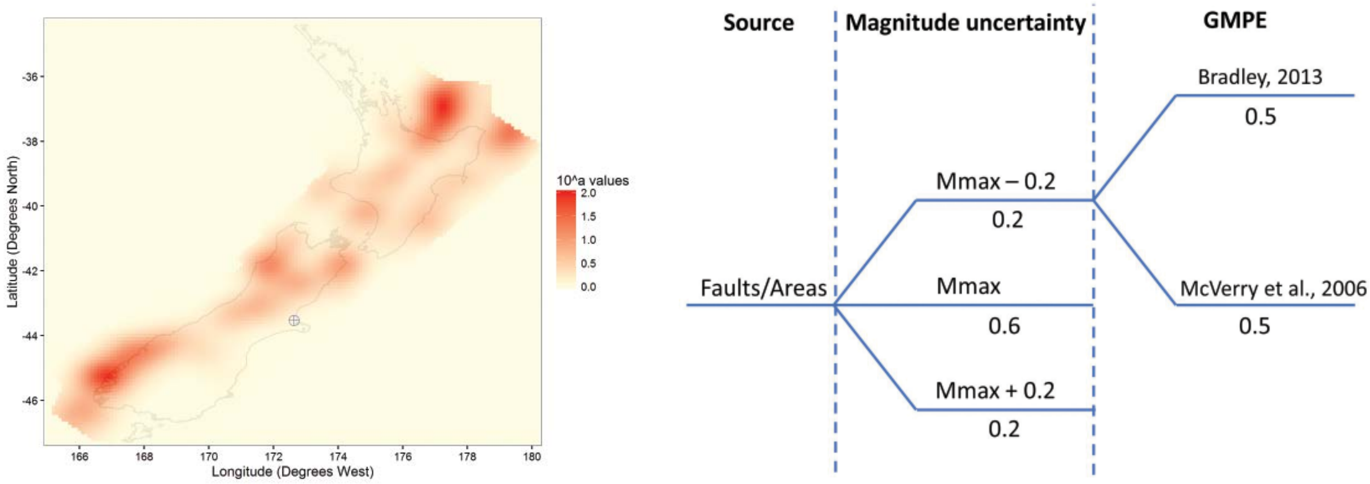


Figure 5: The example output of gridded (smoothed) seismicity model and logic tree model used in this study

earthquake occurrences can be modelled as a Poisson time independent process. However, standard PSHA does not model hazard decay, resulting from and event(s) such as the CES, over the typical 50-year design life of future buildings and structures. The time-dependent model in this study follows the procedure proposed by Bradley (2014, 2016) with the following modifications:

1. The activity rate was modelled as 55 points sources on 0.1° x 0.1°grid over an area of -43.75° to -43.25° Latitudes and 172° to 173° Longitude as shown in Figure 6.
2. Depth uncertainty was incorporated by weighting the activity rate at depths of 2km (10%), 5km (30%), 10km (40%), and 18km (20%).
3. The earthquake recurrence parameter, b-value of 0.9 is considered conservative value which follow the cumulative number of event as shown in Figure 7.

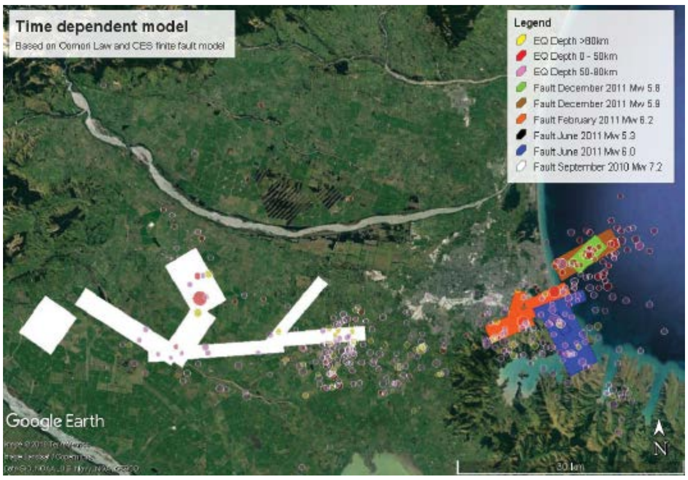


Figure 6: Spatial distribution of Mw·3.5 earthquake events in the CES. The considered region of -43.75° to -43.25° Latitude and 172° to 173° Longitude

UNIFORM HAZARD SPECTRA (UHS), DEAGGREGATION, DEVELOPMENT OF TIME HISTORIES, SSRA AND DSFSI

A PSHA (Cornell, 1968) was used to assess the target uniform hazard spectra and controlling earthquake rupture scenarios for the site. The PSHA was carried out using open-source software OpenQuake 2.1.0 (Pagani, 2014) to evaluate the site seismic hazard. The PSHA model included a time-independent analysis as per the standard PSHA as well as a time-dependent analysis.

The PSHA was carried out for Riccarton Gravel site conditions with an assumed average shear wave velocity in the upper 30m (Vs30) of 450m/s. Bradley (2015) observed the ground motion records associated with the CES and found that there was a systematic shift between the predicted and observed spectral acceleration at different spectral periods. The authors have applied the proposed correction factor for the Central Business District (CBD) to our PSHA results.

The predominant site periods were estimated as 0.6s, thereby the seismic hazard of 25-year and 500-year return periods are deaggregated at a spectral period of 0.6s as a way to identify the predominant earthquake moment magnitude (M) and source-to-site distance that contributes most to the seismic hazard at the site.

Spectrally-matched, single-component horizontal earthquake acceleration time histories were developed for each of the two return periods, 25-year and 500-year. The time history development follows the standard North American practice of selection time histories and spectral matching.

The authors searched the PEER NGA-West2 earthquake record database and the GeoNet strong motion database (Van Houtte et al., 2017), and selected acceleration time histories whose response spectra, after linear scaling,

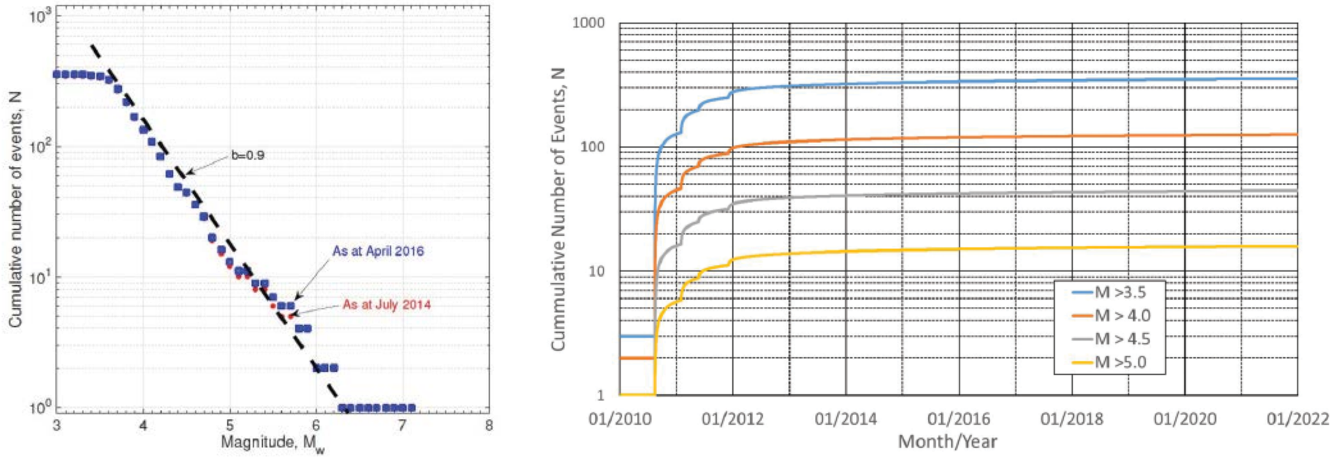


Figure 7: The predicted rate of aftershocks from 1 January 2017(Mw>3.5 à 38 events, Rate@M5=1.7 for next 50 years

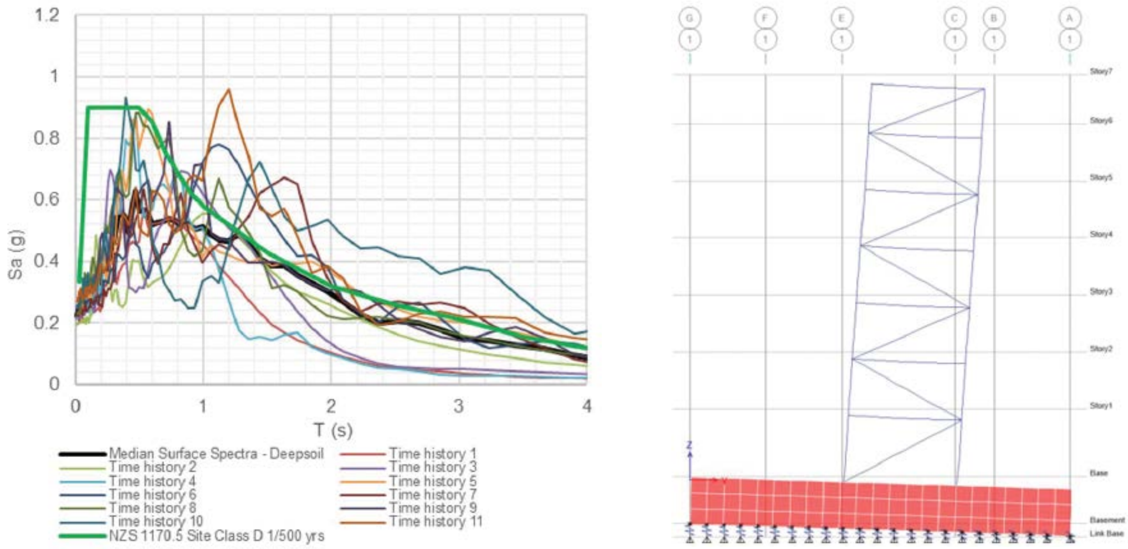


Figure 8: The surface ground response spectra and ETABS model showing displacement with consideration of SFSI

correlated well with the target spectra close to the fundamental site period. The selected scale factors range from 0.5 to 4. The suite of selected seed time histories generally covers the contributing earthquake magnitudes and source-to-site distances as indicated from deaggregation results.

For the time histories selected in this study, the spectral matching was accomplished using RSPMatch09 software developed by Al Atik and Abrahamson (2010). SSRA was completed using the Christchurch specific silty and sandy soils constitutive model developed by Arefi (2014) as well as the Idaho-US gravelly soil model developed by Menq (2003).

1D nonlinear total stress ground response analysis has been undertaken using Deepsoil v6.1 software (Hashash et al., 2016). The ground profile was subjected to each of the eleven time-histories. The surface ground response spectra are presented in Figure 8.

STRUCTURAL MODEL INCORPORATING SOIL FOUNDATION-STRUCTURE INTERACTION

The model of the building was constructed in ETABS including the basement level.

The soil foundation-structure interaction was modelled as bilinear compression only springs (no uplift capacity), at the base of the external basement walls as shown in Figure 8, using ETABS. The force-displacement curve was averaged from half of the building's footprint and distributed into a theoretical 1m strip beneath the walls.

The initial result of the modal analysis showed that the building's fundamental period of vibration to be in excess of 1.5s, in comparison to 0.5s if assuming that the building is fixed at the base. One of the issues in using this approach is that the confining effect of the soil on the side walls was ignored. The period directly from the ETABS model is approaching to the result of pure rocking model whereas in reality the structure behaviour should be in between a full rocking structure and fixed based structure. This is due to the fact that confining effect due the soil acting on the side basement walls were not accounted for. Further verification of the model was completed as a secondary check and refinement. The result reduces the effective period of the building from a full rocking period of 1.5s to around 1.2s.

In addition, an 'corrected' forced based method described by B.Sporn and Pampanin (2013) was used to check the base shear. This was completed by iterating the period and stiffness of the structure until convergence. For an assumed ductility of 1.25, the period corresponding was eventually estimated to be 1.2s and agreement with the result.

SUMMARY AND CONCLUSION

The multi-story building in Christchurch CBD constructed around 1958 was assessed by applying a site-specific geotechnical earthquake engineering design approach using open-source software and automatization scripts, resulting in slightly lower spectral acceleration compared to the NZS 11705:2004 spectra. It can be shown that by using the result from site specific geotechnical engineering design approach, combined with further adjustment and verifications, a more realistic prediction of the building's behaviour can be estimated rather than using a conservative approach of assuming fixed-based model.

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Managing Risk for Workers on Slopes Following the 2016 Kaikōura Earthquake

ABSTRACT

Disaster recovery takes place in an abnormal environment. A defining tension exists between the need to rebuild quickly, but with careful deliberation. This tension poses risks for the health and safety of workers involved, at a time when risk levels are higher than normally encountered in the workplace. A key question is how to implement "good practice" health and safety procedures to protect workers in condensed timeframes that are distinctive post-disaster. This case study on the Kaikōura Earthquake will specifically address the demands placed on rope access workers involved in the reconstruction of the distributed transport network, the hazards encountered and how risk was managed.

Key findings are that the transition from disaster response to recovery is a crucial phase of reconstruction, during which clarification of expectations and information sharing benefit workers. Quantification of risk, including a consideration of societal risk, should be a process that is both transparent and inclusive of workers, according to the law and to "good practice". Preparation activities, such as pre-disaster training, planning and testing of emergency procedures can reduce risk.

Future research is recommended into reconstruction following the Kaikōura Earthquake to evaluate emergent safety culture and develop a model to improve risk communication through a multi-level organization to workers at field level. Improvements in the management of safety for reconstruction workers will allow for more effective and efficient recovery in future natural hazard events affecting critical lifelines and infrastructure, improving the resilience of transportation networks and communities in New Zealand.

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INTRODUCTION / CONTEXT

The Kaikōura Earthquake

The M_w 7.8 Kaikōura Earthquake on 14th November 2016 caused severe social, economic and environmental impacts in New Zealand. Ground shaking, surface rupture and thousands of co-seismic landslides damaged infrastructure on a regional scale, across the north east of the South Island and in Wellington. The township of Kaikōura was severely affected, suffering acute isolation during the peak tourist season. State Highway One (SH1) and the Main North Rail Line (MNL) are critically important strategic assets for New Zealand. Both were severely impacted by the earthquake, requiring closure for urgent repairs, lasting for 13 months.

Mitigation of the landslide hazard above the road and rail is ongoing; operational restrictions, such as single lane access and reduced speed limits still apply in places. The economic recovery of the Kaikōura and Hurunui Districts, the Canterbury and Marlborough Regions and the nationally important tourism and freight industries is directly reliant on a fully functioning and resilient transportation network (Davies et al., 2017; Ministry of Transport, 2017; Mason & Brabhaharan, 2017; New Zealand Government., 2016).

NCTIR Alliance

Large-scale natural disasters require multi-agency responses. In December 2016, the New Zealand government passed the Kaikōura/Hurunui Earthquakes Recovery Act and agreed to fund the repair of SH1 and the MNL, north and south of Kaikōura. The North Canterbury Transport Infrastructure Recovery (NCTIR) was established to restore the transport network infrastructure between Picton and Christchurch. NCTIR is an alliance partnership between the New Zealand Transport Agency (NZTA) and KiwiRail (the asset owners) and four large construction companies (Fulton Hogan, Downer, Higgins and HEB) (NZ Govt. 2016).

Due to the complex nature of the reconstruction work, a large number of personnel with specialized rope access training and experience were needed on the slopes in Kaikōura to complete the scope of works in a timely manner. All the major rope access contractor companies operating in New Zealand and one Canadian company became involved in the reconstruction, as subcontractors to the NCTIR Alliance.

Risk Levels for Workers During Disaster Reconstruction

Reconstruction following a natural disaster is often large in scale, long in duration and complex in terms of the range of hazards, to which workers are exposed. An over-arching characteristic of disaster recovery is compression of infrastructure repairs in time and in a limited space. Both time and space compression have critical implications for protecting the health and safety of workers during the immediate and sustained phases of the response and recovery (Jackson et al., 2002; Johnson & Olshansky, 2016; Olshansky et al., 2012). During disaster reconstruction, workers are required to carry out critically important, urgent and dangerous work, at some personal risk. Even trained and highly skilled individuals increasingly have to cope with events of a scale larger than they would normally encounter. Inadequate training of some workers, due to the numbers required, results in situations and responsibilities being encountered, which fall outside their accustomed roles. There is often a more prolonged exposure to high-risk situations than they are equipped to deal with (APHA, 2008; Jackson 2002; Olshansky et al. 2012; Sim, 2011).

Intense pressure to open the transport network in the shortest possible time, contributed to the Kaikōura reconstruction being a high hazard industry sector with an elevated risk profile for workers on slopes. This paper identifies some approaches used by rope access workers during the reconstruction to implement "good practice" in health and safety standards and outlines risk management strategies used to manage the elevated risk levels, at a time where urgency to rebuild quickly was an overriding factor. Our recommendations for improvements form part of a long-term strategy to reduce risk during the emergency response and recovery for workers on slopes, where conventional means of access are not available following future large earthquakes, and where slope instability impacts critical infrastructure.

THE REGULATORY ENVIRONMENT

The Kaikōura Earthquake occurred in the year following a significant reform of New Zealand's Health and Safety at Work legislation, brought about by the Pike River Mine tragedy in 2010 and the subsequent findings of a Royal Commission of Enquiry. Post-disaster reconstruction in New Zealand is governed by two parallel pieces of legislation (and by risk management and "working at height" guidelines and qualifications).

The Health and Safety at Work (HSW) Act, (2015) governs health and safety at work, but recognizes that other New Zealand legislation may affect workers. The Act addresses such overlaps by providing that other legislation can be considered when deciding whether health and safety duties are being met. Where two pieces of legislation apply, *the duty holder must follow both* (Worksafe, HSWA Special Guide 2017). There is no distinction in the HSW Act (2015) between post-disaster times and "normal" times, yet an important distinguishing characteristic of disaster recovery is that it takes place in an abnormal environment (Johnson & Olshansky, 2016). Under the new Health and Safety at Work Act (2015) a framework for continual improvement includes appropriate scrutiny and review of actions taken by persons performing actions or exercising powers (Health and Safety at Work (HSW) Act, 2015).

The Civil Defence and Emergency Management (CDEM) Act (2002) creates a framework within which New Zealand can prepare for, cope with and recover from local, regional and national emergencies. The CDEM Act requires communities to achieve acceptable levels of risk by correctly identifying risks, adopting risk reduction management practices and provide for planning and preparation for emergencies, and for response and recovery (CDEM Act, 2002). The CDEM Act (2002) does not specify particular health and safety environments for workers during or after emergencies, but does recognize that the safety, health and well-being of a "community" is an integral part of the generic recovery structure after a natural disaster. The "community" is not specified by the Act, but must surely include the workers who have been involved in reconstruction?

The AS/NZS ISO 31000:2009 Risk Management Principles and Guidelines outline good practice processes for managing risk in New Zealand. According to these guidelines every aspect of the risk management process needs to be systematic, transparent and inclusive, and facilitate continual improvement of an organization and a dynamic response to change. In the case of risk management for workers the priority should be protecting life and safety from harm. Agencies responsible for the safety of workers have an overarching responsibility to make good decisions about exposure of workers to known risks (ACC & Worksafe, 2013; Jolly et al., 2014; Worksafe, 2017).

"Working at height" is the term used to denote a preventative safety measure where work positioning is achieved using Personal Protective Equipment (PPE) to prevent a person from falling. "Working at height" methods allow the worker to access the place of work and perform tasks while suspended in areas where conventional means of access are not possible (IRATA ICOP, 2014). Workers must have either International Rope Access Trade Association (IRATA) or Industrial Rope Access Association of NZ (IRAANZ) qualifications (or both) in order to utilize "working at height" methods in the workplace in New Zealand.

Some occupations are unavoidably exposed to hazards that are in the nature of their jobs. The requirements of the role played by rope access workers involved in the Kaikōura reconstruction are such that it could not be performed without exposure to some risk (Fig.1). A lack of New Zealand-based rope access technicians with the relevant experience meant that many in the workforce were contractors from overseas or were newly qualified technicians with no prior geotechnical experience.

HAZARDS

Working at heights is intrinsically hazardous; workplace accidents can have severe consequences. Worldwide, falls from height remain the most common cause of serious and fatal injuries in the workplace (Fleming, 2001; IRATA ICOP, 2014). Additional hazards to personnel are encountered in the geotechnical field. Environments can include falling rocks, toxic dust and unstable surfaces, as well as the frequent use of heavy machinery, drills and compressed air, which require intensive management processes. High levels of experience and supervision are needed to ensure that safe working methods are maintained (IRAANZ., 2012).

Aftershocks

A particular issue for workers in the Kaikōura reconstruction is the dynamic nature of the risk, which remains elevated due to the increased likelihood of aftershocks following a significant earthquake. Although the risk is expected to decline with time (as the aftershock sequence decays) the recovery effort may well be over by the time the probability of seismic events returns to background levels (GNS, 2016). In the year following the earthquake, during the most intense phase of reconstruction activities, the probability of one or more M_w 6.0-6.9 aftershocks in the Kaikōura area was initially estimated at 98% (extremely likely). This forecast was updated every 3 months; by February 2018 the probability estimates had fallen to 53% (Geonet, aftershock forecasts, 19thDec. 2016 & 5th Feb. 2018)). A large aftershock, if

Figure 1: Rope access technicians (circled) scaling head scarp, Slip 7 north of Ohau Point, Kaikōura, Feb. 2017. Photo: R.Musgrave

centred close to an occupied worksite on or below an unstable slope could have had severe consequences.

Tsunami

A locally generated tsunami is characterized by a short time interval between initiation and run up. Multiple tsunamis generated by either a fault rupture offshore, or by underwater landslides into the Kaikōura Canyon (or both) are possible following an aftershock near the Kaikōura coast. These types of tsunami have arrival times of between 10 minutes and 1.5 hours following an earthquake (Walters et al., 2006). Many occupied worksites on the coastal transport route were (and still are) situated close to sea level.

Post Seismic Rainfall-induced Landslides

A large earthquake not only triggers severe co-seismic landsliding but can also reduce the stability of slopes for a long period of time post-earthquake. The probability of recurrence of large-scale landslides is very high, as slopes have been weakened and fractured by recent seismic shaking (Huang & Li. 2014; Qiu et al., 2017; Tang et al., 2011). In addition, critical rainfall thresholds for triggering landslides and debris flows decrease significantly (compared to the pre-earthquake thresholds), subsequently increasing the frequency of rainfall-induced landslides in regions affected by strong ground shaking (Lin et al., 2006; Zhang et al., 2014). The seismically damaged slopes north and south of Kaikōura are now more susceptible to rapid failure in high-intensity rainfall events. Secondary effects such as rock falls, landslides and debris flows after heavy rain have potential to cause significant problems for people working in the immediate areas where slopes have been seismically weakened.

Figure 2: Construction of shallow landslide barrier, Slip 18 south of Kaikōura, Nov. 2017.

Figure 3: Installing mesh by helicopter sling load, a high-risk activity, Slip 18. Heli-operations were often conducted over an open highway. Photo R. Musgrave

THE ROLE OF ROPE ACCESS TECHNICIANS IN EMERGENCY RESPONSE AND RECOVERY

Rope access technicians were positioned on sites above the Inland Kaikōura Road (Inland Route 70) within days of the Kaikōura Earthquake. The value of rope access techniques to facilitate safer access for the opening of this critical lifeline was evident early on in the emergency response, as the slopes above the road were unable to be accessed by traditional means. Rope access workers were engaged to remove the critical hazards at the source by "scaling" (removal of loose rocks with crow bars) and were also used as "spotters" positioned on the landslides to observe initiation of movement and provide early warning. These actions were implemented to reduce the risk for other workers at road level who were clearing debris for emergency access, and with minimum disruption to New Zealand Defence Force convoys travelling the route daily to take essential supplies into Kaikōura.

On November 30th 2016, the Inland Kaikōura Road was provisionally opened to civilian convoys. Work then began on SH1 and the MNL, first south and then north of Kaikōura. Construction workers began to remove debris from the toe of the landslides, in order to facilitate access to begin repairs on the road and rail. Initially, the rope access technicians were providing a support role, reducing risk for other workers on the project as in the response phase, and thus enabling important and urgent work below the earthquake damaged slopes to proceed. Later, the construction of temporary and permanent engineered rock-fall and landslide risk mitigation structures began on the slopes. This specialist activity requires a high degree of skill and experience (Figs. 2-3).

RISK MITIGATION FOR EMERGENCY RESPONSE

Aftershocks were occurring frequently at this time, creating a culture of extreme caution amongst rope access workers, who needed to descend into the zones of highest rock-fall hazard on slopes to perform tasks. Safety concerns had to be balanced with a commitment to assist in the emergency response and play what was considered to be a critically important role. Key safety considerations included:

- Limiting time spent and number of people in high risk zones, minimizing exposure to individuals.
- The rope access teams employed a "top down philosophy" which dictates removal of rock fall hazard

Figure 4 (left): Air bags used to remove an unstable column of rock while operators retreat to a safer location, Slip 10, south of Kaikōura. Photo R. Musgrave. **Figure 5** (right): Sluicing to remove loose debris below abseiler, Slip 7, north of Ohau Point, Kaikōura. Photo R. Musgrave

before descending below, and avoidance of areas with high hazard lower on slopes (where possible).

- Rescue systems were rigged prior to descent, with standby rescuers remaining at the top of slopes, to facilitate very rapid extraction of operators from the rockfall hazard zone if required.
- Only the most experienced and highly qualified rope access team was engaged in the response phase.
- The rope access team included two rope-access qualified engineering geologists who were able to report site observations to the ground-based geotechnical team at the time.

RISK MITIGATION FOR RECOVERY / RECONSTRUCTION

Avoid or Substitute

Where possible, operators avoided accessing the lower slopes of the landslides, by substituting alternative methods for removal of hazards (Figs. 4-5).

Temporary (non-engineered) Risk Mitigation Structures

In order to begin the construction of engineered risk-mitigation design structures, it was necessary in some cases to first install temporary structures for the protection of workers required to spend long periods of time below significant rock fall hazard on the lower slopes of landslides (Figs. 6-7).

