NEW ZEALAND GEOTECHNICAL SOCIETY INC

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# NZ GEOMECHANICS NEWS

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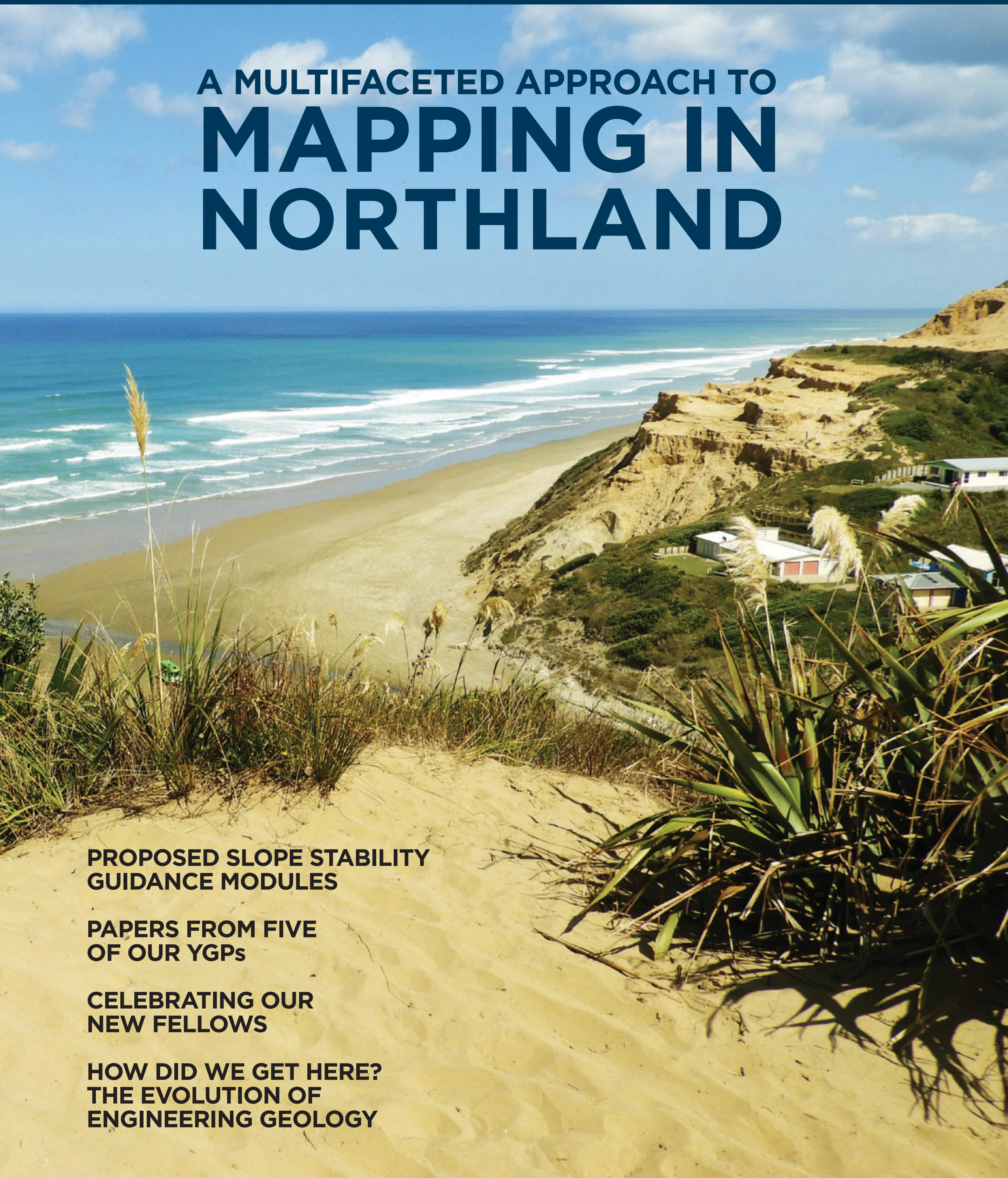