Additional Training

During the Kaikōura reconstruction, and due to the difficulties around access and safety, geotechnical professionals relied on observing slopes from a distance giving a broad overview and using information relayed by rope access workers about detailed ground conditions. Over the course of the reconstruction, some members of the geotechnical team became qualified for rope access to IRATA Level 1, allowing them to reach on foot difficult to access sites and observe slope conditions more closely (under the supervision of more experienced rope access operators).

Some rope access contractors also received additional training in basic structural geology, risk awareness, hazard identification and factors affecting stability on the earthquake damaged slopes. This enabled them to understand better the main factors that lead to slope failure, identify and report unsafe conditions and take appropriate action. Observing, monitoring and reporting slope conditions by all on site proved an effective way of managing risk in the work environment.

The NCTIR Rainfall Trigger Action Response Plan (TARP)

To manage the elevated risk to workers during rainfall, telemetered slope monitoring instruments and rain gauges were installed at numerous locations along the coastal transport route on sites most affected by slope instability. The NCTIR Rainfall Trigger Action Response Plan (TARP) was implemented as a predictive risk management tool in March 2017, to reduce risk for the travelling public and recovery workers. Decisions are made (using real-time monitoring of rainfall) to close worksites (and the road and rail), based on forecasted rainfall in relation to antecedent

Figure 6: (left): Example of temporary rockfall mesh installed above an occupied worksite, Slips 18 &19, south of Kaikōura. This structure contained a ~100 m3 failure which occurred during rainfall, June 2017. **Figure 7** (right): Temporary rockfall catch fence above worksite, Slip 18. This structure was impacted 4 times by rocks ~1m3 during the course of construction of a shallow landslide barrier directly below. Photos: R. Musgrave

rainfall conditions, using a model developed by Glade et al. (2000) for determining rainfall-triggering thresholds for landslides in Wellington, New Zealand. Rope Access workers and the geo-technical team then used the model thresholds as a guide (along with ground observations and slope monitoring within their teams), for dictating avoidance of the slopes during or immediately after significant rainfall events.

Emergency Procedures
All qualified rope access teams have the training and capability to perform a rescue of an injured worker on ropes (IRATA ICOP, 2014). However, in January 2017, senior rope access contractors became concerned that the particular hazards encountered in the work environment on the slopes in Kaikōura, combined with the possibility of a large aftershock occurring, required specialist rescue training over and above "normal" rope access requirements. The possibility of multiple rock falls and slope failures in an aftershock could mean that many severely injured casualties would require rescuing simultaneously from different sites. It was felt that the capability to perform rescues in this scenario did not exist. The rope access teams requested that senior team members with specialist medic training also received "long line" rescue training where rescues are performed from underneath a helicopter, giving the option to perform rapid extraction of many injured persons from slopes if necessary.

Evacuation Planning
All rope access teams had their own evacuation plans in case of an emergency and identified "safe" places to muster, relevant to each worksite. Often, the safest means of egress from a worksite on a slope was identified as up to a muster point on a ridge, rather than down to road level. In January 2017 it was pointed out by rope access workers that the tsunami "safe" places and muster points identified in the current tsunami evacuation plan for worksites along the coastal route were at sea level, in the tsunami evacuation red and orange (must evacuate) zones (ECAN Tsunami evacuation zones, [MAP] ND). Worker input prompted a broad scale review of evacuation plans.

DISCUSSSION

Assessing Risk
IRATA requirements specify that site-specific risk assessments be carried out, with input from all rope access team members, before work commences. These assessments are qualitative and use a risk matrix to assess the potential likelihood and consequences of

hazardous events. Before commencing work, all tasks should be organized, planned and managed so that there is an adequate margin of safety to reduce risk (IRATA ICOP, 2014). Most experienced rope-access trained contractors are proficient in their work but lack formal training in geology or engineering geology. It is important that persons with training and experience in rockfall and landsliding are involved in assessing risk on site, because under or over estimating risk can affect the outcomes of the risk analysis (AGS, 2000).

Quantifying Risk

Quantifying risk is useful because it allows a comparison of hazards and enables authorities and workers to prioritize risks in order to inform decision-making (AGS, 2000; Massey et al., 2012; Rovins et al., 2015; Taig et al., 2012). In New Zealand, managing the risk from landslide hazards follows principles and guidance set in Australia by the Australian Geomechanics Society (AGS). The AGS recommends that some degree of quantification of risk is attempted in all cases, even if crude or preliminary, especially where loss of life is a possibility. This allows comparison with the acceptance criteria for loss of life, which is also quantified (AGS, 2000).

Quantitative risk assessment is increasingly being used to inform government and private sector policy decisions in New Zealand. The Christchurch Earthquake Sequence (CES) set a precedent for its use. Individual annual fatality risk was the criterion for establishing upper limits of risk tolerability from rock fall and cliff collapse on the Port Hills. The Christchurch City Council then used this criterion to guide decision-making regarding the safe occupation of buildings below or on the edge of cliffs (Massey et al., 2012; Taig et al., 2012).

In 2012, two small eruptions from Mt. Tongariro produced multiple volcanic hazards in the Tongariro National Park, prompting closure of the popular Alpine Crossing track for 6 months. The key reasons for this extended closure were safety concerns for track users, however decisions had to made prior to the track opening, to determine whether the risk was tolerable, to allow Department of Conservation workers and GNS scientists access to the closed areas (Jolly et al. 2014). The period following the November 2012 eruption was a time of considerable uncertainty requiring a transparent decision-making process concerning access close to the active volcanic vent. Discussions with the New Zealand government agency responsible for health and safety in employment emphasized that there should be no compromise to staff safety standards by the Department of Conservation or GNS Science. Life safety risk mitigation was the paramount consideration, which had to be balanced against losses for the local, regional and national economy (Jolly et al., 2014). GNS scientists performed basic quantitative risk assessments within days of the eruption to analyze life safety risk from ballistic hazards, using an expert elicitation panel. This process balanced the urgent need to collect scientific data and repair infrastructure, with health and safety in employment regulations, in order to facilitate informed decision-making (Jolly et al. 2014).

In the Kaikōura situation, fatality risk for individuals is of primary consideration and should be quantified in order to manage overall risk in the workplace on slopes (AGS, 2000). The entire risk management process must be transparent and inclusive of stakeholders at all levels according to New Zealand law, to risk management standards and good practice (AS/NZS ISO 31000:2009; HSW Act (2015); Jolly et al., 2014).

Risk Evaluation, Establishing Criteria and Uncertainties

Risk evaluation assists with decision-making after risk has been assessed, to decide whether to accept or treat the risks and to set priorities for action. To make decisions, the level of risk is compared against criteria, to determine what is acceptable, tolerable or otherwise. The UK Health and Safety Executive (HSE) judges the tolerability of risk first in terms of the absolute levels of risk to individuals - only if individual risk is tolerable is it then reasonable to proceed. Individual risk is used as the primary measure of risk, but societal risk must also be considered (HSE 2008).

In general, higher risks are likely to be tolerated for workers in industries with hazardous slopes, than for society as a whole. Upper limits of tolerability of 10^{-4} per year individual fatality risk for members of the public and 10^{-3} per year for employees are suggested in the UK (AGS 2000; HSE 2008; Taig et al. 2012). In New Zealand, such criteria have not yet been firmly established at Government level and in the private policy sector with regard to the workforce.

A significant aftershock on a nearby fault, which ruptures at shallow depths has the potential to cause multiple rock falls, landslides and generate a tsunami on the coast near Kaikōura. In this scenario, significant hazards exist for people on worksites on (and below) steep slopes at sea level. Loss of lives is a real possibility. If the possibility of loss of lives exists, the probability that the incident might actually occur should be sufficiently low that relevant risk criteria are met (e.g. probability x number of deaths $<10^{-3}$ for workers). This accounts for society's particular intolerance to events that cause many simultaneous casualties and is embodied in societal tolerable risk criteria (AGS 2000).

There will be an element of risk in all decisions that are made: zero risk is not achievable. Furthermore, it is not advisable to use quantitative risk estimates as the sole determinant for making decisions in light of the uncertainties in many estimates of risk. The assessed risk may span the acceptance criteria, requiring a high degree of confidence about what is tolerable when making decisions (AGS 2000; HSE 2001; HSE 2008; Taig et al. 2012).

Exposure

The reconstruction following the Kaikōura Earthquake was a large-scale civil construction project for New Zealand. Since January 2017, over 7500 different workers have worked over 4,300,000 hours on 180 different worksites (Bell, 2019). On Dec 15th 2017 State Highway 1 to Picton re-opened and the consequent traffic flow increased to approximately 5000 vehicles daily. This was a significant milestone for the Kaikōura community, the freight and tourism industries, the New Zealand Government and the NCTIR alliance (NZTA 2018).

The benefits of opening the highway were clear to all working on the project. However, conducting the repair works above an open highway with traffic passing below work-sites caused concern for rope access contractors. Many consequential stoppages lengthened the duration of the project and affected productivity, adding to the frustration for workers, stakeholders and the public. More importantly, this decision changed the level of risk workers were exposed to, as they were required to spend a longer period of time in the hazard zone that would be normally be acceptable to them.

The result of re-opening the highway while rope access work was still ongoing was that the temporal probability (of an individual being at a given location, given the spatial impact of the hazard) increased significantly for workers, because the scope of the project was increased by an unspecified period of time. An increase in temporal probability (exposure) increases the fatality risk for people working on or below slopes (and for members of the public using the road).

Testing of Emergency Plans

Under the HSW Act (2015), emergency plans must be prepared for each workplace prior to work commencing. There must be provision for the testing of evacuation plans and training and instruction given to workers. This is also a key component of IRATA risk management procedures (IRATA ICOP, 2014). The CDEM Act (2002) specifies that scheduling of training and exercises to validate plans falls under "Readiness" activities (CDEM 2005). Pre-disaster plans can improve the speed and quality of post-disaster decisions. Organizations involved in recovery should plan and act simultaneously (Johnson & Olshansky 2016).

The fast pace of the Kaikōura reconstruction and competition for limited resources made it difficult to prioritize the formulation of emergency plans when they were most needed, early in the response and transition phases. The testing and refinement of emergency plans did not occur until the recovery phase was well underway.

Production Pressure

Worksafe recognizes that both physical and psychological factors are at play in the work place: deadlines create stress and fatigue amongst workers and can compromise efforts to maintain a work environment with acceptable levels of health and safety (Worksafe, 2017). Since the Kaikōura Earthquake, a number of milestones have been heralded as major successes during the rebuild of the transport corridor. The scale of works completed or near completion would normally have taken many years during a time of standard operations or "business as usual".

Prolonged closure of SH1 and the MNL have incurred a high economic and social cost for New Zealand (Davies et al., 2017; Ministry of Transport, 2017; Mason & Brabhaharan, 2017; New Zealand Government., 2016). High profitability and high health and safety standards can be complementary factors; however, it is important to acknowledge that tension can arise between different goals (profitability and safety) in specific decision-making processes. The heightened risk for rope access technicians on coastal slopes in Kaikoura had to be balanced against increased risk levels for users of the alternate transport route, which experienced an increased rate of crashes in the time that SH1 and the MNL were closed, due to the huge increase in traffic (including heavy vehicles) on roads that were not built for that purpose.

Cost-Benefit Analysis vs. the Cautionary Principle

Commonly, operating companies and regulators conceptualize risk as a product of frequencies and consequences, often by quantification. A cost-benefit analysis is a method for quantifying the advantages and disadvantages of different solutions and providing a basis for their prioritization and is used as the justification for implementing (or not) risk-reduction measures. A cost-benefit analysis is a useful tool, giving insights into risk and the considerations involved. However, there are limitations on its use as the results are conditional, based on a variety of assumptions. In the context of high-risk industry sectors, such as reconstruction after a natural disaster, it must be recognized that some benefits and costs (loss of life or injury) are exceedingly difficult to quantify in monetary terms. Any attempt to provide a comparison of costs and benefits in the context of major accidents with low probabilities of occurrence is nearly impossible (Sorskar &

Abrahamsen 2017).

The cautionary principle is a fundamental principle in safety management giving full weight to risk and uncertainty, thus representing an extreme safety perspective. According to this principle, in a context with uncertainty and risk, caution should be the ruling principle, by the implementation of risk-reducing measures, or by not starting an activity (Sorskar & Abrahamsen 2017).

RECOMMENDATIONS

Workers are an integral part of the disaster recovery community. The requirement to protect workers should be a primary consideration in a recovery done well in order to improve safety, capability and efficiency in processes and outcomes after future disasters. Future strategies for worker protection include:

- Establishment of a National register of trained and experienced rope access operators.
- Appropriate ratios of inexperienced to experienced operators maintained.
- Increasing the capacity for planning the transition between response and recovery and for the formulation and testing of emergency plans.
- Workers and regulators require support mechanisms for decision-making during times of uncertainty.
- Assessing, quantifying and communicating risk through formal channels is critically important for the protection of workers.

Staff Training and Experience

Natural hazard events like the Kaikōura Earthquake will occur again in New Zealand, requiring input from contractors, consultants and stakeholders in a collaborative approach. Having a skilled New Zealand based workforce to call on in the event of a future emergency will increase the capability of regions to respond in a safe and effective manner. A register of all contractors and consultants, who have had experience with this type of geotechnical work in Kaikōura and following the Canterbury Earthquake Sequence (and have undergone additional training) would allow the experience gained during this reconstruction to benefit New Zealand in the future.

IRAANZ and IRATA qualifications are adequate for training people to work safely at heights. The training and certification are conducted in a controlled environment, which does not prepare technicians for the additional hazards encountered on unstable slopes in a seismically active area. Newly qualified rope access technicians require a high level of supervision by experienced operators. Appropriate ratios of inexperienced to experienced operators should be carefully managed in hazardous work environments.

IRAANZ and IRATA may be best placed to take the lead and develop a framework to formalize a register of rope access technicians trained and experienced in geotechnical work, and more specifically disaster response and recovery work.

Planning the Transition from Response to Recovery

Disaster recovery starts while the response is still active. The transition is a process, which should be planned, documented and communicated (CDEM Act 2002). Increasing the capacity for planning the transition, by adding personnel and technical assistance is a solution to the tension between speed and deliberation that exists in disaster reconstruction (Johnson & Olshansky 2016).

Workers require information and support during the transition, as the priorities for action differ during these phases. What is considered to be a tolerable level of risk for workers may change after a disaster. In the early phases of a response it may be appropriate to take risks if there is a reasonable chance of saving lives. At some point an incident must transition to recovery. At this point it is no longer considered appropriate to allow workers to expose themselves to significant risks to perform duties (Jackson et al. 2002). Where significant risks exist, adequate mitigation and protection must be in place according to law (HSW Act 2015).

Decision Making Support

Following disasters, decisions are made which may differ from those made during non-crisis (business as usual) situations. Crisis decisions are made in high-risk/low operating time environment with large uncertainties. Decision support mechanisms for regulators and operators have been shown to contribute to the prevention of major accidents (Sorskar & Abrahamsen 2017).

A suggested approach to clarify decision-making at practitioner level is based on the Fire and Emergency NZ (FENZ) "safe person" concept where an emergency responder "will, may, or will not" take risks, depending on the context (Fig. 8). The regulatory decision-maker should be able to take a dynamic approach, i.e. be able to give weight to an extreme economic perspective, or an extreme safety perspective, or a perspective on a continuum in between (according to the adopted risk management approach and the context) (Sorskar & Abrahamsen 2017) (Fig. 9).

COMMUNICATION CHANNELS BETWEEN SCIENTISTS, ENGINEERS AND CONTRACTORS

Knowledge and information sharing are critically important for successful worker protection after a disaster (APHA

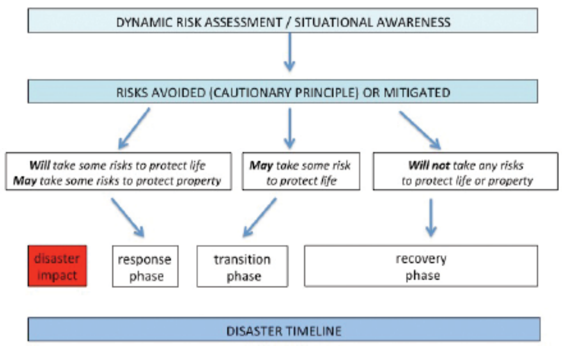

Figure 8: Suggested decision-making approach for operators during different phases of disaster Practitioners require communication and support from regulators during transition between the phases. Where clear boundaries do not exist, risk levels and accident rates should be trending down. Image adapted from (FEMNZ 2017).

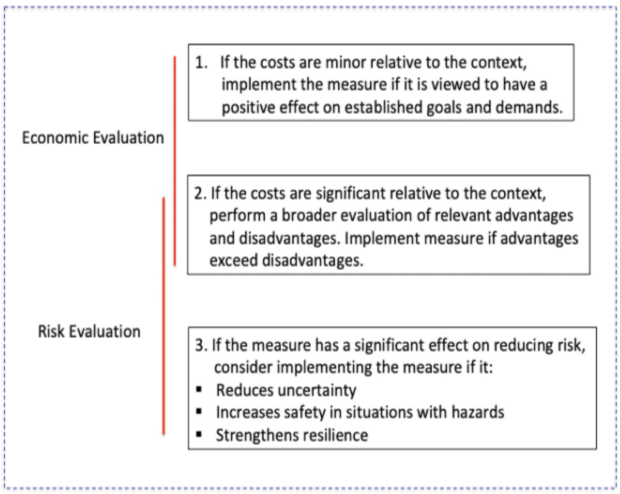

Figure 9: Suggested decision-making approach for regulators and stakeholders during times of uncertainty. A dynamic approach is recommended, giving weight to either an economic or safety perspective, depending on the context, Image adapted from (Sorskar & Abrahamsen 2017).

2008; GAO 2007; Jackson et al. 2002; Sim 2011). Assessing risk should be an inclusive process involving workers representatives, experts, stakeholders and technical partners. Experts involved in risk assessments will need to collaborate beyond professional boundaries during recovery (Rovins et al. 2015).

An organizational model where the fast pace of disaster reconstruction is matched by equally fast-paced development of safety culture in organizations involved, would ensure that health and safety "good practice" can be implemented at all times in future disasters in New Zealand. This may be achieved through a communication network that transfers risk information quickly through the different levels of a complex organization, providing communication channels between recovery actors to facilitate information transfer, inclusion and transparency about known risks.

CONCLUSIONS

Following the Kaikōura Earthquake, the New Zealand Government, stakeholders, consultants and contractors faced considerable uncertainty and had to balance the tensions between urgency to rebuild and the need for due care with regard to health and safety concerns. Rope Access workers played an important role in the emergency response and recovery, reducing the long-term risk post-earthquake. The Kaikōura reconstruction has highlighted the difficulty in maintaining a balance between reducing the risk to workers to tolerable levels and allowing nationally important strategic recovery works to proceed rapidly. Mitigating the risk to life safety is of upmost importance, although this will be balanced with contextual factors such as economic losses, political priorities, the well-being of affected communities and the increased risks associated with the use of alternate transport routes, especially where the distributed transport system lacks redundancy.

Improvements in the management of safety for reconstruction workers will allow for more effective and efficient recovery in future natural hazard events which affect critical lifelines and infrastructure, thus improving the resilience of transportation networks and communities in New Zealand. This report distils lessons from the Kaikōura reconstruction, where significant disaster recovery challenges led to different management approaches being used to reduce risk.

The key findings of this study were:

- There was no formal process to effectively communicate information about hazards and risk to workers at field level.
- The planning and development of emergency procedures was often in a constant state of "catch up" to keep pace with the rapid reconstruction work.
- Additional training reduces risk for workers.
- Workers require support, communication and clarification of expectations from stakeholders and government during the transition from response to recovery.
- In Kaikōura the additional risks posed by ongoing seismicity and secondary natural hazards required additional input from experts with knowledge and experience in assessing, quantifying and managing these types of risks.

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Correlation between standard accelerated UV tests and onsite UV degradation for high strength woven polyester reinforcement geotextiles in New Zealand

ABSTRACT

Ultraviolet (UV) radiation is critical to geosynthetic resins. It causes polymer bond breakage leading to loss of all properties including discoloration, tensile strength and tensile elongation. This paper presents the test results of outdoor weathering of a high strength woven polyester reinforcement geotextile at site locations in Auckland and Christchurch, New Zealand. The onsite exposed samples are tested for loss in tensile strength and elongation over time and compared with results of accelerated UV lab tests. The standard accelerated UV lab tests for geotextiles include the Xenon Arc Test according to ASTM D4355 and the QUV Test according to EN 12224. These results help establish some form of correlation between standard accelerated UV lab tests and onsite UV degradation of high strength woven polyester reinforcement geotextiles in New Zealand.

Keywords: Reinforcement geotextile, UV degradation, outdoor weathering, accelerated UV lab test

1. INTRODUCTION

Ultraviolet (UV) radiation is electromagnetic radiation of wavelengths 100 nm to 400 nm, while visible light is electromagnetic radiation of wavelengths 400 nm to 700 nm and infrared radiation consist of wavelengths from 700 nm to 1 mm. The Sun emits a combination of UV radiation, visible light and infrared radiation; some of which is absorbed by the Earth's thick atmosphere. UV radiation is subdivided into UV-A (315 nm to 400 nm), UV-B (280 nm to 315 nm) and UV-C (100 nm to 280 nm) radiation. The UV radiation that reaches the Earth's surface is largely UV-A radiation with a small amount of UV-B radiation.

The exposure of a geosynthetic material to UV radiation generally has a negative impact on its intended engineering performance. It causes polymer bond to break or scission of main polymer chain, resulting in property changes over the exposure period (CUR 243, 2012). This is especially important when it concerns reinforcement applications because the material loses tensile strength and becomes more brittle. The general practice is to cover the reinforcement material, typically with a layer of soil cover, within a short period of time to minimise the loss of tensile strength and embrittlement. Very often questions are raised concerning how long is a reasonable and practical time limit to allow for before it should or can be covered up and how much of tensile strength loss and embrittlement happens during that period of exposure.

Accelerated UV lab test standards are available to benchmark the performance of geosynthetics against UV degradation. These accelerated tests that are set on standard artificially generated UV spectrum and intensity, are important in determining comparative UV degradation performance of geosynthetics but they have limitations for a variety of reasons. The UV radiation levels incident onsite is dependent on its geographical latitude and altitude; climatic and weather conditions; ground shading

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Auckland yard	Christchurch yard
Address: 14 Goodman Place, Penrose	Address: 24 George Bellew Road, Harewood
Co-ordinates: 36° 55' 07" S 174° 48' 28" E	Co-ordinates: 43° 29' 53" S 172° 31' 52" E
	

The actual locations of the test are shown with ●

Figure 1: Two sites selected for the outdoor exposure test.

and reflectivity conditions; just to name a few. As such correlation between actual onsite exposure and standard lab test is a difficult subject. However, when such standard tests are calibrated with onsite UV degradation tests, they can then be used in a reliable way to predict UV degradation resistance under similar prevailing conditions.

It was with this in mind that a testing program was undertaken in New Zealand. A woven polyester geotextile of nominated ultimate tensile strength of 1000 kN/m in the machine direction (MD) and 100 kN/m in the cross direction (CD), commonly used for embankment basal reinforcement, was chosen as the test specimen. Controlled sites free from external disturbances were chosen for the exposure sites. Auckland was chosen as the representative location for North Island conditions while Christchurch was chosen as the representative location for South Island.

2. ONSITE WEATHERING TEST PROTOCOL

The protocol for the outdoor weathering exposure considered sampling of the high strength woven polyester reinforcement geotextile (Mirafi® PET1000/100) for outdoor weathering exposure for 5, 14, 28 and 42 days. The selected exposure locations are within secured compounds to rule out disturbance or damage due to external factors. Samples are laid basically flat to reflect the geotextile laid out condition at construction site.

2.1 Sites for outdoor exposure

Figure 1 shows the two sites selected for the outdoor exposure of the geotextile samples.

2.2 Sampling and exposure procedure

The following procedure was followed:

- The high strength woven polyester reinforcement geotextile samples were pre-cut into 2 m wide x 1 m long panels at the factory and wrapped with protective wrapping to protect the samples from sunlight and transportation damage.
- A set of 5 samples for each location was prepared which included 4 samples for exposure testing at the designated site and 1 sample kept wrapped to be used for the control.
- At each site a 200 g/m² nonwoven geotextile was first placed on plywood boards followed by the high strength woven polyester reinforcement geotextile samples and fixed down on a plywood board. Each sample was clearly identified.
- The boards were then placed in locations where they were exposed to the maximum amount of sunlight for the exposed period.
- The samples were left to be exposed to rain and sunlight during exposure period at each site.
- The location chosen were away from warehouse traffic to ensure the samples were free from any disturbance that could cause damage to the geotextile.
- After each exposure period, the sample was carefully removed, folded and wrapped with the plastic wrapper to be sent to the laboratory for testing.

Figure 2 shows the geotextile sample preparation process for exposure at the Auckland yard.

Figure 2: Geotextile sample preparation process for exposure at the Auckland yard.

Sample No.	Exposure			Peak tensile strength (MD)		Strain at peak strength (MD)	
	Start	End	(days)	(kN/m)	(% retained)	(%)	(% retained)
1	-	-	0	1099.4	-	11.5	-
2	24/3/2017	29/3/2017	5	1101.2	100.2	11.1	98.7
3	24/3/2017	7/4/2017	14	1087.4	98.9	10.9	98.7
4	24/3/2017	21/4/2017	28	1023.0	93.0	10.7	83.8
5	24/3/2017	5/4/2017	42	998.9	90.9	10.4	83.7

Table 1: Test results of MD peak tensile strength and strain at peak strength for Auckland exposed samples.

Sample No.	Exposure			Peak tensile load (MD)		Strain at peak load (MD)	
	Start	End	(days)	(kN/m)	(% retained)	(%)	(% retained)
6	-	-	0	1095.5	-	11.5	-
7	8/6/2017	13/6/2017	5	1103.2	100.7	11.1	97.2
8	8/6/2017	22/6/2017	14	1089.9	99.5	10.9	95.3
9	5/5/2017	7/6/2017	33	1079.0	98.5	10.7	93.7
10	5/5/2017	16/6/2017	42	1054.6	96.3	10.4	90.9

Table 2: Test results of MD peak tensile strength and strain at peak strength for Christchurch exposed samples.

3. TEST RESULTS

The onsite exposed samples were tested according to ISO10319 at a GAI-LAP accredited lab. Accelerated UV tests which included the Xenon Arc Test according to ASTM D4355 and the QUV Test according to EN 12224 were conducted to establish correlation between them and onsite UV degradation.

3.1 Real time exposure test results

Table 1 shows the test results of MD peak tensile strength and strain at peak strength for the samples exposed in Auckland. Table 2 shows the test results of MD peak tensile strength and strain at peak strength for the samples exposed in Christchurch.

The peak tensile strength retention after 42 days of exposure was 91% for sample exposed in Auckland (see Table 1) while the peak tensile strength retention after 42 days of exposure was 96% for sample exposed in Christchurch (see Table 2). For reinforcement geotextiles it may be more relevant to understand the retained strength at working strain levels than at peak levels. For embankment basal reinforcement applications, working strain levels typically may vary between 2% to 5%.

Table 3 shows the test results of MD tensile strengths at 2% and 5% strains for the samples exposed in Auckland. Table 4 shows the test results of MD tensile strengths at 2% and 5% strains for the samples exposed in Christchurch. At strain levels of 2% and 5%, it appears that there has been no measured loss in tensile strengths for up to 42 days of samples exposure at both Auckland and Christchurch (see Tables 3 and 4), even though a drop in peak tensile strength over time is seen.

3.2 Recorded UV Data

Data from the National Institute of Water and Atmospheric Research (NIWA) ground based measuring sites located in Auckland and Christchurch has been used to compare the strength loss in the samples to the measured radiant energy for the period of exposure for each site. Solar radiation contains three types of measurement namely global, diffuse and direct. The global solar radiation measured in MJ/m² which includes both radiation from

Sample No.	Exposure	Tensile strength at 2% strain (MD)		Tensile strength at 5% strain (MD)	
	(days)	(kN/m)	(% retained)	(kN/m)	(% retained)
1	0	153.9	-	388.7	-
2	5	165.6	107.6	397.9	102.4
3	14	168.2	109.3	404.0	103.9
4	28	191.0	124.1	433.3	111.5
5	42	162.5	105.6	394.8	101.6

Table 3: Test results of MD tensile strength at 2% and 5% strains for Auckland exposed samples.

Sample No.	Exposure	Tensile strength at 2% strain (MD)		Tensile strength at 5% strain (MD)	
	(days)	(kN/m)	(% retained)	(kN/m)	(% retained)
6	0	176.3	-	415.1	-
7	5	186.1	105.6	431.4	103.9
8	14	187.4	106.3	435.6	104.9
9	33	177.3	100.6	425.3	102.5
10	42	182.1	103.3	435.7	105.0

Table 4: Test results of MD tensile strength at 2% and 5% strains for Christchurch exposed samples.

direct sunlight and from diffuse (scattering) sources in the earth's atmosphere such as clouds was obtained from NIWA hosted New Zealand's National Climate Database. The details for the solar radiation experienced at each site are as follows:

- Auckland (see Figure 3):

Latitude -36.96177dec.deg, Longitude 174.7764dec.deg, Elevation = 5m

Measuring period 24th March to 4th May

- Christchurch (see Figure 4):

Latitude -43.53074dec.deg, Longitude 172.60769dec.deg, Elevation = 6m

Measuring period 5th May to 15th June for 33 day and 42 day samples

Measuring period 8th June to 22nd June for 7 day and 14 day samples

The graphs clearly show higher level of solar radiation at the Auckland site when compared with the Christchurch site where the testing occurred later into the autumn months. The 5 day and 14 day testing in Christchurch occurred towards the end of the 42 day test resulting in lower radiation values for this period late into autumn. Sunshine hours have also been taken measured and the results shown in Table 5.

The results in Table 5 above indicated that there is no meaningful relationship between recorded sunshine hours and solar radiation.

3.3 Accelerated UV test results

The samples used for the accelerated UV tests were taken from the same geotextile roll as those used for onsite weathering tests. Both the accelerated UV tests were conducted at GAI-LAP accredited labs. Table 6 shows the test results of peak tensile load and strain at peak

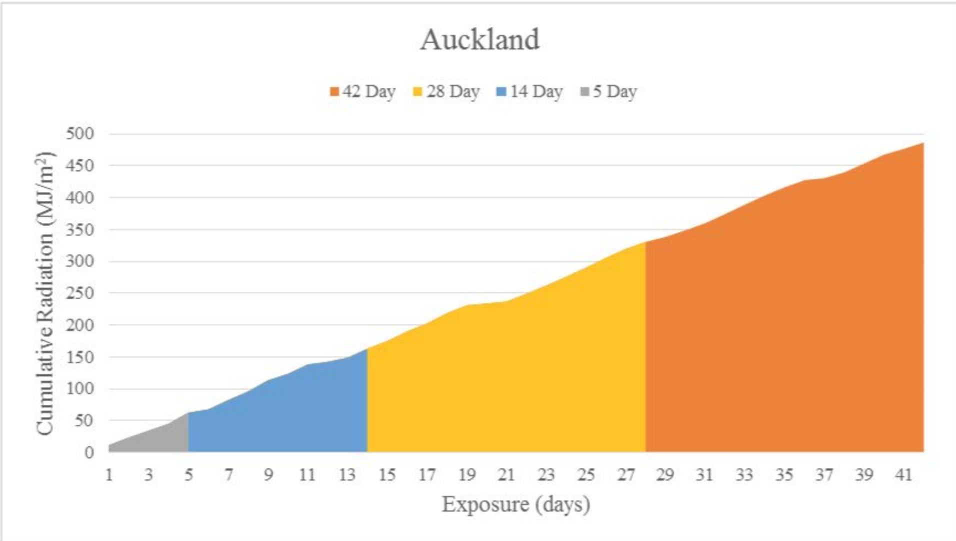

Figure 3: Solar radiation experienced at Auckland site over days of exposure.

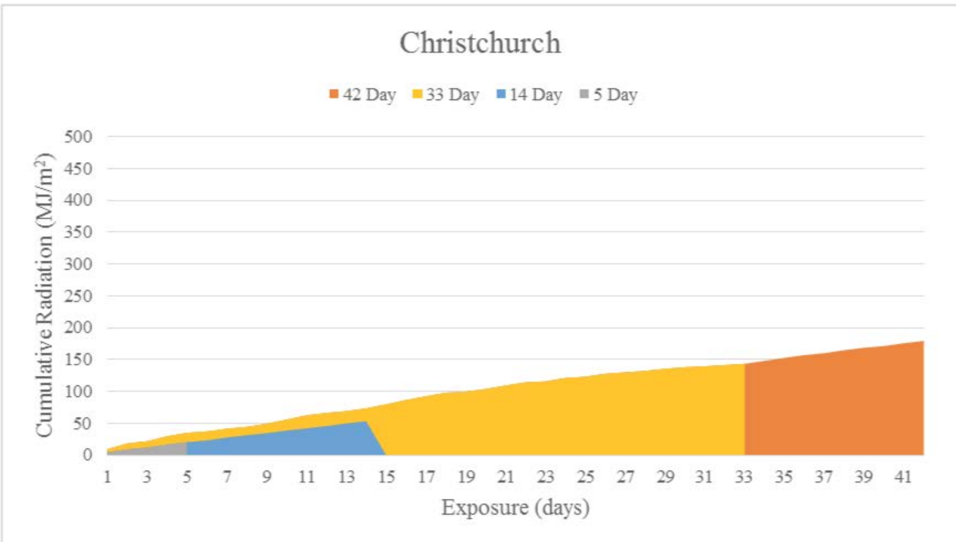

Figure 4: Solar radiation experienced at Christchurch site over days of exposure.

	Auckland Site				Christchurch Site			
Exposure Days	5	14	28	42	5	14	33	42
Sunshine Hours	43.8	95.8	144.1	225.2	32.7	73.1	113.6	167.4
Average (hr)	8.76	6.84	5.15	5.36	6.54	5.22	3.44	3.99

Table 5: Results of sunshine hours for each site for the test period.

Sample No.	Exposure	Peak tensile load		Strain at peak load	
	(hours)	(kN/m)	(% retained)	(%)	(% retained)
11	0	1212.1	-	9.6	-
12	50	1082.7	89.3	10.5	109.4
13	100	999.0	82.4	9.2	95.8
14	200	821.2	67.8	9.0	93.8
15	300	790.5	65.2	8.8	91.7
16	500	684.6	56.5	8.0	83.3

Table 6: Xenon Arc accelerated test results according to ASTM D4355.

Sample No.	Exposure	Peak tensile load		Strain at peak load	
	(hours)	(kN/m)	(% retained)	(%)	(% retained)
17	**0**	**1279.7**	**-**	**10.7**	**-**
18	**50**	**1124.3**	**87.9**	**9.8**	**91.6**
19	**100**	**1017.3**	**79.5**	**8.9**	**83.2**
20	**200**	**875.4**	**68.4**	**8.9**	**83.2**
21	**300**	**790.6**	**61.8**	**8.5**	**79.4**
22	**430**	**753.2**	**58.9**	**8.9**	**83.2**

Table 7: QUV accelerated test results according to EN12224.

load for the Xenon Arc accelerated UV test conducted according to ASTM D 4355. Table 7 shows the test results of peak tensile load and strain at peak load for the QUV accelerated UV test conducted according to EN 12224.

4. DISCUSSION

The standard accelerated UV lab tests for geotextiles typically use the peak tensile strength for relating the loss in tensile strength with exposure time. As such for correlating these standard accelerated UV lab tests with real time onsite weathering characteristics, the peak tensile strength will be used as the benchmark. For the real time onsite weathering exposures, the late summer into autumn season, which coincides with the peak construction period in New Zealand, was chosen for the exposure to represent an average condition for the exposure intensity. Figure 5 shows the residual strength (as a % of the original peak tensile strength) versus exposure days for real time onsite weathering exposures in Christchurch and Auckland and the accelerated UV tests according to ASTM D4355 and EN 12224.

For reinforcement geotextile applications the recommendation for construction period weathering exposure limit is generally based on evidence showing little or no loss in tensile strength over the recommended exposure. If for unexpected circumstances the exposure limit is exceeded, then correlation factors established between accelerated UV tests and onsite UV degradation tests may be used to estimate residual strength.

The rate of tensile strength reduction with time for all tests appear to be linear over the duration of tests. The rate of strength reduction for samples subjected onsite weathering in Christchurch is 0.1% per day while the rate of strength reduction for samples subjected onsite weathering in Auckland is 0.25% per day (see Figure 5). The rate of strength reduction for samples subjected to the accelerated UV test according to ASTM D4355 is 2.7% per day while the rate of strength reduction for samples subjected to the accelerated UV test according to EN 12224 is 3% per day (see Figure 5).

Therefore, the accelerated UV test according to ASTM D4355 corresponds to acceleration factors of 27 and 11, when correlated with the real time onsite weathering in Auckland and Christchurch respectively. And the accelerated UV test according to EN 12224 corresponds to acceleration factors of 30 and 12, when correlated with the real time onsite weathering in Auckland and Christchurch respectively.

The cumulative 42 day solar radiation was measured at 486.71 MJ/m^2 and 179.48 MJ/m^2 for Auckland and Christchurch respectively. This ratio of 2.7 in increased solar radiation for Auckland compares favorably to the ratio for onsite weathering rates for the two sites over the same 42 day period of 2.5.

The loss in peak tensile strength over 14 days of real time onsite weathering exposure is 1.1% and 0.5% for Auckland and Christchurch (see Tables 1 and 2). The loss in strain at peak tensile strength over 14 days of real time onsite weathering exposure is within 5% for both Auckland and Christchurch (see Tables 1 and 2). This shows that there is hardly any significant loss of peak tensile strength for a weathering exposure of 14 days for both Auckland and Christchurch. The loss of less than 5% of strain at peak tensile load also shows that embrittlement is not a concern for a weathering exposure of 14 days for both Auckland and Christchurch. Furthermore, there is no apparent loss in tensile strength at working strain levels of between 2% and 5% for up to 42 days of weathering exposure for both Auckland and Christchurch (see Tables 3 and 4). The results therefore support the general recommendation of construction period weathering exposure of not more than 14 days in New Zealand for the concerned geotextile may be allowed. The prudent practice however is to cover the reinforcement geotextile material, typically with a layer of soil cover, as quickly as possible on the construction site.

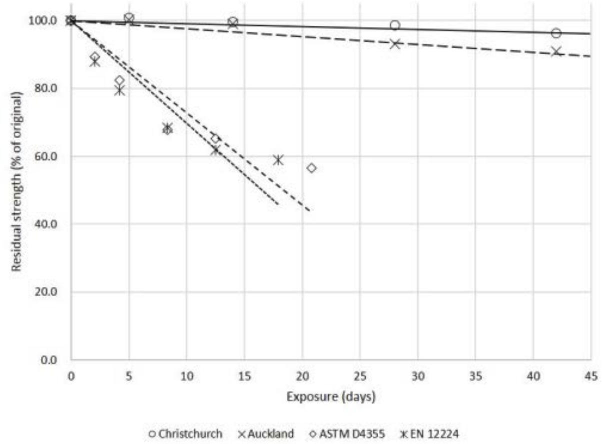

Figure 5: Residual strength versus exposure days for real time exposures and accelerated UV tests.

5. CONCLUSIONS

The rate of strength reduction for the concerned geotextile samples subjected to onsite weathering in early autumn is 0.25% per day and 0.1% per day for Auckland and Christchurch respectively. The accelerated UV test according to ASTM D4355 corresponds to typical acceleration factors of 27 and 11 when correlated with the real time onsite weathering for the concerned geotextile in Auckland and Christchurch respectively. The accelerated UV test according to EN 12224 corresponds to typical acceleration factors of 30 and 12 when correlated with the real time onsite weathering for the concerned geotextile in Auckland and Christchurch respectively. For the concerned geotextile, a construction period weathering exposure of not more than 14 days in New Zealand is typically acceptable.

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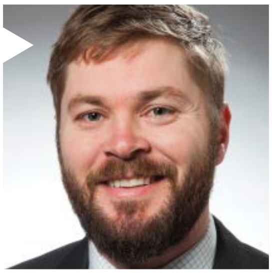
Development of an Empirical Correlation for Predicting Shear Wave Velocity of Christchurch Soils from Cone Penetration Test Data



Dr McGann, a senior lecturer at the University of Canterbury, received his BS from Montana State University in 2004, and his MS and PhD from the University of Washington in 2009 and 2013. His research activities include regional geotechnical site characterisation, finite element technology, detailed geotechnical numerical modelling, and soil-foundation-structure interaction.

Merrick L Taylor
Merrick Taylor is a chartered geotechnical engineer based in Auckland with 12 years industry experience working on a wide range of New Zealand and international projects with consultants Arup and Beca. He has a BE(Hons) and PhD in Civil Engineering from the University of Canterbury, and MSc from Imperial College London, and has an interest and developed expertise in seismic hazard assessment and earthquake geotechnical engineering.

Brendon Bradley
Brendon Bradley is an Associate Professor at the University of Canterbury with experience in seismic hazard and ground motion analysis, structural and geotechnical response analysis, and seismic risk assessment. His recent awards include: 2014 New Zealand Young Engineer of the Year Award; and the 2014 Shamsher Prakash Foundation Research Award.

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ABSTACT
Following the companion study of McGann et al. [1], seismic piezocone (SCPTu) data compiled from sites in Christchurch, New Zealand area are used with multiple linear regression to develop a Christchurch-specific empirical correlation for use in predicting soil shear wave velocities, Vs, from cone penetration test (CPT) data. An appropriate regression functional form is selected through an evaluation of the residuals for regression models developed with the Christchurch SCPTu database using functional forms adopted by previous empirical correlations between Vs and CPT data. An examination of how the residuals for the chosen regression form vary with the predictor variables identifies the need for non-constant depth variance in the regression model. The performance of the model is assessed through comparisons of predicted and observed Vs profiles and through forward predictions with synthetic CPT data. The new CPT-Vs correlation provides a method to estimate Vs from CPT data that is specific to the non-gravel soils of the Christchurch region in their current state (caution should be used for western portions of the Springston Formation where SCPTu data were sparse). The correlation also enables the utilization of the large, high-density database of CPT logs (> 15, 000 as of 1/1/2014) in the Christchurch region for the development of both site-specific and region-wide models of surficial Vs for use in site characterization and site response analysis.

KEYWORDS: cone penetration test (CPT), shear wave velocity, multiple linear regression, seismic piezo-cone (SCPTu), empirical correlation

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1 Introduction

Site effects related to the influence of near-surface (< 50 m) stratigraphy strongly affect observed surficial ground motions. Seismic waves must always pass through these near-surface soil and rock layers before reaching the surface, thus site effects tend to be a systematic feature of observed ground motions at a particular location, while path and source effects, which also strongly affect surficial ground motions, can vary significantly for earthquakes originating from different sources. The systematic nature of site effects at a particular site, in combination with the ready availability of direct measurements and estimates of the characteristics and properties of the near-surface soils, indicates that local site effects can be modeled with potentially greater accuracy than source and path effects and therefore, offer a potentially more efficient means with which to predict the character, and effects of, future surficial ground motions at specific locations [2–5].

The small strain shear modulus is a fundamental parameter required to evaluate the dynamic response of soil deposits using seismic site response analysis. It defines the shear stress-strain response for low levels of strain ($< 10^{-4}\%$), and is typically used to define normalized relationships describing the reduction in soil shear modulus with increasing levels of strain that is critical to nonlinear and equivalent linear site response analyses [e.g., 6, 7]. The small strain shear modulus is highly susceptible to disturbances that are nearly unavoidable in any laboratory assessment [8], therefore, in-situ measurements or estimates of shear wave velocity, V_s, which is related to the small strain shear modulus through the linear elastic wave propagation equation, are used to obtain the low strain soil shear stiffness profiles necessary for dynamic site response analyses.

Misko Cubrinovski

Misko is a professor in Geotechnical and Earthquake Engineering at the University of Canterbury, Christchurch. His research interests and expertise are in geotechnical earthquake engineering and in particular problems associated with liquefaction, seismic response of earth structures and soil-structure interaction. Misko is on the the leadership team of QuakeCoRE, NZ Centre for Earthquake Resilience, and is the leader of its Flagship Research Programme 2.

Near surface shear wave velocities can be directly estimated using surface wave measurement techniques such as the spectral and multi-channel analysis of surface waves techniques (SASW and MASW, respectively) [9, 10], linear and microtremor array methods [11–14], as well as by techniques requiring one or more boreholes such as crosshole and downhole techniques [15, 16] and P-S suspension logging measurements [17]. Near surface V_s profiles can also be obtained via empirical correlations with common geotechnical investigations such as the standard penetration test (SPT) [e.g., 18–21] and the cone penetration test (CPT) [e.g., 19, 21–27]. Such empirical correlations are typically developed through regression analysis using a series of predictor variables from the conventional geotechnical investigations (SPT or CPT) and V_s measurements obtained through one of the previously mentioned techniques. Many of the more recent empirical correlations have been developed using data for general soil deposits (i.e., cohesive and cohesionless) from globally located sites of varying geological ages in order to obtain prediction correlations that can be applied in a general manner.

Direct in-situ V_s measurements are preferable to the indirect V_s estimations obtained from the application of empirical correlations, however, they have disadvantages that limit their use in general practice. Surface wave methods are useful in that they are generally non-intrusive, but they require the solution of an inverse problem, which is often ill-posed. As a result, V_s profiles estimated using surface wave techniques often have problems related to the non-uniqueness of the solution [28] and to the equivalence problem [11, 29]. These issues can be manifested in the resulting profiles via decreased resolution with increasing depth, an inability to identify thin layers, and difficulties in resolving the portions of layers adjacent to large velocity contrasts [8]. Borehole-type measurement techniques are inherently invasive, though this in itself does not preclude their use, as invasive site characterization techniques (e.g., SPT and CPT) are common in practice. Compared to surface wave methods, borehole-type measurement techniques have greater capacity to resolve inclusions and anomalies that may be missed by surface-based approaches, but have increased temporal and financial expenses associated with drilling (especially for crosshole techniques, which require multiple boreholes) [8], and only represent the subsurface conditions at a single point, rather than the conditions averaged along a line. The downhole technique, of which the seismic cone penetration test (SCPT) is a specialized subset, requires only a single borehole, but can suffer from depth limits depending on the energy of the seismic wave source. The suspension logger test can be used for great depths (> 100 m), and is arguably the most precise invasive measurement method currently available, but this test has limited application in soft sediments [8].

The noted difficulties associated with direct V_s measurement techniques, along with the expenses related to the specialized equipment and training associated with their use in practice, typically results in their use only at certain higher-importance sites and a corresponding scarcity of V_s data available for region-wide subsurface characterization. In contrast, site investigation techniques such as SPT and CPT are commonly applied to a broader scope of projects, and empirical correlations based on the data obtained by such tests can be used to provide the V_s data necessary for both region-wide and site-specific ground response assessments. This is especially true in Christchurch, where the extensive site investigations made following the 2010-2011 Canterbury earthquake sequence [2, 30–34] have resulted in a large, high density database of CPT logs (> 15000 as of 1/1/2014) for sites located throughout Christchurch and the surrounding suburbs and towns.

This paper presents the development of an empirical correlation between CPT data and soil V_s. The paper extends on McGann et al. [1] who used a SCPTu database obtained from 86 sites in the greater Christchurch, New Zealand area to evaluate the suitability of four existing empirical models for estimating the in-situ V_s of Christchurch soils from CPT data. The existing CPT-V_s correlations [22–24] were shown to be biased towards overestimating the observed V_s profiles on average, with the Andrus et al. [22] providing the predictions with the least amount of prediction error. Reduction or loss of age effects was presented as one of the possible reasons for the observed overestimation bias in the existing correlations when applied to Christchurch soils, as the examined Christchurch sites do not display an increase in V_s with effective deposit age in line with that displayed by the existing Andrus et al. [22] data set. The observations discussed in McGann et al. [1] demonstrated the need for the development of a new Christchurch-specific CPT-V_s correlation through regression analysis, and this development is discussed in the current paper. Firstly, the development focuses on the selection of an appropriate functional form for the regression analysis. Secondly, the quality of the regression using the selected functional form is assessed, with particular attention given to the dependence of the model prediction and standard deviation on various predictor variables, and also direct comparison for selected profiles. The details of the correlation development are presented in the ensuing sections, followed by a discussion of the recommended Christchurch-specific CPT-V_s correlation determined through this process.

2 Evaluation of regression function forms

The first step in the development of a regression between CPT data and V_s is an assessment of potential functional forms. McGann et al. [1] examined the predictive capabilities of several existing CPT-V_s correlations to SCPTu data from the Christchurch region, and the functional forms of the those examined relationships form the basis of those considered herein. Specifically, six distinct relations between V_s and various CPT-based variables are considered as candidate regression functions. The considered regression forms include the Andrus et al. [22] form:

$$V_s = a{q_t}^b{I_c}^d z^e \tag{1}$$

where a, b, d, and e are regression coefficients, q_t is the cone tip resistance corrected for pore pressures acting behind the cone tip [35], I_c is the soil behaviour type index [36], and z is the depth; the Hegazy and Mayne [23] form rearranged to solve for shear wave velocity:

$$V_s = aQ_{tn}\exp(bI_c)\left(\frac{\sigma'_{v0}}{p_a}\right)^{0.25} \tag{2}$$

where Q_{tn} is the normalized cone resistance [36, 37], σ'_{v0} is vertical effective stress, and p_a is atmospheric pressure; the Robertson [24] form:

$$V_s = \left[10^{a+bI_c}\left(\frac{q_t - \sigma_{vo}}{p_a}\right)\right]^{0.5} \tag{3}$$

in which all terms are previously defined; the form recommended for use in CPT-V_s regression analysis by Wair et al. [21]:

$$V_s = a{q_t}^b{f_s}^d{\sigma'_{vo}}^e \tag{4}$$

where f_s is the cone frictional resistance; and two additional hybrid forms that consider different combinations of terms from the forms in Eqs. (1) and (4):

$$V_s = a{q_t}^b{f_s}^d z^e \tag{5}$$

$$V_s = a{q_t}^b{I_c}^d{\sigma'_{vo}}^e \tag{6}$$

Multiple linear regression in logarithmic space is used with the Christchurch SCPTu data set for each of the six considered regression forms. Fig. 1 shows a comparison between the measured V_s values from the database and the V_s values estimated using each considered regression form (indicated by equation number). The plots and associated coefficients of determination, r^2, shown in Fig. 1 indicate the relative compatibility of each regression form to the current data set. Based on these results, the Hegazy and Mayne [23] and Robertson [24] forms, Eqs. (2) and (3), respectively, appear to be the least applicable to the Christchurch data set, while the remaining four polynomial-based forms all provide a similar representation of the measured data.

The residuals for the fitted regression lines provide another means of evaluating the various regression forms. To this purpose, the residuals, ε, are defined as:

$$\varepsilon = \frac{\ln(y_i) - \ln(\bar{y}_i)}{s_{Y|x}} \tag{7}$$

where $\ln(\circ)$ is the natural logarithm function, y_i are the measured V_s values, $\bar{y}_i$ are the V_s values returned by the regression lines, and $s_{Y|x}$ is an estimate of the conditional standard deviation [e.g., 38, pp. 306-325]:

$$s_{Y|x} = \sqrt{\frac{\sum(\ln(y_i) - \ln(\bar{y}_i))^2}{n-4}} \tag{8}$$

where $n = 513$ is the number of data pairs included in the regression. Fig. 2 shows how the computed residuals vary with the depth, z, of the SCPTu data set, which showed the largest variation in residuals of the considered CPT variables. Plots showing the variation of the residuals with the remaining CPT-based variables, q_c, f_s, z, estimated V_s, and I_c are available in McGann et al. [39]. The marker colours in Fig. 2 represent the soil behaviour type index of each data point as indicated, while the black lines represent the moving averages (solid line) with 95% confidence intervals (dashed lines) [e.g., 40] for the residuals.

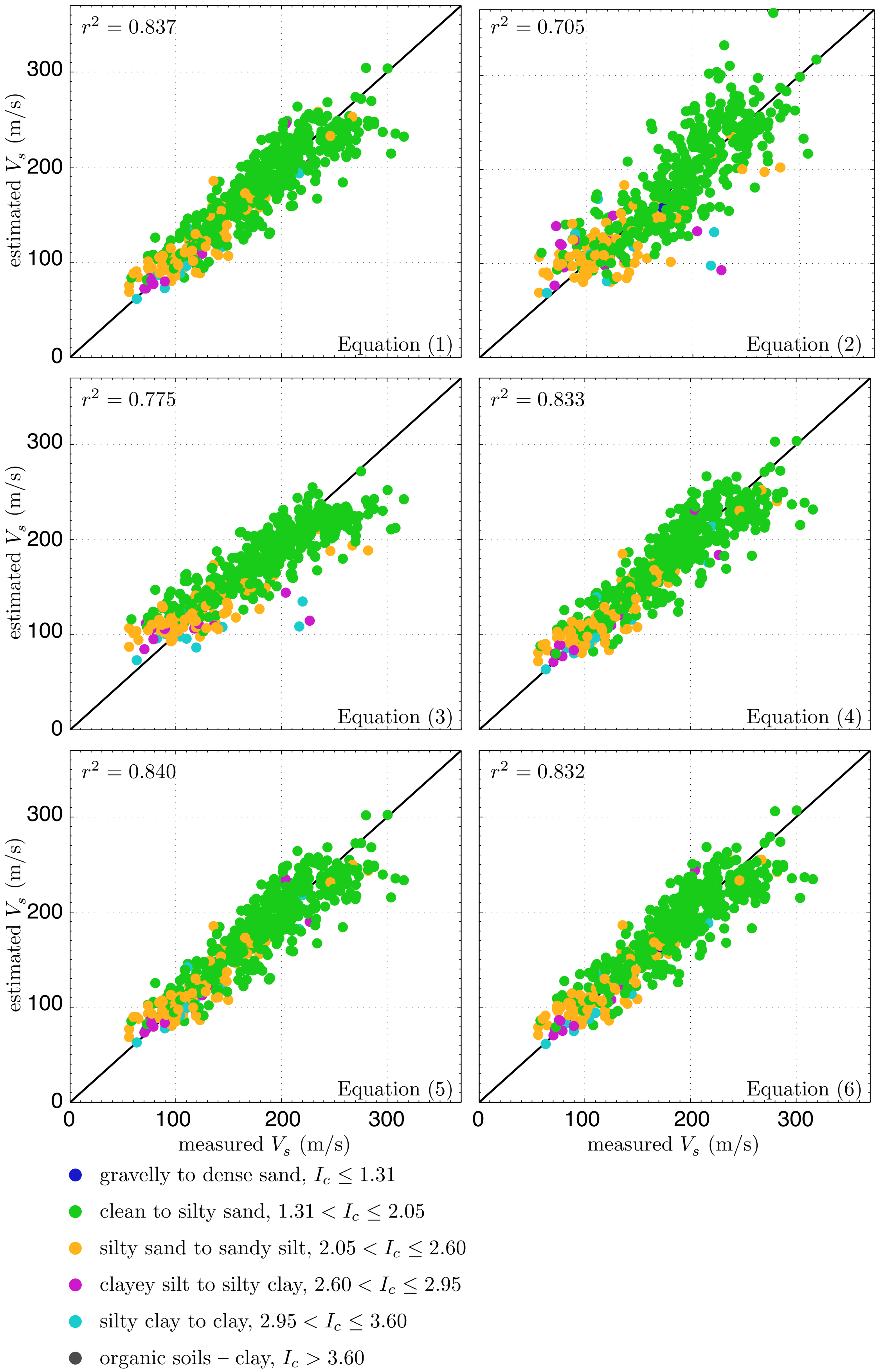

Fig. 1: Comparison of estimated and measured V_s for indicated regression functional forms. Marker colour indicates the I_c soil behaviour type index of the data points as noted.

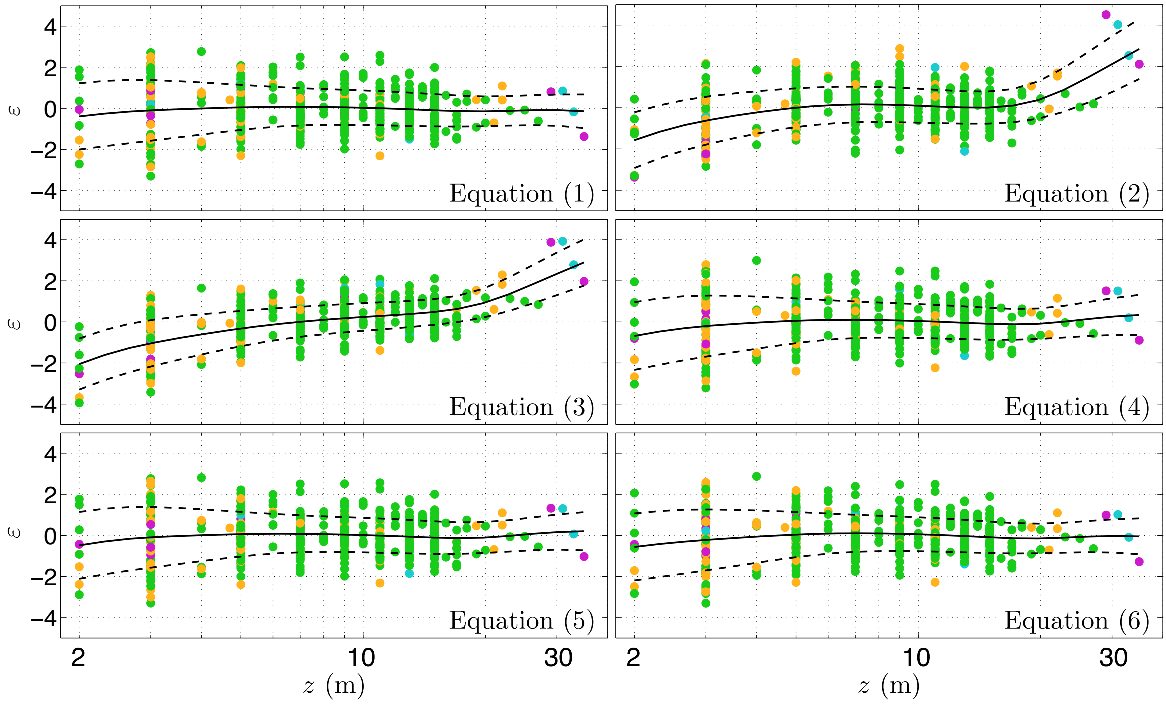

Fig. 2: Variation of residuals with depth, z, for indicated regression forms. Marker colour indicates the I_c soil behaviour type index of the data points as noted in Fig. 1.

As shown in Fig. 2, the regression forms given by Eqs. (1), (4), (5), and (6), the first two of which are the forms of Andrus et al. [22] and Wair et al. [21], respectively, produce reasonably consistent, and nearly identical, residual distributions across the range of depths in the database. In contrast, Eqs. (2) and (3), the Hegazy and Mayne [23] and Robertson [24] functional forms, display regions of concentrated bias, tend to overestimate the measured V_s data at shallow ($z < 4$ m) locations and underestimate the data at deeper locations ($z > 20$ m). The residual variation plots for the other CPT variables (not shown here, see [39]) show similar trends to those demonstrated in Fig. 2; the regression functional forms of Eqs. (2) and (3) return residuals that are biased towards under- or over-prediction of the measured V_s for certain ranges of q_c, f_s, estimated V_s, and I_c, while the remaining four regression forms produce residual distributions that are similar in form and consistent across the CPT-based variable ranges in the Christchurch-specific data set.

After ruling out the regression forms of Eqs. (2) and (3), the selection criteria for the most applicable functional form becomes more subtle. As shown in Figs. 1 and 2, the differences between the V_s estimates provided by the four remaining regression forms, Eqs. (1), (4), (5), and (6), are practically negligible over the principal ranges of the data set. Given this similarity in performance, consideration for the predictor variables included in the regression equations and how these variables affect the use of the resulting correlation becomes important. The first distinguishing characteristic between these four functional forms is the use of depth, z, or initial vertical effective stress, σ'_{v0}, as an indicator for the state of stress in the soil. From a theoretical standpoint, σ'_{v0} is preferable, however from a practical standpoint, depth may be a better choice. For a given site and CPT record, the depth is an inherently known quantity, while σ'_{v0} is typically estimated based on assumptions of soil mass density and groundwater table depth, and errors or uncertainties in estimated values for density, water table depth, and σ'_{v0} could lead to less reliable V_s predictions. This distinction is somewhat supported by the r^2 values provided in Fig. 1, which are slightly larger for the regression forms that consider depth, Eqs. (1) and (5), instead of effective stress, Eqs. (4) and (6).

The second decision relates to the use of f_s or I_c, as these terms are the only differences between the remaining two regression forms, Eqs. (1) and (5). Since each functional form appears to be equally applicable to the Christchurch data set by the measures presented here, the form given in Eq. (5) is chosen due to its use of f_s, which is directly measured by the CPT, instead of I_c, which is a computed variable (function of q_c, f_s, and depth). While I_c is a commonly-used indicator of soil behaviour type that carries useful connotations for many geotechnical engineers, its computation requires an additional step not required by f_s and its use in the CPT-V_s correlation may lead to erroneous predictions due to various available presentations of the I_c function or soil behaviour type zones [e.g., 36, 41, 42].

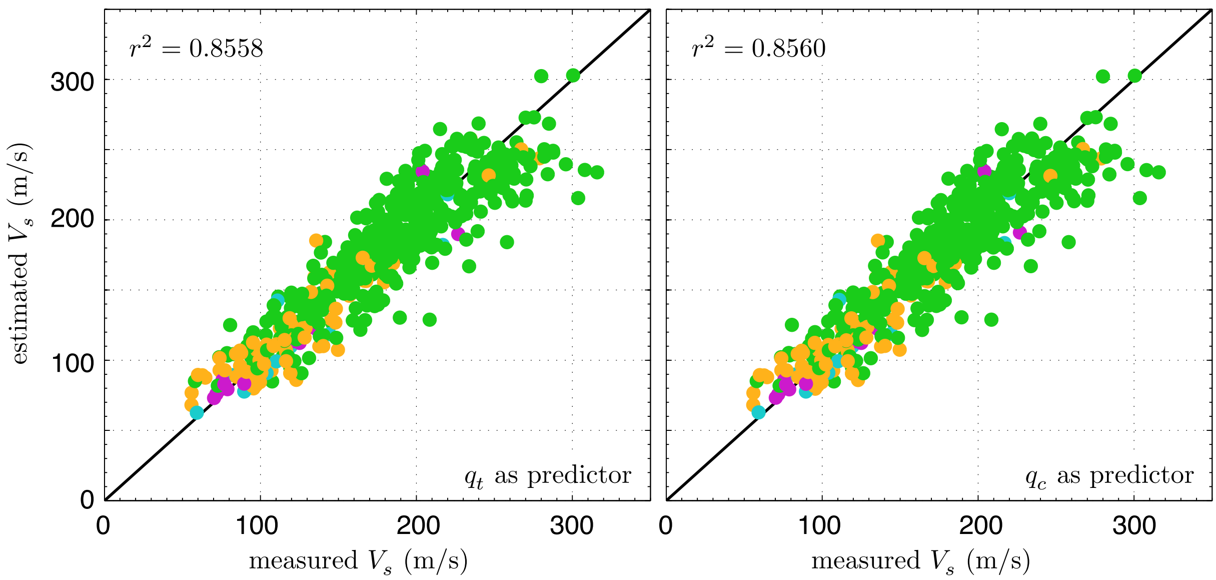

Fig. 3: Comparison of estimated and measured V_s for regression forms that consider corrected and raw cone tip resistance (q_t and q_c, respectively) as indicated. Marker colour indicates I_c as noted in Fig. 1.

It is also of interest to assess the effects of consideration for the raw cone tip resistance, q_c, instead of the pore pressure corrected tip resistance, q_t. As shown in Fig. 3, there is effectively zero difference in the quality of the regression when consideration is made for the raw cone resistance. The V_s values estimated using q_c as a predictor variable differ from those estimated using q_t by a maximum of 0.65%. This similarity makes sense in the context of the Christchurch SCPTu sites, which as discussed in [1], are predominantly composed of soils with I_c values in the clean to silty sand behaviour type zone where pore pressure readings are typically small (i.e., $q_c \approx q_t$). Given the similarity in results shown by Fig. 3, it is evident that for the considered soils, either q_c or q_t can be used in the correlation without significantly changing performance. The raw tip resistance may be used for non-piezocone data or for piezocone data with questionable (or highly uncertain) pore pressure data, as well as for cases where the cone specifications needed to compute q_t are unavailable. The pore pressure corrected resistance can be used in all other appropriate cases. The model equations and the plots in this paper and in [1] are shown in terms of q_c for consistency of presentation. Based on the evaluations made during this discussion, the chosen regression functional form is defined as (with q_c as the tip resistance term):

$$V_s = a{q_c}^b {f_s}^d z^e \tag{9}$$

This regression equation is based entirely on terms directly measured by the CPT (q_c, f_s, and z). These terms are the least uncertain quantities that can be considered in the regression function, and this study has shown that the quality of the regression to the considered Christchurch-specific data set is not significantly different when consideration is given to these directly measured terms.

3 Consideration for non-constant conditional variance

Fig. 4 summarizes the performance of the regression form given by Eq. (9) when applied to the Christchurch SCPTu database. As indicated by the moving average trend lines (solid black lines), the residuals for this correlation are relatively consistent across the considered ranges of CPT-based variables, though it is evident from the depth variation plot that there is some variance in the data set with changes in z, as the data points are more spread out and confidence intervals (dashed black lines) are wider for shallow locations. As discussed in further detail in McGann et al. [1], there are several potential reasons for the larger bias at shallow depths, including: the averaging approach used to compare the unequal CPT and V_s measurement intervals (i.e. potentially greater variability in soil composition, and CPT readings, near the surface); the difference in the inclination of the path travelled by the shear waves emanating from the source for shallow and deep locations; and the increased uncertainty in the shallow CPT measurements due to lack of confinement. During the existing correlation evaluation study discussed in McGann et al. [1], the suitability of the shallowest data points was assessed by considering comparisons with and without the data points with $z < 3$ m. It was determined that these points did not appear to unduly influence the overall bias of the comparisons, thus they were retained in the data set.

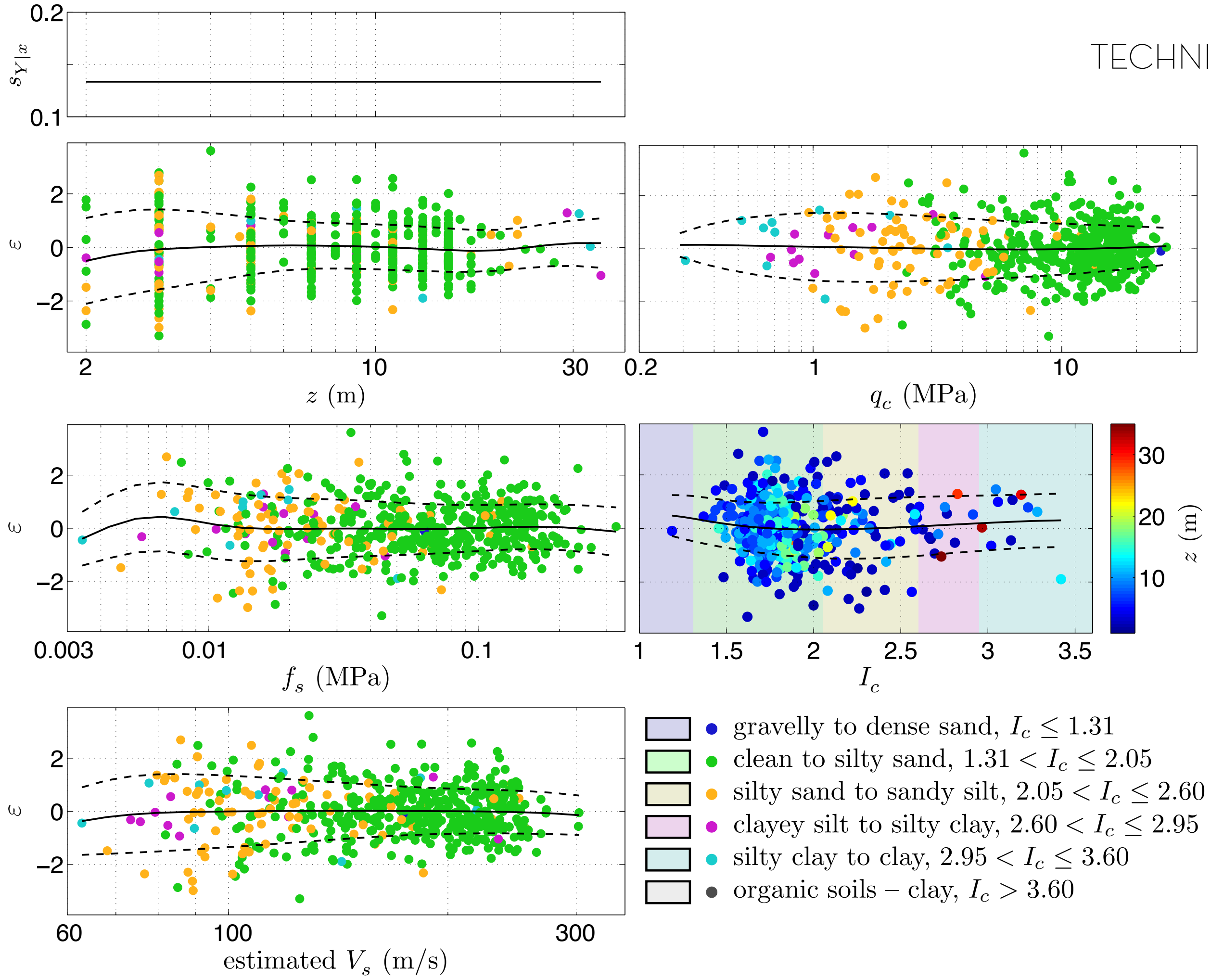

Fig. 4: Variation of residuals with z, q_c, f_s, I_c, and estimated V_s for multiple linear regression, using functional form of Eq. (9), with constant variance for all predictor variables. Marker colour indicates soil behaviour type index, I_c, (or depth, z, in the I_c subplot) as noted.

As indicated in the upper left-hand plot of Fig. 4, the presented results correspond to a regression analysis that considers constant conditional variance, hence constant conditional standard deviation, $s_{Y|x}$, with depth (and all other predictor variables). These results imply that consideration for a conditional variance that is non-constant with depth in the regression analysis could lead to an improved prediction of shear wave velocity. Fig. 5 presents the performance of the regression with consideration for a non-constant conditional variance that varies with depth in the manner shown in the corresponding upper left-hand plot. Based on an examination of the residuals, it was determined that the depth is the only predictor variable from Eq. (9) with which there was an apparent non-constant variance. As shown for $s_{Y|x}$ in the upper left plot of Fig. 5, a piecewise linear variation of variance with depth was considered as this was the simplest model appropriate for this purpose given the trends in the data. The depths at which the piecewise variance function changes slope ($z = 5$ m and $z = 10$ m) were manually chosen from a number of different depth ranges based on their effect on the regression results.

For the non-constant conditional variance model, the spread in the residuals evident in Fig. 4 at shallow depths becomes less pronounced and the overall distribution of the residuals with depth becomes more consistent in Fig. 5 (consistent in the sense that the distribution of the residuals is more even across the full range of depths in Fig. 5 as compared to Fig. 4). Because shallow depths typically correspond with lower shear wave velocities, the non-constant variance regression analysis naturally results in a tighter distribution of residuals at the lower ranges of the estimated V_s plot as well. Based on visual comparison of Figs. 4 and 5, the non-constant variance regression model produces more consistent residual distributions for all considered CPT-based variables. Though the confidence intervals for the moving average trend line are somewhat wider at deeper locations ($z \geq 10$ m) in the non-constant variance case, this model yields a smaller standard deviation ($s_{Y|x} = 0.108$) than the constant variance model ($s_{Y|x} = 0.136$) in addition to the more consistent residual distributions evident in Fig. 5.

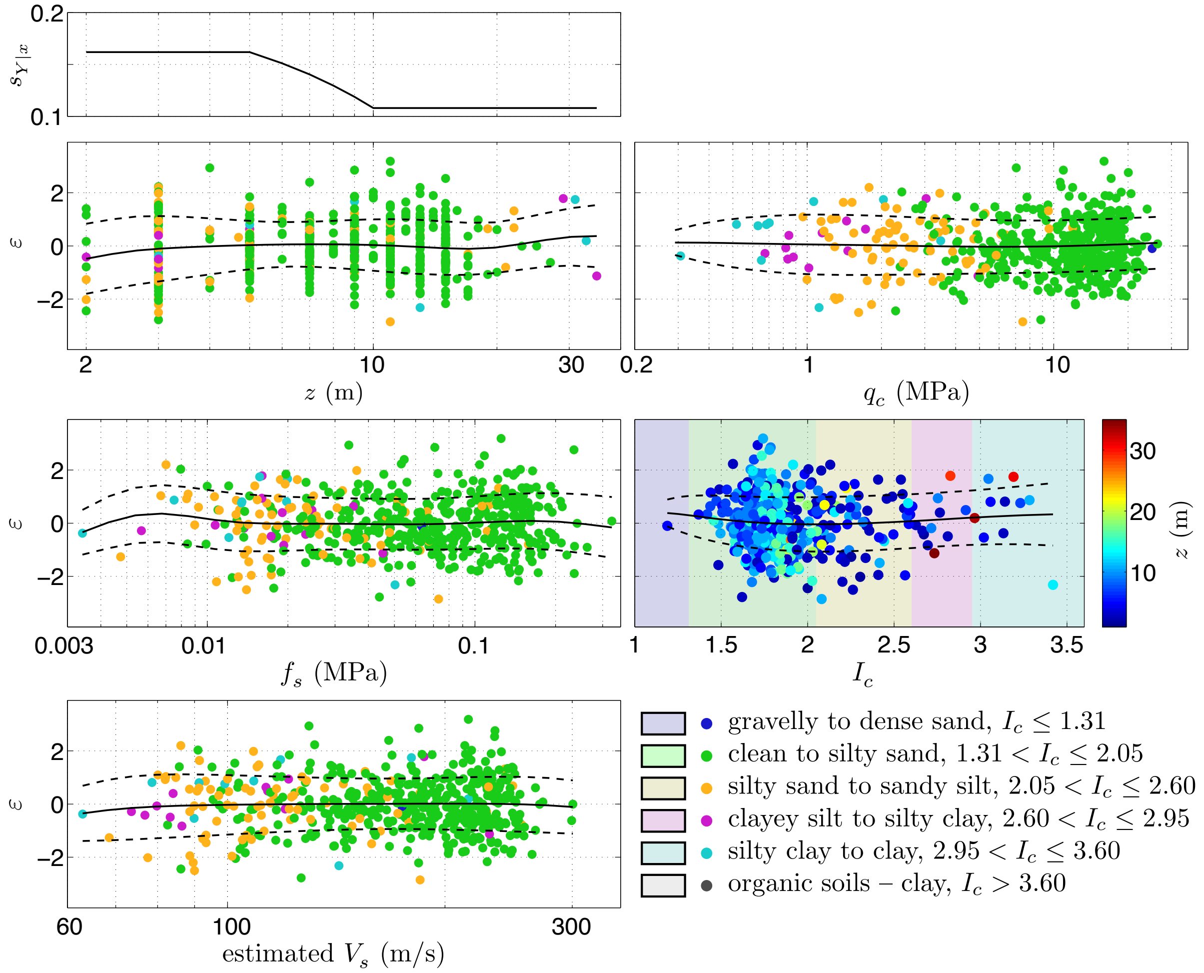

Fig. 5: Variation of residuals with z, q_c, f_s, I_c, and estimated V_s for multiple linear regression (using functional form of Eq. (9)) with non-constant depth variance. Marker colour indicates soil behaviour type index, I_c, (or depth, z, in the I_c subplot) as noted.

4 Final CPT-V_s model

4.1 Model equations

The Christchurch-specific CPT-V_s correlation is determined from multiple linear regression in natural log space using the non-constant variance distribution and functional form, Eq. (9), discussed in the previous sections. The recommended best-fit equation for predicting V_s from CPT data in the non-surficial gravel portions of the of the shallow Christchurch and Springston Formations [43] is:

$$V_s = 18.4\, q_c{}^{0.144} f_s{}^{0.0832} z^{0.278} \tag{10}$$

where q_c and f_s (units of kPa) are the raw cone tip and frictional resistances, respectively, z is the depth below the ground surface in metres, and V_s is the shear wave velocity in units of metres per second. Due to the near identical nature of q_c and q_t on average for the applicable soils (see Fig. 3), the pore pressure corrected tip resistance (q_t) may be used in place of q_c without change in the regression constants or performance. The piecewise standard deviation for the regression model is given by:

$$\sigma_{\ln(V_s)} = \begin{cases} 0.162 & \text{for } z \leq 5 \text{ m} \\ 0.216 - 0.0108z & \text{for } 5 \text{ m} < z < 10 \text{ m} \\ 0.108 & \text{for } z \geq 10 \text{ m} \end{cases} \tag{11}$$

from which the prediction of V_s for a given percentile can be obtained as:

$$V_{s\,x} = V_{s\,50} \exp\left(z_x \sigma_{\ln(V_s)}\right) \tag{12}$$

where x is the desired percentile, $V_{s\,50}$ is the median prediction given by Eq. (10), and z_x is the standard normal variate for the xth percentile (e.g., $z_x = 0, 1$ for the 50th and 84th percentiles, respectively). As shown by the plots of Fig. 5, the regression model of Eq. (10) produces consistent estimates of V_s across the full ranges of depth, cone and frictional resistance, and I_c soil behaviour type represented in the database, as well as for the full range of estimated V_s values returned by the correlation.

Support for the validity of the employed regression approach is provided by Fig. 6, which compares the cumulative probability distribution (CDF) for the analytical lognormal distribution with the CDF of the residuals, ε, for the empirical correlation computed using Eq. (7). The shown similarity between the empirical and theoretical cumulative distributions indicates that the data are lognormally distributed with respect to the prediction equation, providing confirmation that the normality assumption in the regression model, i.e., $\ln(V_s) = f(\ln(q_c), \ln(f_s), \ln(z))$, is appropriate for the data set.

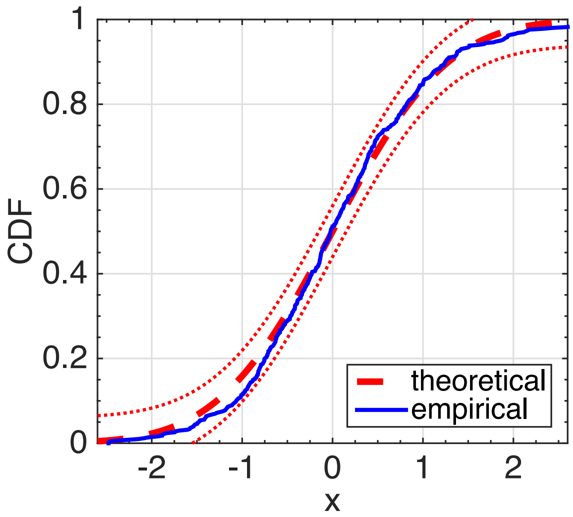

Fig. 6: Comparison of CDFs for empirical residuals and analytical lognormal distribution. Dotted lines represent Kolmogorov-Smirnov goodness-of-fit bounds [44] for $\alpha = 0.05$.

Fig. 7 shows the variation of the residuals with the factor of safety against liquefaction, $\mathrm{FS_{liq}}$, computed using the method of Idriss and Boulanger [45] for each data pair. As discussed in McGann et al. [1], the $\mathrm{FS_{liq}}$ of the considered soils was used to assess the potential reduction or loss of natural V_s increase with time due to the accumulation of age effects. In this context, lower $\mathrm{FS_{liq}}$ values are generally indicative of disturbed sites that may have lost age effects due to the 2010-2011 Canterbury earthquakes, and higher $\mathrm{FS_{liq}}$ values (larger than about 2.0) generally indicate that the age effects may be undisturbed. The background colours in Fig. 7 correspond to the likelihood of liquefaction classes of Taylor [46] (summarized in [1]), while marker colour corresponds to the soil behaviour type index. When the plotted $\mathrm{FS_{liq}}$ are interpreted as an indicator of apparent soil age as discussed in McGann et al. [1], Fig. 7 essentially indicates that the Christchurch-specific CPT-V_s correlation given by Eq. (10) performs equally well for the full range of liquefaction histories and apparent ages in the current data set.

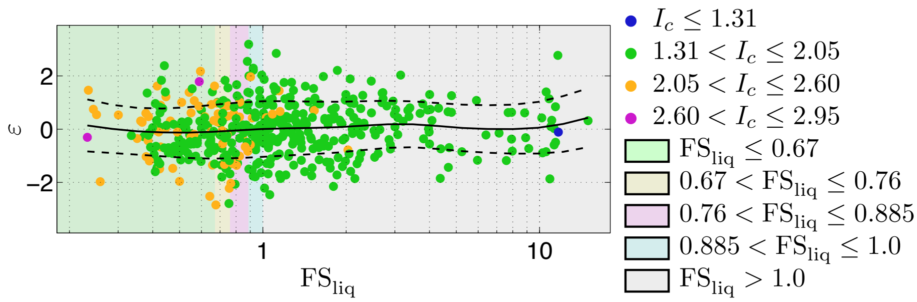

Fig. 7: Variation of residuals with estimated $\mathrm{FS_{liq}}$, for multiple linear regression with non-constant depth variance. Marker colour indicates I_c as indicated in Fig. 1. Background colour indicates liquefaction likelihood class [1, 46].

4.2 Comparisons with specific SCPTu profiles

Figs. 8–11 compare V_s profiles estimated using the Christchurch-specific CPT-V_s correlation of Eq. (10) with the measured V_s profiles for four sites in the Christchurch SCPTu database. The central solid line in the V_s plots of Figs. 8–11 defines the median estimated profile ($V_{s\,50}$), while the shaded region bounded by dashed lines indicates the shear wave velocities at +/− one standard deviation from the solid line (i.e., $V_{s\,16}$ and $V_{s\,84}$ from Eq. (12)). McGann et al. [39] provide similar plots for the full set of 86 SCPTu sites. The white circular markers representing the measured V_s values are plotted at the center of the typically 2 m thick measurement intervals used in the SCPTu soundings. The solid vertical lines associated with each marker indicate the thickness of the measurement intervals over which the measured V_s values are assumed to be constant.

The sites shown in Figs. 8 through 11 were selected to demonstrate the ability of the current regression model to handle a variety of soil behaviour types and soil stiffness conditions. Figs. 8 and 9 represent relatively stiff (higher q_c) soil sites that have the general behaviour types of a reasonably clean sand and a silty sand, respectively. Figs. 10 and 11 are representative of relatively soft (lower q_c) soil sites with the same two respective general soil behaviour types (clean and silty sand). As shown, the measured and estimated V_s profiles appear visually similar for all four site soil conditions, and the measured V_s values generally fall within one standard deviation from the median prediction for each site.

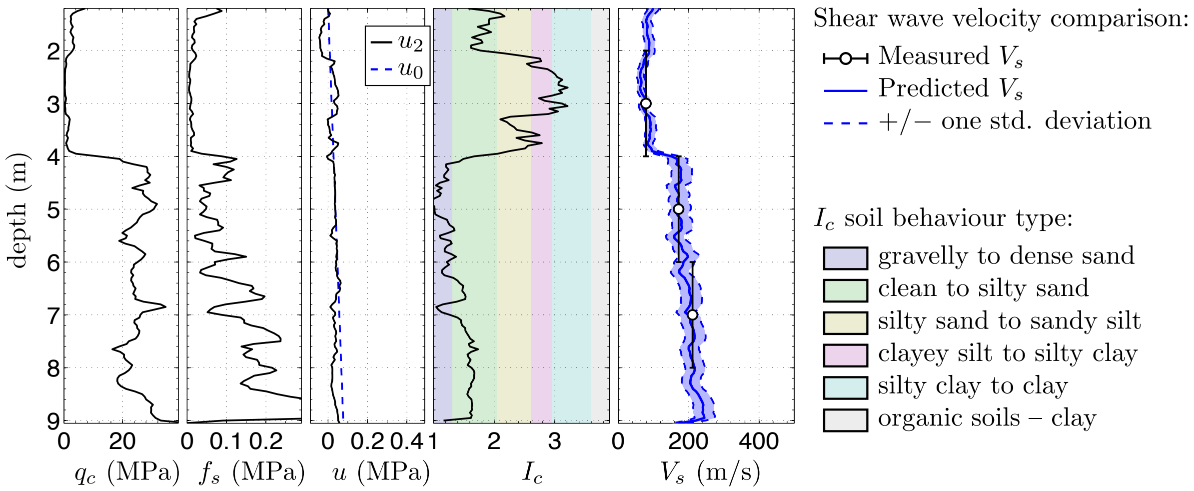

Fig. 8: Comparison of measured V_s values with Christchurch-specific CPT-V_s estimated profile for site KAS46, representative of a clean sand (lower I_c) soil site (for $z > 4$ m) with stiff (higher q_c) response. The groundwater table at this site was reported at 1.0 m depth.

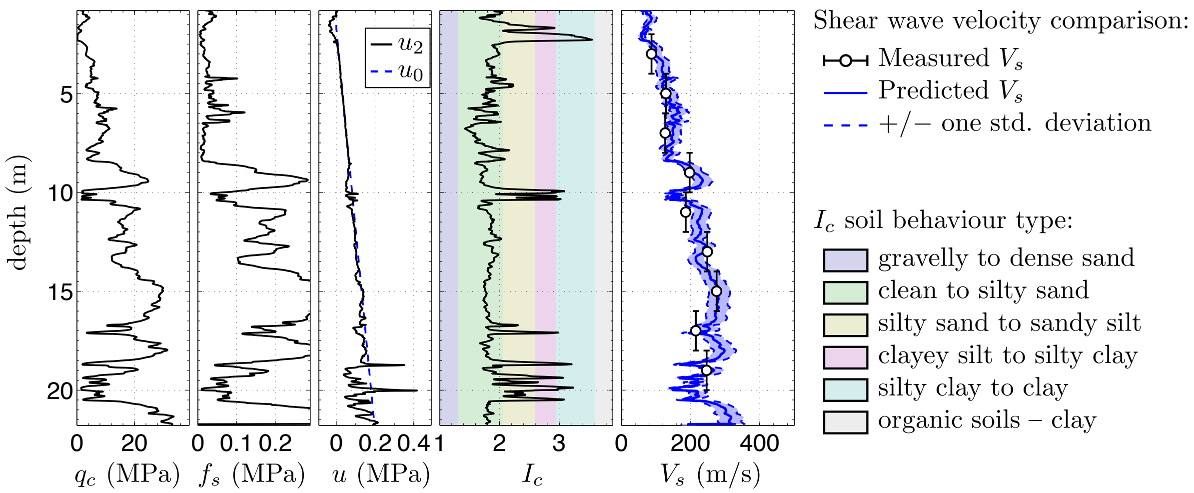

Fig. 9: Comparison of measured V_s values with Christchurch-specific CPT-V_s estimated profile for site WAI14, representative of a silty sand (higher I_c) soil site with stiff (higher q_c) response. The groundwater table at this site was reported at 1.7 m depth.

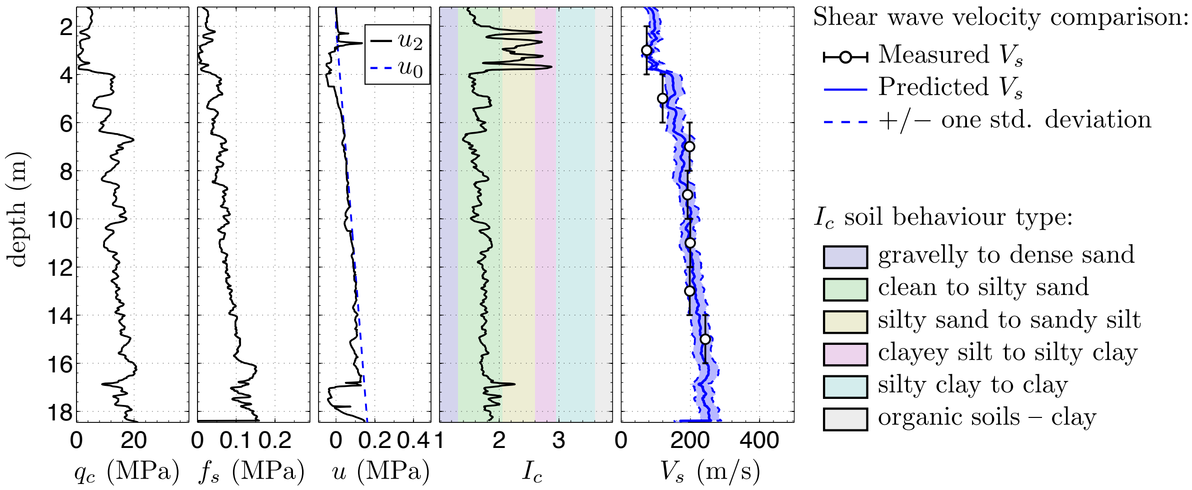

Fig. 10: Comparison of measured V_s values with Christchurch-specific CPT-V_s estimated profile for site ARN28, representative of a sand (lower I_c) soil site (for $z > 4$ m) with soft (lower q_c) response. The groundwater table at this site was reported at 2.0 m depth.

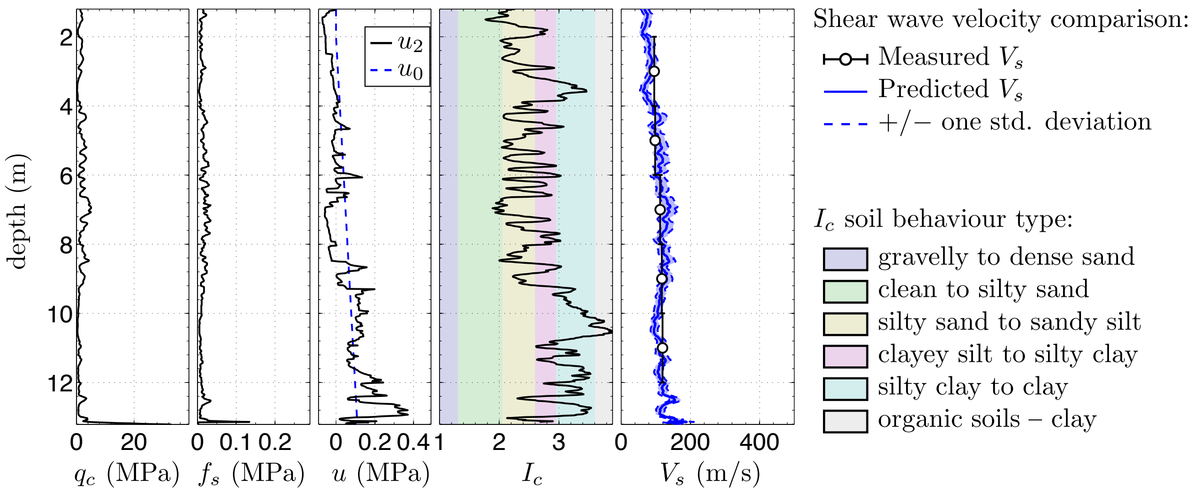

Fig. 11: Comparison of measured V_s values with Christchurch-specific CPT-V_s estimated profile for site BDL08, representative of a siltier (higher I_c) soil site with soft (lower q_c) response. The groundwater table at this site was reported at 2.0 m depth.

4.3 Model forward prediction using representative profiles

To demonstrate and assess the performance of the Christchurch-specific CPT-V_s correlation in a purely forward prediction, as opposed to the comparisons of prediction and observation given by Figs. 8–11, six synthetic CPT profiles are generated and the regression model is applied to predict V_s. These synthetic profiles are based on three soil behaviour type cases: a clean sand ($I_c = 1.55$), a silty sand ($I_c = 1.95$), and a sandy silt ($I_c = 2.45$). Fig. 12 shows the synthetic profiles and corresponding V_s profile predictions for each considered case. Two q_c profiles are assumed for each I_c case, one which represents a softer version of each soil behaviour type, and one which represents a stiff version. The chosen I_c and q_c values are informed by the distributions of these terms within the Christchurch SCPTu database [1] (q_c is used to avoid the assumption of penetration pore pressure distributions necessary to compute q_t).

In order to apply the regression model to these synthetic profiles, f_s values are computed from the I_c equation of [36] for each combination of q_c, I_c, and depth z. The soft-soil q_c values for each I_c case are approximately equal to the smallest values possible without requiring a negative frictional resistance. As shown in Fig. 12, the V_s predictions generally appear to be appropriately sensitive to changes in the predictor variables. Increasing the soil stiffness (via an increase in q_c) for a given soil behaviour type results in higher shear wave velocities with depth. Due to the manner in which the soft soil profiles (lower q_c) were defined, they hold a consistent relationship with I_c over the three considered soil behaviour type

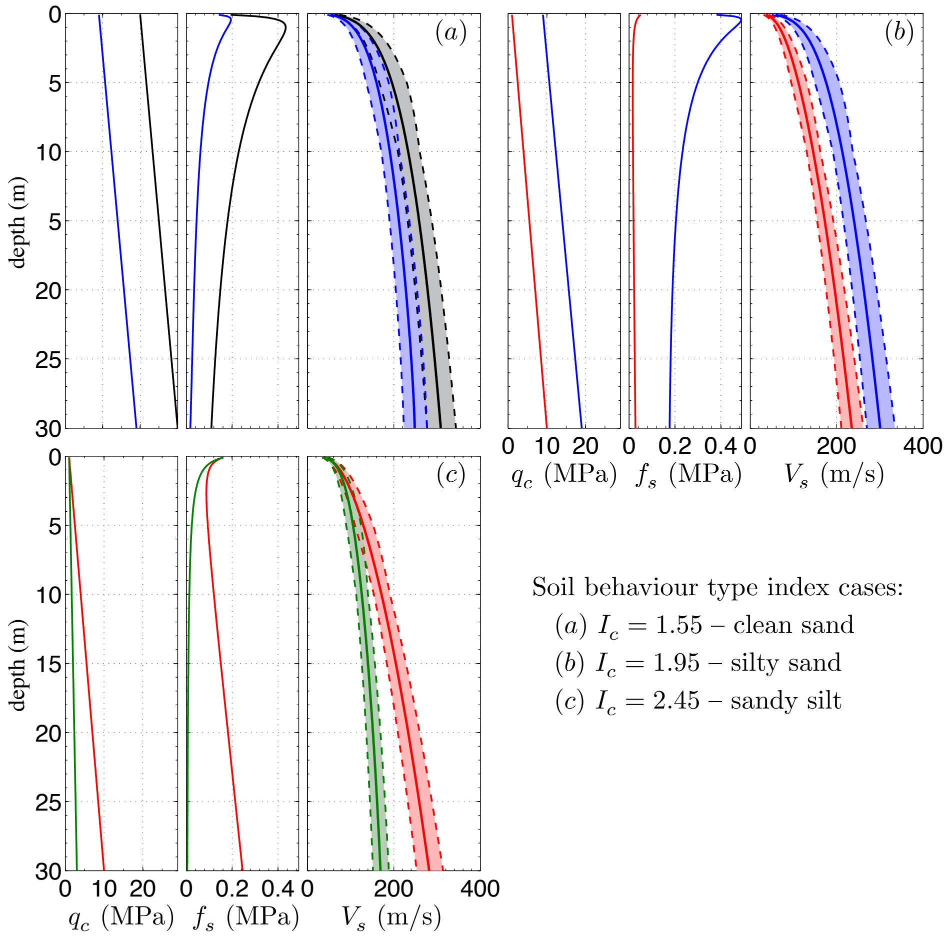

Fig. 12: Estimated V_s for synthetic soft and stiff CPT profiles based on the three indicated I_c cases. Solid lines in V_s plots are median predictions. Dashed lines indicate +/− one standard deviation from median.

indices that doesn't apply to the stiff soil profiles. Comparison of these soft soil profiles across the three soil behaviour type cases shows that increasing I_c generally leads to a decrease in V_s for a given depth. Overall, the performance of the regression model is consistent with expectations in all of the considered synthetic forward prediction cases.

5 Conclusions

Multiple linear regression analysis was used to develop a CPT-V_s relationship for Christchurch, New Zealand soils of the shallow Christchurch and Springston Formations [43] using the SCPTu database discussed in McGann et al. [1]. The selected regression equation depends on the raw cone tip and frictional resistances measured via the CPT, q_c and f_s, respectively, and the depth, z below the ground surface. The regression analysis considers non-constant variance with depth to create a correlation that returns consistent residuals with variations in the predictor variables, as well as with the estimated shear wave velocity and CPT-based soil behaviour type index. The new CPT-V_s correlation provides a viable method to estimate V_s from CPT data that is specific to the non-gravel soils of the Christchurch and Springston Formations within the study area in their current (i.e., post 2010-2011 earthquake sequence) state. Caution is suggested when applying the new CPT-V_s correlation to sites located in the western portions of the Springston Formation where the SCPTu data are sparse (see McGann et al. [1], Fig. 1).

As part of the recovery effort following the 2010-2011 earthquakes in Christchurch, much effort has been made to characterize the near-surface soils of the region for such purposes as residential site classification, liquefaction susceptibility assessment, and other practical and research-related evaluations. As a result, there is a large, spatially dense database of CPT records for Christchurch city and the surrounding suburbs most affected by the earthquakes. In combination with the empirical CPT-V_s prediction equation

developed in this paper, this abundance of CPT data presents a unique opportunity for a region-wide estimate of the near-surface V_s profile of the soils in Christchurch, as well as the opportunity for site-specific V_s estimates anywhere there is a CPT record. Such site-specific and region-wide V_s characterizations should prove useful in future research efforts to evaluate near-surface site effects via site response analysis and to better understand the spatial variability in the shear wave velocity (and small strain shear modulus) of the near-surface soils of the region.

The ongoing earthquake recovery and research efforts in Christchurch have also resulted in a wealth of V_s measurement data obtained independently of the SCPTu database used in this study using various borehole-based [15–17] and surface-wave analysis techniques [8]. Using some of these available data, McGann et al. [47] show that the Christchurch-specific CPT-V_s correlation compares favorably with V_s profiles measured using surface-wave analysis methods at several Christchurch strong motion stations. In addition to the previously mentioned benefits of the new correlation, agreement between measured and estimated V_s values presents the potential for improved site-specific shear wave velocity characterization through the combination of directly measured and CPT-estimated V_s profiles. Because the CPT provides a nearly continuous variation of resistance data with depth, the V_s profile obtained from the Christchurch-specific CPT-V_s correlation is also nearly continuous. This increased resolution can aid in refining the locations of layer boundaries where large velocity contrasts occur for measurement techniques with relatively coarse measurement resolution (e.g. surface-wave analysis), and result in improved inputs for site response analyses where the location of large impedance contrasts is a critical factor in defining the overall response of the system.

6 Acknowledgements

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Recent Evolution and Implementation of Geotechnical Design Practice in New Zealand

ABSTRACT:
Following the 2010-2011 Canterbury Earthquake Sequence a Royal Commission of inquiry provided approximately 200 recommendations to the Ministry of Business, Innovation & Employment (MBIE). Of these, over thirty were specific to geotechnical engineering. The royal inquiry recommended that MBIE work with industry to improve geotechnical engineering practice in New Zealand. In response, MBIE collaborated with the New Zealand Geotechnical Society to develop the Earthquake Geotechnical Engineering Practice Series which is commonly referred to as "The Modules". This paper presents a discussion on the uptake of the Modules and the education program which was developed to promote consistency in everyday engineering practice. The authors also discuss the findings of an online survey and the limitations of the Modules and frustrations voiced by practitioners. The paper closes with the author's thoughts on future guideline topics for geo-professionals and other engineering fields to improve the standard of earthquake geotechnical engineering in New Zealand.

1. INTRODUCTION

1.1 Background

At the 19th International Conference on Soil Mechanics and Geotechnical Engineering that was hosted by the ISSMGE in Seoul, 2017, Price and Stannard [immediate past chair of the New Zealand Geotechnical Society (NZGS) and ex-Chief Engineer of the Ministry of Business, Innovation and Employment (MBIE), respectively] introduced the New Zealand Earthquake Geotechnical Engineering Practice Series (The Modules) to the international community. The Modules were developed following the devastating Canterbury Earthquake Sequence (CES) events in 2010 and 2011. The Canterbury Earthquakes Royal Commission (CERC) of inquiry provided 189 recommendations to MBIE. Of these, over 30 were specific to earthquake geotechnical engineering, with CERC recommending that MBIE work with industry to improve earthquake geotechnical engineering practice in New Zealand. The Modules were developed with that recommendation and goal in mind.

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1.2 Current Guidance in New Zealand

The Modules have been published as a series and released progressively; each on a different aspect of earthquake geotechnical engineering. The Modules are issued as guidance under Section 175 of the Building Act 2004 and herein referred to as Modules. Module 1 provides an overview of the Modules followed by Modules 2 and 3 that provide guidance on geotechnical investigation, identification, assessment, and mitigation of liquefaction hazards. Modules 4, 5 and 6 provide guidance on earthquake resistant foundation design, and the design of ground improvement and retaining walls. Use of the Modules took on more urgency, as the CES events were followed by the Kaikoura earthquake in November 2016 and ex-Tropical Cyclone Gita, which resulted in infrastructure damaging floods and strong winds during February 2018.

The series of Modules was produced as a collaborative partnership between the NZGS and MBIE. Principally the Modules were developed for practicing geotechnical professionals who have a sound background in soil mechanics and earthquake engineering in New Zealand. However, other engineering disciplines are using the Modules daily. The authors set out to explore the implementation of the Modules and gather more information from "users" by way of an online survey.

A full copy of all the Module documents may be downloaded from the MBIE website: **https://www.building.govt.nz/building-code-compliance/geotechnical-education.**

2 ONLINE SURVEY

2.1 Survey development

The authors developed an online survey, using a web-based software system, and invited members of several Technical Societies to respond. The survey was available between 28 August and 7 September 2018 and sent to members of NZGS, the New Zealand Society for Earthquake Engineering (NZSEE) and the Structural Engineering Society New Zealand (SESOC). Membership of the societies total 1245, 800 and 2100 respectively.

2.2 Survey boundaries and questions

An opening question asked if respondents agreed to their feedback to be used for this paper. Answering 'no' led the respondent out of the survey with an automatically generated "thank you" response.

The survey questions were grouped into four sections. Section 1 was aimed at establishing respondents' engineering background, professional experience, engineering discipline and geographic location. Section 2 set out to gather an understanding of the respondents' daily use of the Modules. The questions presented in Section 3 asked respondents if they had attended any of the Module training sessions and if any barriers were encountered that prevented them from attending such training. Section 4 of the online survey asked what other topics of guidance should be included in future Modules.

2.3 Sample sizes

A total of 495 people responded to the online survey over the two-week period. Response rates spiked when two reminder emails were sent out to members. Only 17 respondents did not agree to their information being used as part of this paper. An additional 95 respondents only partially completed the survey.

The authors cannot accurately provide a survey response rate, due to the limitations of how the survey was disseminated to technical society members. As such, the results of this survey are indicative to the respondents only and are not representative of the New Zealand engineering community. A total of 383 respondents filled out the survey to completion and this has provided some interesting insights into the evolution and implementation of earthquake geotechnical engineering design practice in New Zealand.

3. BACKGROUND, EXPERIENCE AND LOCATION

3.1 Engineering disciplines

In response to Question 1 of the survey, just over half identified themselves as geotechnical engineers, more than two fifths as structural engineers, and the remaining respondents as either civil engineers or 'other' disciplines. The respondents who identified themselves as 'other' were; students, engineering geologists, geotechnical drillers, geophysical scientists or coastal engineers.

Respondents were generally well distributed across New Zealand, with approximately two thirds located in the three major cities of Auckland, Wellington and Christchurch. A higher representation from larger cities is not surprising in New Zealand, as a significant number of

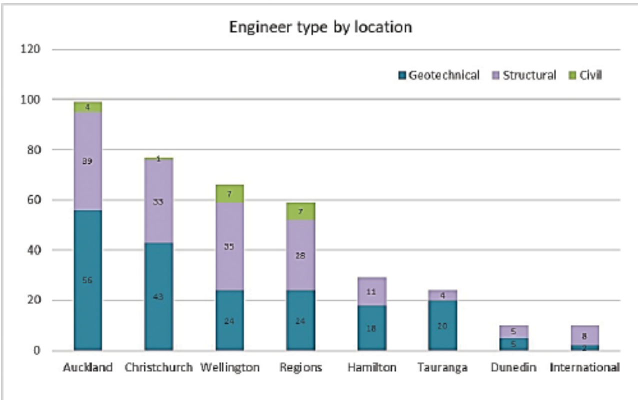
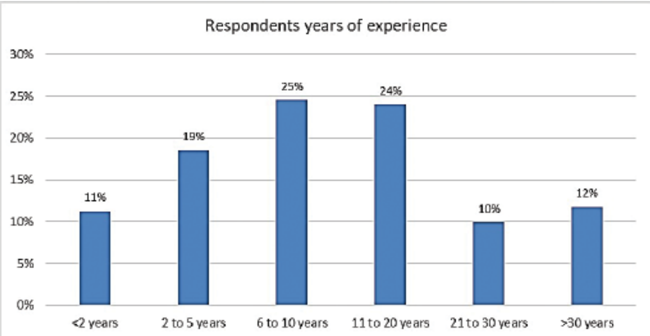


Figure 1: Summary of survey respondents' engineering discipline and geographic location.

engineers work for major consultancies that are typically based in these cities to enable them to respond to large projects. A small percentage of the respondents were based overseas in Australia, Chile, Fiji, India, Peru, UK, Mexico, Philippines and Vanuatu. Figure 1 summarises the number of respondents by engineering discipline and geographic location.

3.2 Work experience

Half of the respondents had between 6 to 20 years' experience (Fig. 2). The majority (over 80%) of the respondents were in the main cities of New Zealand with the exception of engineers with between 21 to 30 years'



experience.

Figure 2: Summary of survey respondents' years of experience.

Approximately half of recent graduates (i.e. those with less than 2 years' experience) use the Modules in their "everyday engineering activities". Similarly, just over half of respondents with over 30 years of experience, use the Modules regularly.

All 24 respondents (across all six experience levels) located in Tauranga, stated that they use the Modules in their everyday engineering activities. This compares to between 62% and 76% of the time for the other five New Zealand cities where respondents were based.

The authors have considered that this question may have been interpreted by some respondents as to whether they use the Modules every day, rather than do they use the Modules in their "everyday engineering activities", meaning their standard or routine work they complete. Nevertheless, the authors are keen to broaden the use to the Modules, particularly for those in the early stages of their career, as they are the future of the industry and it will help to foster a consistent approach across the industry for the geotechnical design of seismically resilient buildings.

4. USE OF MODULES

4.1 Daily use

Question 5 sought to gain an understanding of the respondents use of the Modules in their everyday engineering activities. Over half of the respondents indicated that they use the Modules between 15% and 40% of the time. Clearly, most respondents do not use the Modules that frequently during their day.

Just under 20% of respondents rarely use the Modules while about 15% of the respondents indicated that they use the Modules between 40% and 60% of the time. The remaining 20% of respondents use the Modules 60% or more each day.

This relatively low use of the Modules may be attributed to several factors, such as; their current distribution and acceptance across the industry, limitations on practical applications, and a wide variety of fields within geotechnical earthquake engineering. The authors acknowledge that the relative use of the Modules in the respondents everyday engineering activities is low, and to increase usage and their acceptance in everyday engineering work, more active or targeted discussions around their applicability, training on their use and confirmation of technical content considered "best practice" is warranted.

4.2 Frequency of use

Of the seven Modules available to practitioners, Question 6 sought to understand how often each Module was being used by the respondents in their everyday engineering activities. Module 3 (Liquefaction Hazards) was identified as the most used by respondents who identified themselves as geotechnical engineers, followed by Module 6 (Earthquake Resistant Retaining Wall Design) and Module 4 (Earthquake Resistant Foundation Design). Conversely, respondents who identified themselves as Structural Engineers used Module 6 most often, followed by Module 4.

Frequent use of Module 3 by Geotechnical Engineers was expected, as liquefaction assessments are becoming routine and part of best practice earthquake engineering in New Zealand.

Depending on scope, objectives, structure type and stage of design, both structural and geotechnical engineers are engaged to design foundations and retaining structures. Only 14% of structural engineers who responded use Module 2 (Geotechnical Investigations for Earthquake Engineering). Module 2 provides guidance on appropriate investigations to assess the ground conditions to support the seismic design of new structures, including appropriate minimum scope and methodology for undertaking such investigations.

Structural engineers are often engaged early in a project and typically engage geotechnical engineers once concept or developed designs have been completed. This can often lead to tension between the disciplines as to the appropriate quantum of site investigation. The

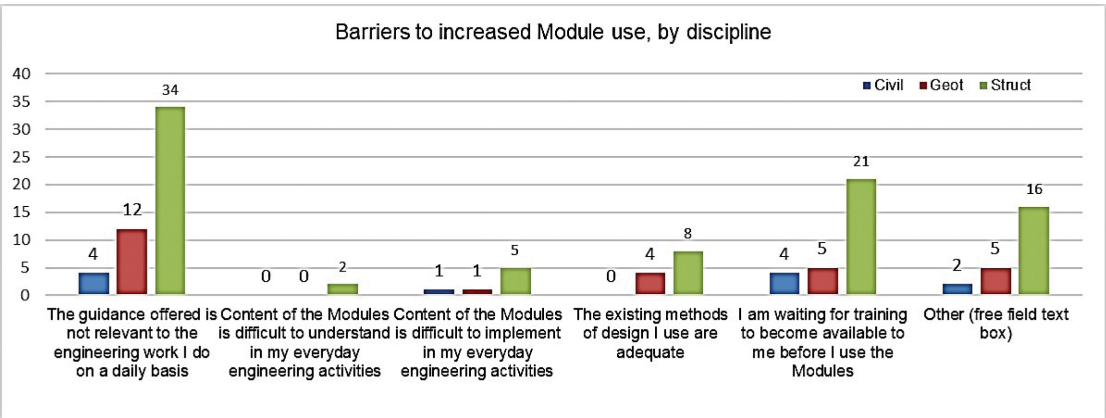


Figure 3: Barriers identified in using the Modules by engineering discipline.

authors note that Module 2, specifically, needs further dissemination amongst engineering professionals who are involved in earthquake engineering to promote consistency in geotechnical investigation applications and expectations.

5. MODULES ADOPTION

5.1 Barriers to use

A third of the respondents stated they do not use the Modules in their everyday engineering activities. Of these respondents, approximately 65% were structural engineering professionals, two fifths were geotechnical engineers and the remaining civil engineers or other professionals. The main barrier identified was that the guidance currently offered in the Modules is not relevant to the work they do on a daily basis. Training availability was identified as the second largest barrier to Module use.

Geotechnical engineers made up 27 out of the 129 respondents who answered 'no' to this question, with 12 indicating that the guidance was not relevant to their work. Figure 3 summarises the reported barriers to Module use in everyday engineering activities.

The last question within Section 2 asked if more detailed worked examples were provided to help the user understand the content, would their usage of the Modules increase? Just under three quarters of the respondents (274 out of 383) answered 'yes'.

5.2 Training

MBIE, NZGS and Engineering New Zealand (ENZ) are collaborating and working closely to deliver ongoing training to the wider New Zealand industry in the content and use of the Modules. This is being completed via a combination of traditional "classroom-based" and online forums. The currently-available online training courses can be accessed through the MBIE website.

Training on the use of the Modules has been developed and made available to all members of ENZ via their website and fortnightly email notification process. Technical societies send out additional reminder emails

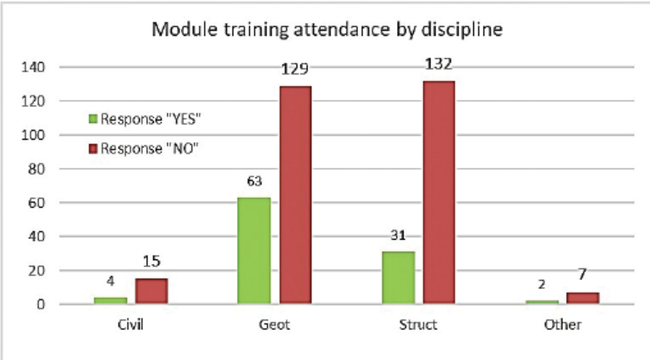


Figure 4: Number of participants who have attended specific training on the use of the Modules.

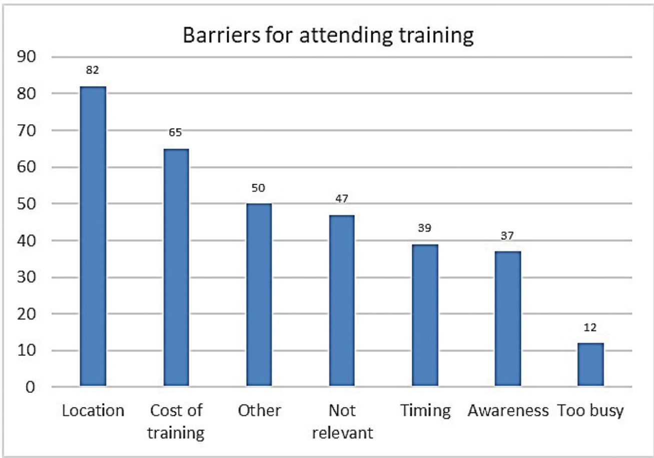


Figure 5: Barriers identified to attending the Module face-to-face training.

to their members, if appropriate, to assist in advising their members of the upcoming training opportunities.

Classroom training on Module 3 was rolled out between 19 and 28 September 2016, and 28 August and 8 September 2017 in Auckland, Christchurch, Wellington and Dunedin. Similar face-to-face training on Module 4 was presented between 7 and 21 March 2017 and 11 to 31 July 2018 in Auckland, Christchurch, Wellington, Nelson, Hamilton and Invercargill.

One quarter of respondents have attended one of the training sessions. Figure 4 below provides a breakdown of the responses to Question 9 by discipline type.

The authors note that training on the use of Module 6, at the time the survey was completed, had not been released. It is assumed, that respondents who indicated that they had attended Module 6 training, are likely to have attended the Retaining Wall Design Practice, which is recommended prerequisite training for engineers attending the Module 6 training course.

The survey results indicate that the Module 4 training courses have had most attendance followed by Module 3, with 59 and 48 respondents attending these sessions, respectively. Conversely, attendance data collected by ENZ indicates that they have had 306 and 138 people attend training sessions on Module 4 and 3, respectively.

The cost of face-to-face training sessions was identified as the second biggest barrier to respondents attending the training, followed by the respondents either being unavailable to attend the training (due to work commitments), or not being aware such training was available. The authors were expecting to see a high return rate of respondents being 'too busy' to attend the training, however, this was not the case (refer to Fig. 5). There is a desire for engineers to attend the Module training sessions, however there are several other barriers.

Of the respondents who indicated that the training was not beneficial to their needs, this was summarised as generally being too high level or too simplified; or, that most geotechnical practitioners should be able to understand and apply the technical content by self-reading and on-the-job learning. Such geotechnical practitioners also identified that a lot of training content was 'routine' to them and not additional learning benefit.

6. FUTURE GEOTECHNICAL MODULES

6.1 Survey responses

The final section of the online survey gathered respondents' thoughts on what other aspects of earthquake geotechnical engineering, or geotechnical engineering in general, should be the focus of future Modules. The survey included two options or a free field text box. The two options offered (a) slope stability of soil and rock slopes, and (b) soft soils.

Of the 383-people surveyed, 76% responded that they saw value in the development of a new slope stability of soil and rock slopes module, closely followed by 64% responding that a soft soil module should also be developed. A wide variety of fields within geotechnical engineering were presented as other options to focus on, including:

- Soil and rock anchor design for seismic strengthening projects
- Foundation design in expansive soils
- Ground Motion Prediction Equations (GMPE)/ Probabilistic Seismic Hazard Assessments (PSHA) and Site Response assessments
- The use of, interpretation, and appropriateness of hand tool investigations, using Scala Penetrometers, Shear Vanes and Hand Augers
- Soil-Structure-Interaction (SSI) and improved collaboration procedures between Structural and Geotechnical Engineers
- Working with, and designing in, volcanic and pumice soils.

The above responses indicate that a broad range of guidance is sought from the engineering community to assist in their everyday engineering activities and to improve the standard of earthquake geotechnical engineering practice in New Zealand.

6.2 Maintenance and updating of modules

The authors believe that ongoing review and update of the Modules is required to ensure that they remain relevant and a reflection of current best practice. MBIE and the NZGS are working together with ENZ to develop a sustainable long-term review program.

The final review programme (which was still being finalised at the time of writing this paper) will aim to ensure that all Modules are reviewed in a logical and timely manner. At this stage, it is envisioned that one to two documents will be reviewed each year, with a review of the entire suite being completed every five to seven years.

6.3 CPD

Continuing Professional Development (CPD) is a key component of the Chartered Professional Engineer (CPEng) system in New Zealand. All holders of the CPEng quality mark have "an ongoing requirement to undertake education, maintain a current knowledge base and improve skills and knowledge". It is expected, that all CPEng holders complete no less than 40 hours of CPD activities per annum. It is generally expected, that all New Zealand CPEng holders who specialise in the field of geotechnical engineering become familiar with the Modules.

6.4 Future modules

The NZGS plans to continue working collaboratively with MBIE and ENZ and the wider industry to develop four or five additional geotechnical design modules. Planning by the NZGS to develop geotechnical design Modules for slope stability, ground anchors and soil nails, and subsoil drainage has commenced. It is also envisaged, that the scope of the additional documents will be expanded, as appropriate, to include non-seismic design situations.

7. DISCUSSION

While the Modules are clearly a useful set of "tools" for practicing engineers in New Zealand; their use has not been universally or consistently adopted. Better adoption of the Modules, may improve with time, particularly as university study and graduate training programmes start to raise awareness and promote the use of the Modules with their students.

For those considering development of guidelines, the authors would recommend interactions with engineering practitioners are made early in the process. Such interactions could include pre-development surveys to understand the needs and requirements of practitioners. A well-planned and appropriately-targeted training programme should also be developed in conjunction with, and released in parallel with, the Modules.

Finally, the authors have observed that the Modules have resulted in some registrational ambiguity, as while they have been disseminated as guidelines, some users consider them to be Standards. At the NZGS Symposium 2017 in Napier, there was lively debate on whether the Modules should be further developed into NZ Standards. The authors believe the Modules should remain as guidelines, with the understanding that they are regularly updated to reflect International and New Zealand best practice. Standards are typically updated infrequently, after much debate and invariably with compromises.

8. ACKNOWLEDGEMENTS

The authors would like to acknowledge the many practitioners who responded to our survey and those that have contributed to the development of the Modules. Ella Priest Forsyth of Beca kindly developed and ran the online SurveyGizmo, thank you. Engineering New Zealand kindly provided the data on the training sessions. The authors would also like to thank the NZGS, SESOC and NZSEE for allowing the survey to be disseminated to their members, and Rachael Scott-Schumacher for her excel and data analytics skills.

9. REFERENCES

Price, C.H. & Stannard, M. 2017. The development of Modules and continuing professional development in earthquake geotechnical engineering design practice in New Zealand. *Proc. of the 19th International Conference on Soil Mechanics and Geotechnical Engineering*, Seoul.

This paper was previously submitted to and accepted by 7th International Conference on Earthquake Geotechnical Engineerng in Rome (June 2019)

NZGS Student Poster Awards

Rolando Orense

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

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THE NEW ZEALAND Geotechnical Society Student Awards are presented to recognise and encourage student participation in the fields of geotechnical engineering and engineering geology, and to introduce our best student talent to potential employers.

For these awards, students of a recognised tertiary institution in New Zealand prepare an A1 size poster that clearly and concisely presents their work on any aspect or topic in the fields of geotechnical engineering or engineering geology. The posters are judged on technical content, layout, and overall poster appeal and ranked by a panel nominated by the management committee of the NZGS.

THE WINNERS THIS YEAR WERE:

1st Place: Yuri Wong (PhD student, University of Auckland)

2nd Place: Charles Mangos (PMEngGeol student, University of Canterbury)

3rd Place: Ethan Flintoft (BEHons student, Waikato University)

The awards will be presented at the next branch meetings.

Opposite: 1st place: Yuri Wong

Development of a Waikato basin model by the H/V spectral ratio method

Author: Ethan Flintoft Supervisor: Seokho Jeong

Motivation

- Dynamic response of soft sedimentary basin intensifies and prolongs shaking during an earthquake [1], as evident from data recorded in Waikato during the recent Taumarunui earthquake shown in Figure 1.
- To accurately characterize the seismic hazard, we need a good understanding of the dynamic characteristics of the sedimentary basins, such as the fundamental periods and the shear wave velocities.
- Waikato region has a significant population, has the fourth largest regional economy (STATS NZ, 2007), and a number of critical infrastructure systems, such as the Waikato expressway, railways, power transmission lines, and gas pipes, but the current understanding on the dynamic characteristics of the Waikato basin is limited.

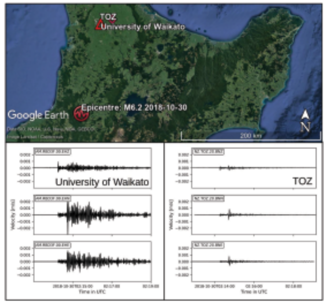


Figure 1: Comparison of ground motions recorded at the Morrinsville (TOZ) and Waikato University stations, during the October 2018 Taumarunui earthquake.

Methodology

1. Install the seismometer on the ground and record the ambient vibration over a short period of time (1-2 hours).
2. Compute horizontal-to-vertical (H/V) spectral ratios as the ratios of horizontal (geometric mean) and vertical component Fourier amplitude spectra, and obtain the fundamental site period (T0) from each sites.
3. Spatially Interpolate the obtained site periods using the Natural Neighbor algorithm to obtain the T0 contour map of the Waikato basin.

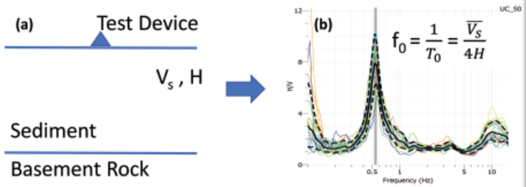


Figure 2: (a) Cross-sectional representation of test device, sediment thickness and basement rock. (b) H/V vs. frequency plot from data collection. The fundamental frequency, f0, is related to the average velocity, Vs, and the depth of the sediment, H [1].

Fundamental vibration periods of the Waikato basin

- 97 tests were conducted around the Hamilton basin. The white triangles in Figure 3 mark those locations. Also shown in the figure are the locations of existing petroleum logs (www.nzpam.govt.nz), at which the depths to the greywacke basement is known.
- We found that the site periods in the basin is as long as 5.4 seconds. The longest periods were found near Te rapa, north-west of Hamilton and near Gordonton to the north-east; we also found that the site periods change rapidly near the areas of longest site periods.
- According to the New Zealand structural design standard for earthquake actions [3], the majority of the Waikato basin is classified as the site class D.

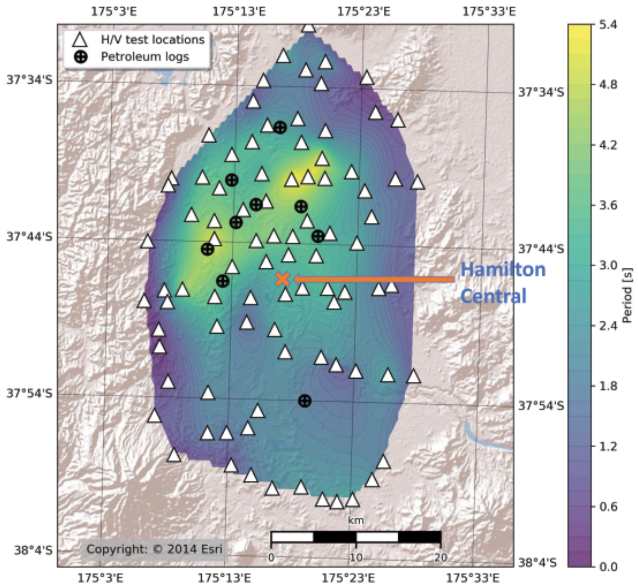


Figure 3: Site period contour map of the Waikato basin, with locations of H/V tests and petroleum borehole logs.

References

[1] Kramer S. (1996), Geotechnical Earthquake Engineering, Prentice Hall
[2] Parolai, S., Bormann, P., & Milkereit, C. (2002). New relationships between Vs, thickness of sediments, and resonance frequency calculated by the H/V ratio of seismic noise for the cologne area (Germany). Bulletin of the Seismological Society of America, 92(6), 2521–2527.
[3] NZS 1170.5:2004, Structural Design Actions Part 5 : Earthquake actions – New Zealand

Site frequency-depth correlation

- As shown in the equation in Figure 2b, a power-law relationship exists between the site frequency and the basin depth. Using the H/V test data and the basin depth obtained by the previous petroleum logs, we developed an empirical model to predict the basin depth, H, as shown in Figure 4.

$$H = 101.6\, f_0^{-1.565}$$

- We compared our model with two overseas studies [2]. We found that both of the overseas models are somewhat consistent with our data, but Parolai's model is more similar to the model we derived.
- All of the observed basin depths (from petroleum logs) used in our regression analysis are deeper than 250m, meaning that the current model is not well constrained where the basin is shallower than 250m.

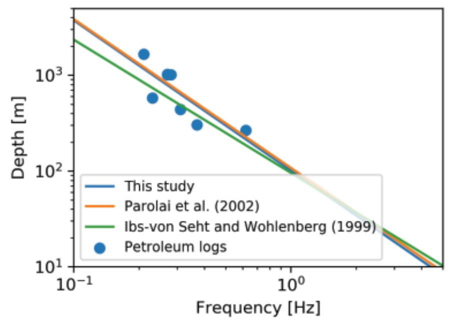


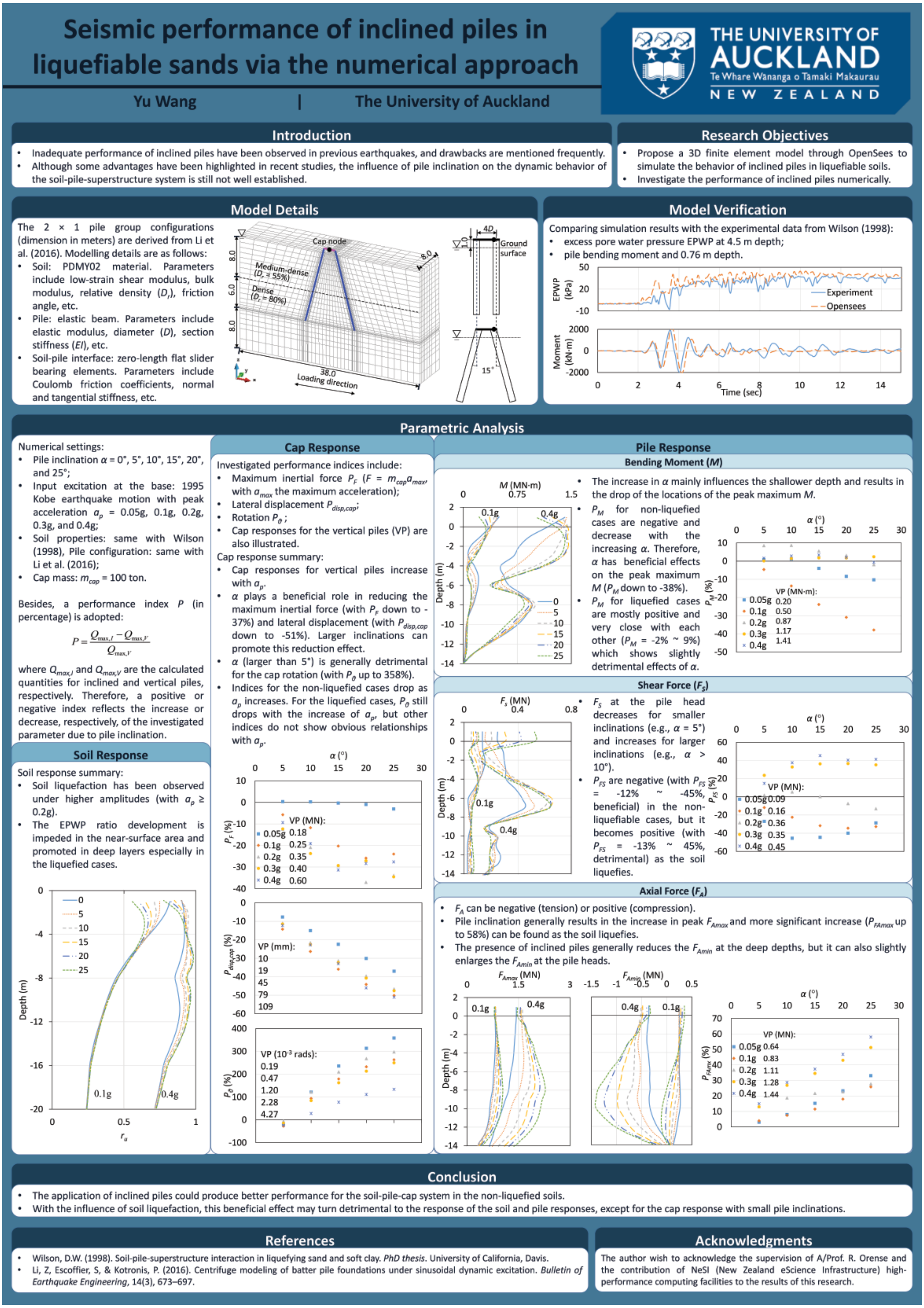
Figure 4: Frequency-depth correlation for the Waikato basin. Also shown are existing (overseas) correlation models by Parolai et al. (2002) and Ibs-von Seht and Wohlenberg (1999).

Key findings

- During the 2018 Taumarunui earthquake, two separate stations closely located in Hamilton recorded significant differences in ground shaking. This is likely due to the amplification and reverberation caused by the response of the sedimentary basin.
- Near Te Rapa and Gordonton the site periods were over 5 seconds. It is expected that those are the points where the basement rock is deepest; we also found that the site periods change rapidly near the areas of longest site periods.
- We developed an empirical model, which estimates the bedrock depth using the obtained fundamental site frequency.

Acknowledgements

This study was made possible thanks to The University of Waikato summer research scholarship program and QuakeCoRE. I would also like to acknowledge and thank Seokho Jeong from the University of Waikato for his patience, guidance and advice throughout the research period.



Seismic performance of inclined piles in liquefiable sands via the numerical approach

Yu Wang | The University of Auckland

THE UNIVERSITY OF AUCKLAND Te Whare Wānanga o Tāmaki Makaurau NEW ZEALAND

Introduction

- Inadequate performance of inclined piles have been observed in previous earthquakes, and drawbacks are mentioned frequently.
- Although some advantages have been highlighted in recent studies, the influence of pile inclination on the dynamic behavior of the soil-pile-superstructure system is still not well established.

Research Objectives

- Propose a 3D finite element model through OpenSees to simulate the behavior of inclined piles in liquefiable soils.
- Investigate the performance of inclined piles numerically.

Model Details

The 2 × 1 pile group configurations (dimension in meters) are derived from Li et al. (2016). Modelling details are as follows:

- Soil: PDMY02 material. Parameters include low-strain shear modulus, bulk modulus, relative density (D_r), friction angle, etc.
- Pile: elastic beam. Parameters include elastic modulus, diameter (D), section stiffness (EI), etc.
- Soil-pile interface: zero-length flat slider bearing elements. Parameters include Coulomb friction coefficients, normal and tangential stiffness, etc.

Model Verification

Comparing simulation results with the experimental data from Wilson (1998):

- excess pore water pressure EPWP at 4.5 m depth;
- pile bending moment and 0.76 m depth.

Parametric Analysis

Numerical settings:

- Pile inclination α = 0°, 5°, 10°, 15°, 20°, and 25°;
- Input excitation at the base: 1995 Kobe earthquake motion with peak acceleration a_p = 0.05g, 0.1g, 0.2g, 0.3g, and 0.4g;
- Soil properties: same with Wilson (1998), Pile configuration: same with Li et al. (2016);
- Cap mass: m_{cap} = 100 ton.

Besides, a performance index P (in percentage) is adopted:

$$P = \frac{Q_{\max,I} - Q_{\max,V}}{Q_{\max,V}}$$

where $Q_{max,I}$ and $Q_{max,V}$ are the calculated quantities for inclined and vertical piles, respectively. Therefore, a positive or negative index reflects the increase or decrease, respectively, of the investigated parameter due to pile inclination.

Soil Response

Soil response summary:

- Soil liquefaction has been observed under higher amplitudes (with $a_p \geq$ 0.2g).
- The EPWP ratio development is impeded in the near-surface area and promoted in deep layers especially in the liquefied cases.

Cap Response

Investigated performance indices include:

- Maximum inertial force P_F ($F = m_{cap}a_{max}$, with a_{max} the maximum acceleration);
- Lateral displacement $P_{disp,cap}$;
- Rotation P_θ;
- Cap responses for the vertical piles (VP) are also illustrated.

Cap response summary:

- Cap responses for vertical piles increase with a_p.
- α plays a beneficial role in reducing the maximum inertial force (with P_F down to -37%) and lateral displacement (with $P_{disp,cap}$ down to -51%). Larger inclinations can promote this reduction effect.
- α (larger than 5°) is generally detrimental for the cap rotation (with P_θ up to 358%).
- Indices for the non-liquefied cases drop as a_p increases. For the liquefied cases, P_θ still drops with the increase of a_p, but other indices do not show obvious relationships with a_p.

Pile Response

Bending Moment (M)

- The increase in α mainly influences the shallower depth and results in the drop of the locations of the peak maximum M.
- P_M for non-liquefied cases are negative and decrease with the increasing α. Therefore, α has beneficial effects on the peak maximum M (P_M down to -38%).
- P_M for liquefied cases are mostly positive and very close with each other (P_M = -2% ~ 9%) which shows slightly detrimental effects of α.

Shear Force (F_S)

- F_S at the pile head decreases for smaller inclinations (e.g., α = 5°) and increases for larger inclinations (e.g., α > 10°).
- P_{FS} are negative (with P_{FS} = -12% ~ -45%, beneficial) in the non-liquefiable cases, but it becomes positive (with P_{FS} = -13% ~ 45%, detrimental) as the soil liquefies.

Axial Force (F_A)

- F_A can be negative (tension) or positive (compression).
- Pile inclination generally results in the increase in peak F_{Amax} and more significant increase (P_{FAmax} up to 58%) can be found as the soil liquefies.
- The presence of inclined piles generally reduces the F_{Amin} at the deep depths, but it can also slightly enlarges the F_{Amin} at the pile heads.

Conclusion

- The application of inclined piles could produce better performance for the soil-pile-cap system in the non-liquefied soils.
- With the influence of soil liquefaction, this beneficial effect may turn detrimental to the response of the soil and pile responses, except for the cap response with small pile inclinations.

References

- Wilson, D.W. (1998). Soil-pile-superstructure interaction in liquefying sand and soft clay. *PhD thesis*. University of California, Davis.
- Li, Z, Escoffier, S, & Kotronis, P. (2016). Centrifuge modeling of batter pile foundations under sinusoidal dynamic excitation. *Bulletin of Earthquake Engineering*, 14(3), 673–697.

Acknowledgments

The author wish to acknowledge the supervision of A/Prof. R. Orense and the contribution of NeSI (New Zealand eScience Infrastructure) high-performance computing facilities to the results of this research.

Above: 2nd place: Charles Mangos

aurecon

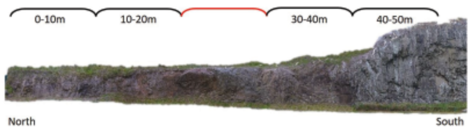
ROCK MASS CHARACTERISATION USING RPAS 3D PHOTOGRAMMETRY: A COMPARISON WITH TRADITONAL AND REMOTE SENSING MAPPING METHODS

UC UNIVERSITY OF CANTERBURY Te Whare Wānanga o Waitaha CHRISTCHURCH NEW ZEALAND

PROJECT OBJECTIVES

- Evaluate the feasibility of using 3D point cloud data derived from a Remotely Piloted Aircraft System to measure discontinuity orientations in a rock face.
- Compare these measurements with Terrestrial Laser Scan and traditional scanline measured methods.

AREA OF STUDY



Halswell Quarry Park was chosen for this study due to the locality to Christchurch. The key requirement was the rock face had to be safely accessed using all three data capture methods.

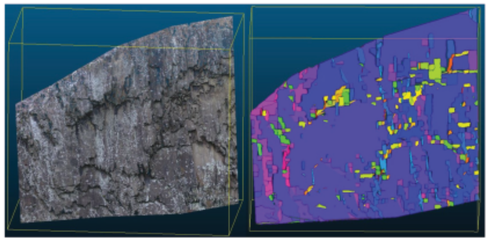
METHODOLOGY

Three data capture methods:

- **Traditional method using a geological compass -** This provided the basis for comparison with the RPAS derived SfM and TLS measurements.
- **Terrestrial Laser Scanner (TLS) -** Four scans were undertaken from four different locations using a Leica ScanStation C10 terrestrial laser scanner.
- **Remotely Piloted Aircraft System (RPAS) -** 218 images were captured in a 7.5-minute flight using a Phantom 4 multi-rotor RPAS manufactured by DJI.

PROCESSING

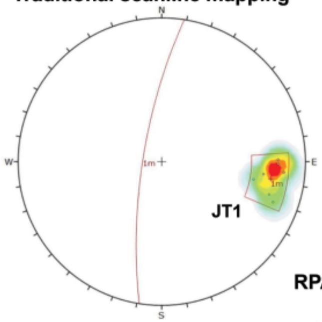
- Pix4D (SfM)
- RocScience Dips – Cluster Analysis
- CloudCompare with FACETS



RPAS derived SfM point cloud before (left) and after (right) FACETS tool.

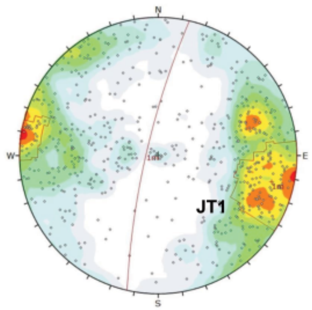
RESULTS

Traditional scanline mapping

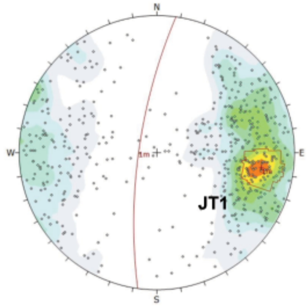


Stereonets of the 40-50m segment showing joint sets identified as Joint Set 1 (JT1) from Cluster Analysis in Dips (Rocscience, 2018).

TLS



RPAS derived SfM



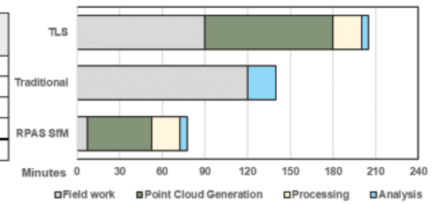
Accuracy comparison

Segment	Traditional (Dip Direction/Dip)	TLS (Dip Direction/Dip)	RPAS derived SfM (Dip Direction/Dip)
0m-10m	273/77	273/73	271/73
10m-20m	302/76	299/88	310/79
30m-40m	284/69	278/70	286/65
40m-50m	279/75	283/80	278/75

Number of measurements

Segment	Traditional	TLS	RPAS SfM
0m-10m	7	119	33
10m-20m	11	172	115
30m-40m	26	475	93
40m-50m	6	615	537
Average	13	345	195

Time comparison



DISCUSSION

The RPAS derived SfM method resulted in:

- Accuracy within 5° of scanline mapping orientations
- Reduced uncertainty using statistical analysis
- Reduction in field work data capture time
- Removal of bias and human error
- Increased safety for field staff

CONCLUSION

In this study, it has been determined that discontinuities (orientation) measured by RPAS derived SfM point clouds are comparable to both TLS and traditional scanline mapping methods.

Data capture with RPAS derived SfM and TLS increased the number of discontinuity measurements and objectively in a much shorter amount of time compared to traditional scanline mapping methods.

RPAS SfM does not replace an engineering geologist in the field but could be extremely helpful tools for identifying and measuring planar features otherwise unobtainable without additional time, cost, or safety risk.

FURTHER RESEARCH

- More dynamic rock faces
- Higher resolution camera on RPAS SfM to capture joint roughness coefficient, aperture width and infilling.
- Automated RPAS flight software

ACKNOWLEDGEMENTS

I would like to thank my Ground Engineering and Digital colleagues at Aurecon NZ Ltd who helped fund this research and provide guidance. I would also like to thank Dr Clark Fenton and Dr Marlene Villeneuve from University of Canterbury for their guidance.

By Charles C. Mangos
Charles.mangos@aurecongroup.com

Above: 3rd place: Ethan Flintoft

Gillies Seve

IT IS WITH much sadness we advise that Dr Gilles Seve passed away after a short illness in March of this year.

Prior to coming to NZ in 2011 Gilles had over 20 years' experience in a wide range of geotechnical engineering projects in South America, Canada, Africa, New Caledonia and many parts of Europe, including his homeland of France.

His international experience encompassed a variety of interesting design projects including large diameter storage tanks, seismic resilience and designing foundations for poor ground. He had carried out geotechnical investigation and field laboratory for preliminary and detailed design of motorways earthworks and his expertise included modelling slope stability and rock fall risk management in seismic areas. Gilles also taught soil mechanics and foundation engineering for over 10 years in the French universities of Paris and Nice, and in the Engineering School in Lyon, France.

Once Gilles arrived in NZ he worked with SCIRT on the Canterbury infrastructure rebuild and then with Christchurch City Council in their consenting team.

Following this, Gilles relocated to Wellington with his wife, and worked for MBIE. This work period included compiling and editing a number of the Earthquake Geotechnical Engineering Guideline Modules in collaboration with the New Zealand Geotechnical Society. He then spent some time working in the consulting industry in New Zealand. Gilles was always ready to teach anyone geotechnical principles and had a knack of reducing complexity to the simple in a sketch.

Gilles enjoyed travel and sailing in particular. He was also an avid gardener and was growing tomatoes through the roof vent on his greenhouse!

Gilles brought a Latin flair and passion to all his work and had a devlish sense of humour. He never let a chance go by to note that he was born and bred in Nice, France and that this was the home of French Rugby. Gilles is survived by his wife Violaine and one daughter, also a practicing engineer.

Prepared by:
John Scott and William Whewell

John A. Hudson, 1940–2019

JOHN A. HUDSON, BSc, PhD, DSc, FREng, Past President of ISRM, passed away on Wednesday 14 February 2019, in London, from complications resulting from a severe stroke.

John Hudson obtained his BSc degree in Mining Engineering from the Heriot-Watt University in Edinburgh, Scotland, in 1965 and his PhD in rock mechanics as applied to both civil and mining engineering from the University of Minnesota, USA, in 1970. After a further two-year period of post-doctoral study at the University of Minnesota, he worked in the Tunnels Division at the Transport and Road Research Laboratory, UK, from 1972-1977, then from 1977-1979 at the Department of Strategic Research Operations in the Department of the Environment, UK. From 1979-1980 he was a Visiting Professor at the University of Wisconsin, USA, and from 1980-1983 at the Building Research Station, UK. During this latter period, he was awarded the DSc degree by the Heriot-Watt University.

From 1983 onwards, John was affiliated to Imperial College of Science, Technology and Medicine, London, as Reader, Professor, and then Emeritus Professor in the Department of Earth Science and Engineering. During this time he supervised 17 PhD students and 50 MSc students. He was elected as a Fellow of the Royal Academy of Engineering (FREng) in 1998 and became a Fellow of the American Rock Mechanics Association in 2009. Professor Hudson acted as an independent consultant on more than 150 projects around the world and gave Short Courses on the principles of rock mechanics in many countries.

His work for the ISRM included being Chairman of the Technical Committee for the ISRM Cambridge Rock Mechanics Symposium, UK, 1984, being the ISRM UK National Group representative, 1987-2011 (as a Member of the British Geotechnical Association), being the ISRM Vice-President at Large (1995-1999), President of the ISRM Commission on Testing Methods (1987-2007), Chairman of the first ISRM EUROCK Symposium (held in Chester, UK, 1992), and Co-Chairman of both of the ISRM SINOROCK Symposia, held in Yichang, China, in 2004, and in Hong Kong, China in 2009.

Not only was John was one of the most active members ever of ISRM, he was one of the most influential teachers of rock mechanics and author and editor of several of the most relevant books on the discipline. He was the Editor of the International Journal of Rock Mechanics & Mining Sciences for 23 years from 1983 to 2006. He authored/co-authored six books and more than 150 technical papers, plus a further three edited books, including "Comprehensive Rock Engineering" (5 vols, 4407 pages) which is the only international benchmark overview of all aspects of rock engineering.

He received the Mueller Award from the International Society for Rock Mechanics in 2015 in recognition of "an outstanding career that combines theoretical and applied rock engineering with a profound understanding of the basic sciences of geology and mechanics".

His most recent major work was the book Structural Geology and Rock Engineering co-authored with John W Cosgrove and published by Imperial College Press, 2016.

Editor's Note: *This Obituary was cobbled together from a number of unacknowledged sources. We hope they will understand.*

Obituary - Paul Baunton

ON SUNDAY 25 November 2018, Paul Baunton, Manager of Emergency Management at Tauranga City Council, sadly passed away after a short illness.

Paul was the driver of ground-breaking tsunami resilience work which changed Tauranga's approach to tsunami risk and resulted in millions of dollars' worth of new evacuation infrastructure. Paul spent over 20 years at Tauranga City Council, working tirelessly to deliver the best for the Tauranga community. His commitment to evidence-based research, particularly in the area of tsunami evacuation planning, has left a long-lasting legacy.

Paul served on the Management Committee of the NZ Coastal Society from 2001 to 2003 and was instrumental in setting up the TCC Geo-professional (Category 1 / 2) accreditation system.

In 2017 Tauranga City Council and Tonkin + Taylor Ltd won the Director's Award for Innovation for the council's tsunami risk mitigation programme. Thanks to Paul, the coastal community now has access to evacuation routes across earthquake-strengthened pedestrian bridges – a New Zealand first. Paul was instrumental in shifting the Tauranga tsunami conversation from sirens to evacuation. He was relentless in his commitment to ensure that the city developed a practical, effective evacuation network that people can have confidence in.

He will be greatly missed by his colleagues and by the emergency management community. He leaves behind a legacy of innovative work that will continue to serve our community for many decades to come.

Matthew Harrex
(with NZGS edits)

NEW ZEALAND GEOTECHNICAL SOCIETY INC
www.nzgs.org

NZGS 2019 STUDENT PRESENTATION AWARDS POSTER COMPETITION

INVITATION TO PARTICIPATE

The New Zealand Geotechnical Society wishes to recognise and encourage student participation in the fields of rock mechanics, soil mechanics, geotechnical engineering and engineering geology.

The 2019 Student Presentation Awards will be a Poster Competition and is open to all students.

Posters will be displayed and awarded at a local branch meeting in April / May 2020.

REGISTRATION CRITERIA

- Applicants must be enrolled as a full-time / part time student at an appropriate University / Institution in New Zealand
- The topic of the poster should be relevant to geotechnical engineering or engineering geology
- There should be one registration form per poster and one co-author is allowed per submission
- The abstract must be no longer than 300 words
- Submission of the abstract should be made on the registration form or an attached Microsoft Word or pdf document
- One figure/image may be included with the abstract but it must have a caption and be referred to in the abstract

Registration forms can be downloaded from the NZGS website at **http://www.nzgs.org/awards/nzgs-student-awards/** and are to be submitted to secretary@nzgs.org by **Friday 15th November 2019**

Completed posters can be provided either at the same time or at any time up to the closing deadline on 31 January 2020.

POSTER CRITERIA

- Standard A1 size.
- Should be submitted in Microsoft Powerpoint or pdf format.

JUDGING

- Judging will be conducted for all entries across the country
- A panel of three judges will be formed to make the final decision
- Posters will be displayed at a local branch meeting
- The awards will be presented at a NZGS local branch meeting

PRIZE MONEY

- $1000 first
- $500 second
- $300 third
- The top three posters will be displayed in the June 2020 issue of the Geomechanics News.

Judging will be based on the following criteria:

- Quality and clarity of the abstract
- Academic content – appropriate introduction, sound methodology, clear results and conclusion
- Poster layout – appropriate use of figures, clear and coherent text, structure and creativity
- Overall poster appeal – concepts are easy to understand, poster engages the viewer and the quality of presentation

For further information or to join the Society (membership is free for students) please visit our website www.nzgs.org or contact the Society Management Secretary at secretary@nzgs.org

IAEG Votes Louise Vick onto Executive Committee

Above: Louise Vick

Above: YEG committee meeting in the garden of the Louvre. From left: Louise Vick, Julia Loffler, Stratis Karantanellis, Alejandro Celli, Pedro Martins (Photo: Andrey Kazeev)

AT THE ANNUAL mid-year board meeting in Paris (7-8 April 2019), the IAEG Executive Committee voted the chair of the new YEG (Young Engineering Geologists) Committee to the Executive Committee with voting rights. Louise Vick, a Canterbury engineering geology graduate, currently holds that position.

Louise, who is currently a Post-doctoral Fellow at The Arctic University of Norway in Tromso, provided the following additional information.

The goal for the committee is to promote the interests of the young members of IAEG within the organisation, and to help young members integrate into the organisation. IAEG recognises that its young members are the future and wants to do everything it can to support their growth.

I was introduced to IAEG during the Auckland 2010 IAEG congress. I really enjoyed the events geared specifically towards young members and wanted to be more involved. Since 2014 I have been working with Pedro Martins (Beca, Tauranga) to develop the role of the young members in a more official capacity. Being elected as chair in 2016 was the first step. Following that, we (Pedro and I) appointed a committee, ratified by the IAEG Board. We had a great amount of support from the board at the time, especially from Scott Burns (President) and Mark Eggers (Vice president for Australasia). Luckily, the new/current president and board are also passionate about supporting the initiatives of the YEG, and in April the committee had it's first official meeting in Paris. The committee comprises seven active people: *Julia Loffler* – Argentina, *Stratis Karantanellis* – Greece, *Alejandro Celli* – Argentina, *Pedro Martins* – New Zealand (originally Brazil), *Bimbola Aluko* – Nigeria, *Andrey Kazeev* – Russia, and myself – Norway (originally NZ). We are proud of the fact that we are gender-balanced, international and a mix of industry personnel and academics. We feel that we can adequately represent the YEGs around the world. The Paris meeting was a success, and Pedro and I stayed on to attend the concurrent IAEG Board meeting where we presented our budget and plan. It was determined that the chair of the YEG committee will now sit on the board. This is a great achievement for us, as we now have a lot more influence within the IAEG.

In September at the Jeju Asia Regional Conference the IAEG council will decide whether the YEG chair will have voting power. We will present our 5 year working plan there.

We are currently working on our webpage (**www.iaeg.info/category/yegs/**) to make it easier for YEGs to see how we are networking, and how they can get involved. For now if people are interested they are welcome to contact me or Pedro.

See page 12 for more on Lousie's adventures in Norway.

NZGS Awards Calendar – May 2019

NZGS SCHOLARSHIPS (BIENNIAL)

2019 scholarship awardees:

- *Doug Mason* (PhD student, University of Canterbury) – $10,000
- *Heba Elsaidy* (PhD student, University of Auckland) – $5000

Important dates for this biennial award are:

Sept 2020	Call for applications
End Oct 2020	Closing date for applications
End Nov 2020	Closing date for full submission
Feb 2021	Successful applicants announced (if any)

NZGS STUDENT AWARDS POSTER COMPETITION (ANNUAL)

The 2018-2019 winners are:

- *Yuri Wong* (PhD student at University of Auckland with Rolando Orense)
- *Charles Mangos* (PMEngGeol student at University of Canterbury with Marlene Villanueve)
- *Ethan Flintoft* (BEHons student at Waikato University with Seokho Jeong)

See 2018-19 winning posters in June issue, see page 116.

Important dates are:

June 2019	Call for applications
Mid Nov 2019	Closing date for applications
End Jan 2020	Closing date for receipt of poster
Feb 2020	Judging and announcement of winners at local NZGS evening event

See call for applications in June issue, see page 123.

NZGS-7ICEGE SCHOLARSHIPS

The recipients of the $5,000 scholarships are:

- *Maxim Millen* (University of Canterbury)
- *Baqer Asadi* (University of Auckland)
- *Ribu Dhakal* (University of Canterbury)
- *Gislaine Pardo* (University of Auckland)

A fifth "Highly Commended" scholarship award ($3,000) was given to *Aimee Rhodes* (WSP Opus).

Rolando Orense

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

r.orense@auckland.ac.nz

International Association for Engineering Geology and the Environment

THE EXECUTIVE COMMITTEE elected at the 2018 Congress in San Francisco came into effect as of 1 January. On behalf of NZGS and New Zealand IAEG members I extend a big thank you to Mark Eggers for his role as VP Australasia over the past four years.

The previous committee under the direction of Scott Burns embarked on programme of work to look at how the IAEG could achieve its purpose in today's changing world recognising that organisations such as IAEG need to remain current to their membership. Mark made a very strong contribution to the rethinking and redefining the purpose and mission of IAEG and setting up a draft strategic plan which has been reported in previous IAEG newsletters.

For this term of the executive Anthony Bowden will be the AGS liaison and we look forward to working together.

The new executive held its first meeting on 5 and 6 April in Paris which was attended by myself and Pedro Martins representing YGP (the YGP group met on 4 April). The meeting addressed a number of issues to support the advancement of the strategic agenda developed by the previous executive as summarised below.

- The executive supported the YGP nominated leader to be full voting member of the IAEG executive – this is to be put to the annual council meeting in September.
- Additional budget was allocated to allow the GYP network to be better established.
- It was agreed to set up a number of working groups (or advisory committees) to assist the executive advance
  - A review of the bylaw and statues (to ensure they support the purpose and mission of IAEG and reflect today's communities)
  - A review of the various technical committees of the IAEG
  - Establishment of promotional activities to support wider IAEG operations
- These working groups are in the process of being set up. Mark Eggers and Ann Williams have been identified as people who may support these working groups.
- The executive also reported on changes to the European privacy laws (with which IAEG must comply as it is registered in Paris with substantive European membership). Changes to the way IAEG manages membership and personal details are required to comply with this new legislation. A proposal to develop a new system to manage member's personal data was accepted and agreed to. Once this is ready for introduction we will be advised. It will not affect the activities of our members (it does not require us to do anything). It will be more secure than at present with access to data limited to each member and to a secure operator for administration purposes only. Any other access will only be possible with your permission.
- The Executive also discussed the election of new groups to the IAEG - these were endorsed and will be voted on at the Council meeting in September.

Doug Johnson

Doug has a Master's degree in Engineering Geology from the University of Canterbury NZ (1984). He has worked on many mining, quarrying and civil engineering projects across a range of complex geological terrains, geographies and on both green and brown field site developments. Doug is currently Managing Director of Tonkin + Taylor and is passionate about people, the client experience, and technical solutions providing long term benefits to the community and the environment.

We understand there have been some teething issues with access to the electronic bulletin of IAEG for some people and this has been followed up. If you are still having issues please contact myself or Teresa so that we can alert the IAEG Secretariat.

For information on upcoming IAEG events and general information please refer to the Web site (INCERT REF) and to IAEG Connecter which is emailed to members on regular basis.

Doug Johnson
NZ IAEG Representative

International Society for Rock Mechanics and Rock Engineering

The period since December 2018 has been relatively quiet ahead of preparations for the next ISRM Board and Council meetings in Foz do Iguassu, Brazil on 12-13 and 15 September 2019 respectively, immediately before the four yearly ISRM Congress, a major event on the ISRM calendar.

14TH ISRM INTERNATIONAL CONGRESS ON ROCK MECHANICS AND ROCK ENGINEERING

The 14th ISRM Congress will take place 13-18 September 2019. Brazil, Argentina and Paraguay will be the host countries for the event, to be held in Foz do Iguassu, a city at the common border between these countries. Foz do Iguassu is famous for the Iguassu Falls on the Brazil-Argentina border.

The call for abstracts resulted in submission of over 700 abstracts (thirty from Australia, four from New Zealand). Paper allocation depends on a formula depending on factors such as past Congress papers, membership (Australia 3.5%, NZ 1% of papers). Draft papers (two from New Zealand) have been reviewed with submission to publishers in late June.

Further information on the Congress is available at www.isrm2019.com. Aside from technical and poster sessions, there are industry-aligned technical sessions, and invited lectures, one of which will be on behalf of another International Society.

Keynote speakers, who are based on geographic area representation, have been confirmed (Dr John Read on "The geotechnical engineer in metalliferous open pit mines" is the selection made by the Congress on behalf of Australasia). The 8th Muller award, a recognition of distinguished contributions to the profession of rock mechanics and rock engineering, will be presented by Prof Peter Kaiser.

There is also a strong young professionals programme, including more social activities such as the Rockbowl quiz and student night. The 2019 Rocha medal winner will present his thesis, with other sessions for an Early Career Forum and young researchers "brightspark" presentations.

A post-congress fieldtrip will visit the nearby 12,600 MW hydroelectric dam and powerstation on the border between Brazil and Paraguay.

MEMBERSHIP

ISRM individual membership worldwide has risen (from 7,800) to 8,200, collected from with 61 National Groups (countries). Europe and Asia have the greatest individual membership (>25%), with other regions including Africa and Australasia having ~5% each – currently 325 in Australia and 192 (up 20 in last 6 months) from New Zealand). There are 155 Corporate memberships, with four from Australia and none from New Zealand.

ROCHA MEDAL (2020):

Twelve theses for the 2020 award, recognising the most meritorious PhD thesis in rock mechanics, were received by 31 December 2018 (0 from Australia, 0 from New Zealand). The Rocha Award Committee (basically the Board) will evaluate the theses with the winner to be announced at the ISRM Council meeting in Foz do Iguassu.

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.

ISRM ON-LINE LECTURES

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

- 24th by Prof. Claudio Olalla (Spain) on13 December 2018 with the title "Computing Foundations with Hoek and Brown Failure".
- 25th by Prof Derek Martin (Canada) on 21 March with the title "Stress-induced fracturing (spalling) around underground excavations: Laboratory and In-situ Observations".

The lectures are available on the ISRM website (ISRM online lectures - e.g. https://www.isrm.net/gca/index.php?id=1340 for Prof Olalla).

COMMUNICATION

The ISRM website (www.isrm.net) has information on the society's intent, structure and activities, including conferences, commissions, awards, products and publications. For those AGS members affiliated to ISRM as individual members there is a member area with access to further products. There is also Linked in, Twitter or RSS access.

Regular means of communication (under ISRM information on the website) are:

- ISRM Newsletter, which has been published quarterly since March 2008.
  Last issue No 45 in March 2019
- ISRM News Journal, now under the editorship of Dr José Muralha (Portugal)
  Issue No 21 summarising 2018 is online

The ISRM Digital Library, which was launched in October 2010, is intended to make rock mechanics material available to the rock mechanics community, in particular papers published from ISRM Congresses and sponsored Symposia. It is part of OnePetro (https://www.onepetro.org), a large online library managed by the Society of Petroleum Engineers, and includes proceedings from 54 ISRM sponsored conferences. ISRM individual members are allowed to download, at no cost, up to 100 papers per year from the ISRM conferences. To use this facility ISRM members must register each year, with further details given on the ISRM website (https://www.isrm.net/gca/?id=992).

COMMISSIONS

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

Commissions run on a voluntary basis and several are very active, with associated publications (e.g. blue and orange books for testing methods), some less so, with one having stopped in the last months (Subsea tunnels) and another currently suspending activities. The summary of 2018 activities prepared by the Technical Oversight Committee (TOC) - Doug Stead, chair, Stuart Read and Norikazu Shimizu is included in the 2018 ISRM News Journal. Commissions run on four year terms and applications for continuance will be considered at the start of the 2019 – 2023 term of the Board.

ISRM FELLOWS

The highest and most senior grade of membership of ISRM is Fellow, being awarded for outstanding achievement and professional contribution through ISRM. The first Fellows were selected in 2010/2011, and to date thirty have been inducted. The majority (2/3) are from Europe and Asia with the balance shared between the other four regions.

Two Fellows currently reside in Australasia are Ted Brown (2011) and Jian Zhao (2015), with Bill Bamford scheduled to be inducted in September during the Congress.

Vale Prof John Hudson (1940 – 2019)

Prof John Hudson, Past President and friend, passed away on 13 February 2019, in London. John was one of the most active members ever of ISRM - Chairman of the Commission on Testing Methods for 19 years (vice the blue book), Vice President, President and Fellow. He was an influential teacher of rock mechanics and author and editor of several of the most relevant books on the discipline. An Obituary is published elsewhere in this issue.

ISRM 2019 – 2023 Term

My term as Australian Vice President will finish at the Foz do Iguassu Congress in September, and Regional Vice Presidents for the 2019 – 2023 term will be elected at the Council meeting on 15 September.

Under the understanding between AGS and NZGS (two terms Australia, one term New Zealand), Ms Sevda Dehkoda from University of Queensland has been selected and endorsed by AGS and NZGS as Australasian Vice-President for the 2019 – 2023 term.

This will be my final report as Vice President, and I would like to thank NZGS for the support provided over the four year period of my term.

Stuart Read

International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) Regional Report for Australasia

June 2019

The ISSMGE is the pre-eminent professional body representing the interests and activities of Engineers, Academics and Contractors all over the world that actively participate in geotechnical engineering.

1. THANK YOU

This report is brief, as I have to withdraw from all my professional activities to deal with an unexpected and serious health challenge. It has been a great honour and pleasure to have represented our Societies on the ISSMGE Board, and I am disappointed to have to withdraw from that role part way through. I would like to acknowledge the support I have had from the leadership of our Societies as I have carried out this role. Several people stand out as having made a considerable impact during my time as Vice President. Graham Scholey, AGS Liaison for the current Presidential term, has been a constant support and has successfully completed a comprehensive refreshment of the AGS representation on the ISSMGE Technical Committees. Ashe Cooper, NZGS representative on the Young Members Presidential Group, has embraced the opportunities presented by this role and achieved great things, organising the inaugural Corporate Associates debate in New Zealand earlier this year, and forging closer links with young members from around the world at a meeting in Singapore in March. Sukumar Pathmanandavel continues to work tirelessly on behalf of the Corporate Associates Presidential Group, and is an outstanding example to all of us of what can be achieved when one truly believes in the cause. And finally, I wish to thank Professor Mick Pender for encouraging me to accept this role. Mick continues to pursue his many interests in geomechanics and to be recognised for his achievements. He is an outstanding example to us all.

Our Societies are in the process of formalising the appointment of my replacement, who will take over the role until the closing of the international conference in Sydney in 2021.

2. CONFERENCES

The ANZ conference, held in Perth in early April 2019, was a great success. This was due in no small part to the efforts of Michael Smith and Hugo Acosta in particular, who worked tirelessly to create a memorable event. I was greatly honoured to contribute to the welcome session, and learnt much from the keynote speakers and the other presenters. Thank you to all who contributed to this event. Our regional conferences provide an outstanding opportunity to catch up with our colleagues from around the region and to benchmark our own practice against the best of our peers.

Preparations for Sydney 2021 continue, with the venue confirmed and PCO appointed. The Local Organising Committee, led by John Carter, with Graham Scholey's support, has been formed and has had useful discussions with members of the ISSMGE Board to start shaping this as an outstanding event. GeoEng2000 is recognised as the Gold Standard, and John and his team are seeking to create an event that is at least as successful. I encourage you to support them, and this world class event, in whatever way you can.

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.

3. ISSMGE CORRESPONDENCE

The ISSMGE publishes a monthly News and Information Circular and a bi-monthly Bulletin. Links to these are forwarded to NZGS members. AGS members are, I believe, provided with a link to them from the AGS home page. In any event, they are freely accessible from the ISSMGE website. The News and Information Circular covers much that would often be included in this report, so I see little benefit in repeating it here. I encourage you all to read this correspondence to find out what the Society is doing for its members.

4. FORTHCOMING BOARD AND COUNCIL MEETINGS

ISSMGE Council meetings occur mid-way between International Conferences, and the next one will be held in conjunction with the African Regional Conference, in Cape Town in October. The next Board meeting will be held at the same venue, the day before the Council meeting.

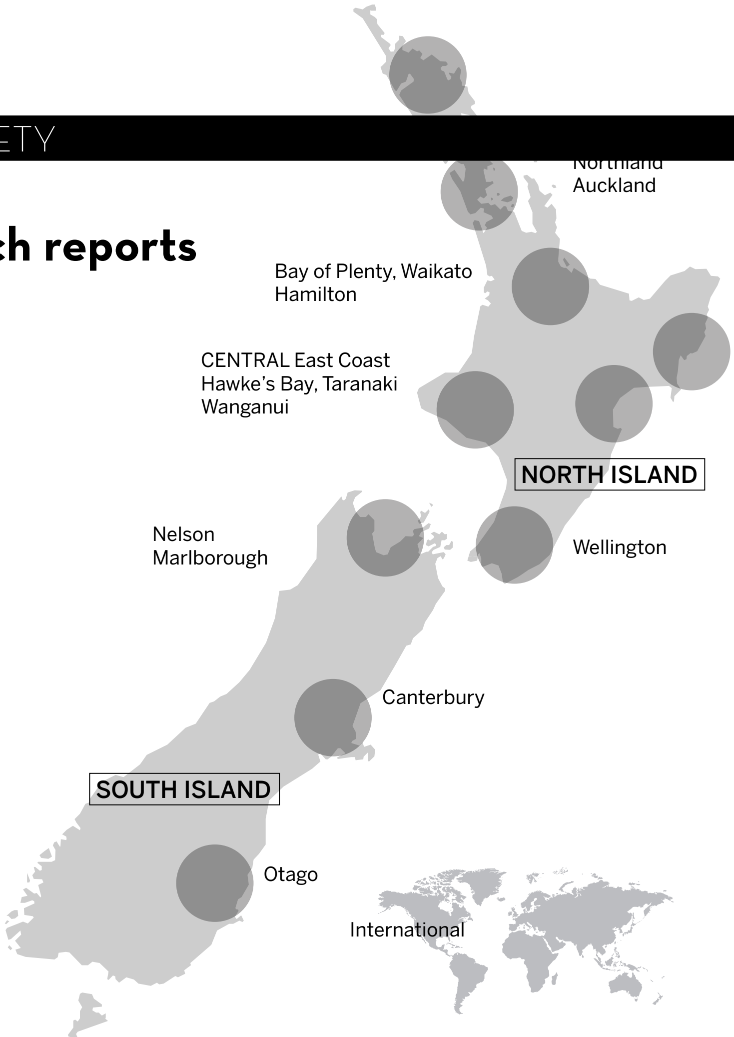
For more information about the ISSMGE, please visit **www.issmge.org**

With very best wishes

Gavin Alexander
ISSMGE Vice President for Australasia
gavin.alexander@beca.com

Graham Scholey
AGS Liaison with the ISSMGE VP for Australasia
gscholey@golder.com.au

Branch reports



AUCKLAND

Auckland has been treated to a continuing series of exciting talks this year. The ISSMGE and NZGS teamed up to present a panel discussion regarding "Collaboration in Geotechnical Engineering – Impact on Research and Project Delivery". The panellists consisted of Ross Roberts, Auckland Council; Liam Wotherspoon, University of Auckland; Susan Tilsley, Beca; and Nick Wharmby, March Construction. Professor Misko Cubrinovski, University of Canterbury, discussed the three most important aspects in the engineering assessment of soil liquefaction with case studies in his 2019 Geomechanics Lecture. Dr Nick O'Riordan, a geotechnical engineer from ARUP, United Kingdom, provided an insight into the interactions when constructing within soils of the Anthropocene age in his 2018 Rankine Lecture titled "Dynamic Soil-structure Interaction". Dr Jonathan Bray, University of Berkley, USA, reminded us of the intricacy of estimating the liquefaction settlement under buildings with a live-streamed revision of the 6th Ishihara Lecture 'Simplified Procedure for Estimating Liquefaction-Induced Building Settlement'.

Our upcoming presentations include:

- "GIS for Geotechnical Professionals" by Colin Mazengarb

Many thanks to the University of Auckland, Geotechnics, Aurecon, and Beca for supporting our talks this year.

WAIKATO

NZGS Geomechanics Lecture

We were very lucky to have Dr Misko Cubrinovski present the 2019 NZGS Geomechanics Lecture in Hamilton on 6 March at the University of Waikato.

Misko took the branch on a journey through three important aspects in the engineering assessment of soil liquefaction, i.e. material characterization of liquefiable soils, insitu state characterization of soils, and system response of liquefiable deposits.

The subject of this lecture was of particular relevance and interest for the Waikato membership. The evaluation of liquefaction susceptibility of our local soils is a hot topic at this time. The emphasis on the complexity of soil liquefaction and risks of over simplification were emphasised and in particular the influence of soil layering on the manifestation of liquefaction… or not.

Thanks to the event sponsor Schick Civil Construction for the kai and refreshments. And thanks to Vicki Moon for the venue organisation.

Future events are coming together nicely:

On 3 July Dr Seokho Jeong from the University of Waikato will be presenting on his recent study of seismic site period of the Waikato Basin using the Horizontal to Vertical (H?V) spectral ratio technique. This follows his recent research pater publication with co-author Dr Liam Wotherspoon from University of Auckland that was presented at the 2019 Pacific Conference on Earthquake Engineering.

A future presentation (date T.B.C) by John Brzeski of Tonkin & Taylor will be on their recent Desktop Liquefaction Study completed for the Hamilton City Council. This comprehensive report was prepared to give Council guidance on when liquefaction assessment is needed and to what level is appropriate. It is obviously of value to practitioners as well, in scoping and executing these assessments knowing the level of detail that Council are anticipating. We hope for an open and stimulating

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

Above: Tony Fairclough, NZGS Chair presenting Chris McGann from the University of Canterbury with the NZ Geomechanics Award

Above: Tony Fairclough, NZGS Chair presenting Misko Cubrinovski from the University of Canterbury with the award for the NZ Geomechanics Lecture

PHOTOS: Duncan Henderson

Q and A session following Johns talk.

Further details on these events will be advertised soon.

We are also working on a site based event later in the winter. TBC

WELLINGTON

We have had a very busy few months here in Wellington with a wide variety of events passed and planned. We are really excited to have Safia Moniz join the committee. Safia has been busy assisting Shirley Wang with preparation for our upcoming YGP Conference.

Recent events include:

May - Screw pile installation, CentrePort, coordinated by Piletech, really well attended with 70+ people, live demonstration followed by tech talk and drinks/nibbles

- **May** – Marc Elmouttie presented on Mining Geoscience Research at the CSIRO, thanks to Griffiths Drilling for sponsorship
- **March** - Thank you MBIE (Nathan Schumacher) for hosting the 58th Rankine Lecture, a fascinating presentation on dynamic soil-structure interaction presented by Nick O'Riordan and attended by a diverse group of ground investigation contractors and consultants
- **March** – Misko Cubrinovski's NZGS Geomechanics Lecture was hosted by WSP Opus, Misko presented on important aspects in the assessment of liquefaction, very relevant to Wellington practitioners with CentrePort's facility right on our door step!
- **February** – A Joint presentation by Martin Wilson (Abseil Access) and Eric Ewe (Geofabrics) on rockfall mitigation measures.

Events planned:

- YGP Conference, Shirley is coordinating the Wellington event. We are still looking for two mentors, so if you are a seasoned practitioner with a coach's style, then lease contact Shirley Wang **(SWang@tonkintaylor.co.nz)**.
- GIS for Geotechnical Professionals, An Evening Presentation, by Colin Mazengarb **(RSVP to jerry.spinks@jacobs.com)**

★ 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**

CHRISTCHURCH

It's been a busy first half of the year in Canterbury with a number of high profile events including the Geomechanics Lecture (Prof. Misko Cubrinovski), the Geomechanics Award (Chris McGann et.al) and a number of lectures from visiting speakers including Prof. Steve Kramer (2018 H. Bolton Seed Lecture), ~~and~~ Dr. Nick O'Riordan (58th Rankine Lecture) and Prof. Jonathon Bray (6th Ishihara Lecture). On top of that we have hosted presentations from industry specialists Geofabrics and Abseil Access making use of the new Turanga Library in Christchurch – A great space!

We have had some great turn-outs to events, with a full-capacity 120 people for Prof. Kramer's presentation, and around 75 for Prof. Bray. We have also trialled a new online RSVP system to give us an idea of numbers attending which has worked well for ensuring catering and room sizes are adequate.

Prof. Kramer and Prof. Bray's talks were also live-streamed around the country to allow wider access to these one-off presentations. A big thanks to Dr. Mark Stringer from the University of Canterbury for organising this.

A huge thankyou to all of our event sponsors, the meetings would be a different environment without the food and beer!

On the 21st June we have our first Christchurch YGP Mini Symposium for South Island based members of the New Zealand Geotechnical Society who are under 35. This is a one day event locally based, less formal version of the highly successful ANZ YGP conference.

As always we would love to hear your feedback on previous events and welcome suggestions for future events.

See you at the next event!

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 and is based in Tauranga.

Kim's experience includes earthworks, seismic assessments, building foundation design, 3 waters projects and resilience workshops. Kim is also a Safety in Design facilitator and the Geotechnical Lead for the Safe Roads Alliance in the Bay of Plenty.

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

Sirini De Silva
I work with RDCL as an Engineering Geologist. I graduated with a BSc(Hons) from UoA in 2017 and briefly worked in Kaikoura for the NCTIR project. I have experience in geotechnical site investigations, ground modelling, materials testing, site hazards and liquefaction assessments.
sdesilva@rdcl.co.nz

Jerry Spinks
Jerry is a Chartered Professional Engineer, who graduated from Canterbury University (Civil) then worked in the UK on a large transport and windfarm projects. Since returning to NZ in 2010 he has developed a keen interest in building seismic assessments and strengthening. He enjoys working closely with structural engineers to develop thorough and reliable assessments of the ground.
Jerry.Spinks@jacobs.com

Safia Moniz
Safia is a Chartered Professional Engineer who has worked in the Caribbean and New Zealand since graduating from the University of the West Indies with a Degree in Civil Engineering (Hons) in 2004. She completed a Masters in Geotechnical Engineering at MIT in 2009. Recent projects include deep foundation design and ground improvement for buildings and bridges.
safia.moniz@holmesconsulting.co.nz

WELLINGTON

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.
aimee.rhodes@opus.co.nz

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.
SWang@tonkintaylor.co.nz

NELSON

Kylie Johnson
I'm an Engineering Geologist for CGW Consulting Engineers based in Nelson. I have been a geologist over much of New Zealand and a keen member NZGS for the past 7 years. I look forward to being part of the team with Rebecca to bring great talks and field trips to our region.
Kylie@cgwl.co.nz

Rebecca.McMahon
A Geotechnical Engineer for Beca, I have been a keen NZGS member for the last seven years and am looking forward to the opportunity to assist Kylie with running events for our region. As I am also a committee member for Engineering NZ. Kylie and I will be looking for ways to combine some site visits and meetings to make the most of the awesome people, projects and places we have here in Nelson.
Rebecca.McMahon@beca.com

CANTERBURY

Duncan Henderson

Duncan is a Geotechnical Engineer at Tonkin & Taylor in Christchurch where he has been since October 2017. He has been involved in a range of projects, most recently in the Geotechnical Structures design team for the NCTIR alliance. Duncan completed his BE(Hons) in 2010 and Masters of Engineering in 2013, both at the University of Canterbury.

DHenderson@tonkintaylor.co.nz

Charles McDermott

Charles is a Senior Geotechnical Engineer with Miyamoto in Christchurch. He is originally from the UK where he graduated with a BEng (hons) in Civil Engineering from Kingston University (2007). Charles moved to Christchurch in 2013 where he has been involved in earthquake recovery and the design of a number of large infrastructure projects.

cmcdermott@miyamotointernational.com

OTAGO

Nima Taghipouran

Nima is a chartered professional engineer based in the WSP-Opus office in Dunedin. Nima graduated from the University of Auckland in 2012. He has been involved in a wide range of medium to large scale projects throughout the lower North Island. His areas of interest include foundation and retaining wall design, slope stabilisation and earthquake engineering.

nima.taghipouran@wsp-opus.co.nz

Eli Maynard

Eli is a Geotechnical and Water Resources Engineer at GeoSolve in Dunedin. He has 6 years' of post-graduate experience gained from a wide range of projects involving water storage dams, flood protection schemes, deep foundations with piling and ground improvement, landslide remediation, irrigation schemes and groundwater evaluation.

emaynard@geosolve.co.nz

QUEENSTOWN

Paul Jaquin

Paul is a Chartered Professional Engineer, and is Work Group Manager for Buildings and Structures in the WSP Queenstown office. He works across a range of disciplines, including building foundations, bridge assessment, retaining walls, rockfall and landslide analysis. Paul holds a PhD in unsaturated soil mechanics and is a recognised expert in mud brick construction, providing advice and engineering expertise internationally.

Paul.Jaquin@wsp-opus.co.nz

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.

Welcome to the first edition of NZ Geomechanics News for 2019.

Our branches have been hugely busy organising events and presentations. Our branch co-ordinators are always happy to hear of any ideas you have. We have been fortunate to live stream a Presentation recently to those outside the main centre which was hugely popular and I would like to thank the University of Canterbury for enabling this to happen. I would also like to thank our branch co-ordinators who bring such enthusiasm to their role. We also welcome two new branch coordinators to Nelson, a new coordinator in Wellington and a new branch has opened up in Queenstown.

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 2018-2019

POSITION	NAME	EMAIL
Chair	Tony Fairclough	chair@nzgs.org
Vice-Chair & Treasurer	Ross Roberts	treasurer@nzgs.org
Immediate Past Chair	Charlie Price	price.charlie@outlook.com
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Elected Member	Camilla Gibbons	Camilla.Gibbons@aurecongroup.com
Elected Member	Eleni Gkeli	eleni.gkeli@wsp-opus.co.nz
Elected Member	Rolando Orense	r.orense@auckland.ac.nz
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Co-opted YGP Representative	Aine McCarthy	aineymcc@gmail.com
IAEG Australasian Vice President (until January 2019)	Mark Eggers	Mark.Eggers@psm.com.au
IAEG NZ Representative (Taking over Australasian VP after January 2019)	Doug Johnson	DJohnson@tonkintaylor.co.nz
ISSMGE Australasian Vice President	Gavin Alexander	Gavin.alexander@beca.com
ISRM Australasian 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.

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/

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

National and International Events

2019

7-11 September
Iguassu Falls, Brazil
ISRM 14th International Congress of Rock Mechanics

1-6 September
Reykjavik, Iceland
The 17th European Conference on Soil Mechanics & Geotechnical Engineering

29 Sept – 2 October
Guimareas, Portugal
3rd International Conference on IT in Geotechnical Engineering (3rd ICIT2019)

10 September
Poland
Third International Conference "Challenges in Geotechnical Engineering" CGE-2019

11-13 September
Vienna, Austria
International Symposium on SPH & other Particle – based. Continuum Methods and their applications in Geomechnics

13-18 September
Foz do Iguassu, Brazil
ISRM 14th International Congress of Rock Mechanics

7-10 October
Cape Town, SA
17th African Regional Conference on Soil Mechanics and Geotechnical Engineers

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

14-17 October
Taipei, Taiwan
16ARC Asian Regional Conference on Soil Mechanics & Geotechnical Engineering

15-18 October
Chicago, USA
44th Annual Conference on Deep Foundations

17-20 November
Cancun, Mexico
XVI Pan American Conference on Soil Mechanics & Geotechnical Engineering

28-29 November
4th International Conference on Geotechnics for sustainable Infrastructure Development

5-7 December
Lahore, Pakistan
15th International Conference on Geotechnical Engineering, & 9th Asian Young Geotechnical Engineers Conference

2020

June 2020
Trondheim, Norway
EUROCK2020

29 June – 1st July
Cambridge, England
TC204 Geotechnical Aspects of Underground Construction in Soft Ground

14-16 July
Greater Naida, India
7th International Conference on Recent Advances in Geotechnical Earthquake Engineering & Soil Dynamics

24-26 July
Lisbon, Portugal
4th European Conference on Unsaturated Soils – Unsaturated Horizons

16-19 August
Texas USA
2020 International Symposium for Offshore Geotechnics ISFOG

30 August – 2 September
Illinois, USA
4th International Conference on Transportation Geotechnics (ICTG)

7-11 September
Budapest, Hungary
6th International Conference of Geotechnical &Geophysical Site Characterization

1-3 October
Perth, WA
Slope Stability 2020 Symposium

13-16 October
Oxen Hill, USA
45th Annual Conference on Deep Foundations

16-18 October
Dunedin, New Zealand
NZGS Symposium 2020 – Good grounds for the future

2-6 November
Kyoto, Japan
Fifth World Landslide Forum

2021

12-17 September
Sydney, Australia
ICSMGE2021
20th International Conference on Soil Mechanics and Geotechnical Engineering

1 June
Torino, Italy
EUROCK2021

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