## A MULTIFACETED APPROACH TO MAPPING IN NORTHLAND

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PAPERS FROM FIVE OF OUR YGPs

CELEBRATING OUR NEW FELLOWS

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# CONTENTS JUNE 2022

**04** Editorial
**06** Chair's Corner
**10** Congratulations to our new Engineering New Zealand Fellows

## NEWS
**12** In Brief

## FROM THE ARCHIVES
**14** A light hearted look at metrication

## FEATURE
**34** Assessment of Current Practices Around the Use of the Standard Penetration Test
**42** Quantifying the pumice content of soil through utilisation of the unique characteristics of pumice
**50** A Multifaceted Approach to Mapping in Northland
**58** Site Behaviour and Management of Sensitive Volcanic Soils
**64** Construction in Christchurch's Soft Peat Deposits

## TECHNICAL
**20** Proposed Slope Stability Guidance Modules
**26** How did we get here? A biased view of the development of engineering geology in New Zealand
**70** Cantilever Pole Walls on Sloping Ground and in Cohesive Soils with Bearing Capacity Varying With Depth
**90** What's On at Waka Kotahi
**92** NZS 4431:2022

## SOCIETY
**94** Student poster Competition

## OBITUARIES
**98** Professor Michael J Pender
**100** Prof. David Hiley Stapledon

**102** International Society Report – ISSMGE
**104** Branch Reports
**107** Branch Coordinators
**110** Management Commitee List, Editorial Policy, Subscriptions, Membership
**111** Advertiser's listing, Advertising
**112** International Events

**COVER IMAGE:** Baylys Beach, Northland (see page 50)
Credit: Lauracieslak | Dreamstime.com

# The New, The Not-So-New, and Something a bit Older

**IN THIS ISSUE** we have many papers from those in the earlier stages of their careers as well as those in the later stages passing on their wise words. Our NZGS Young Geotechnical Professionals who were not able to attend the Cairns conference were invited to have their paper included in *NZ Geomechanics News*. We were very pleased to receive the five papers that we include in this issue and perhaps not quite as glamorous as a trip to the sunny north of Australia, at least there are fewer H&S risks in reading this publication than the local wildlife in northern Australia poses!

We also have the winners of our annual student poster competition which again presented our judges with a very high standard of entries.

It is great to see such a broad range of topics presented by some of the generation who will be our future industry leaders, and a bit of a treat for those of us a little beyond the "young" category to be able to read.

Likewise, we have some wise words from some of the leading names in our industry who are a little further along in their careers than the YGPs. I wonder in another 30 plus years or so, who of our YGPs will become the next leading names in the NZ industry?

Along the same theme, we decided to have a look in the archives to see what was happening 50 years ago in the June edition of *NZ Geomechanics News* in 1972. We found an interesting article that was clearly a big topic of the day, causing unrest and likely a little nervousness in the industry – metrification. I cannot imagine the ensuing carnage if this was to happen today, the computers would struggle, the modelling programs would need rewriting, the automated calculations wouldn't work, jeepers there would be a mess! It only reinforces in my mind the old "rubbish in, rubbish out" saying and that if you cannot write the calculation by hand, you probably don't understand if the program is giving you the right answer.

Reading the "three generations" of Geomechanics contributions makes me appreciate the huge change that our industry has gone through in just a few decades, one of our "Not-so-New" articles presents the evolution of Engineering Geology as a recognised discipline in New Zealand. What is going to be the key articles of the day in this magazine in 2072?

This issue is also "New" in that we welcomed Robert Kamuhangire to the editorial team, but we have also sadly said farewell to Don MacFarlane as Editor. Don has given many hours over the years to *NZ Geomechanics News* and more broadly to NZGS and the Engineering Geology industry in New Zealand, he has also recently been made a Fellow of Engineering New Zealand. We would like to take the opportunity to say a big Thank You to Don and all the best for the next adventure!

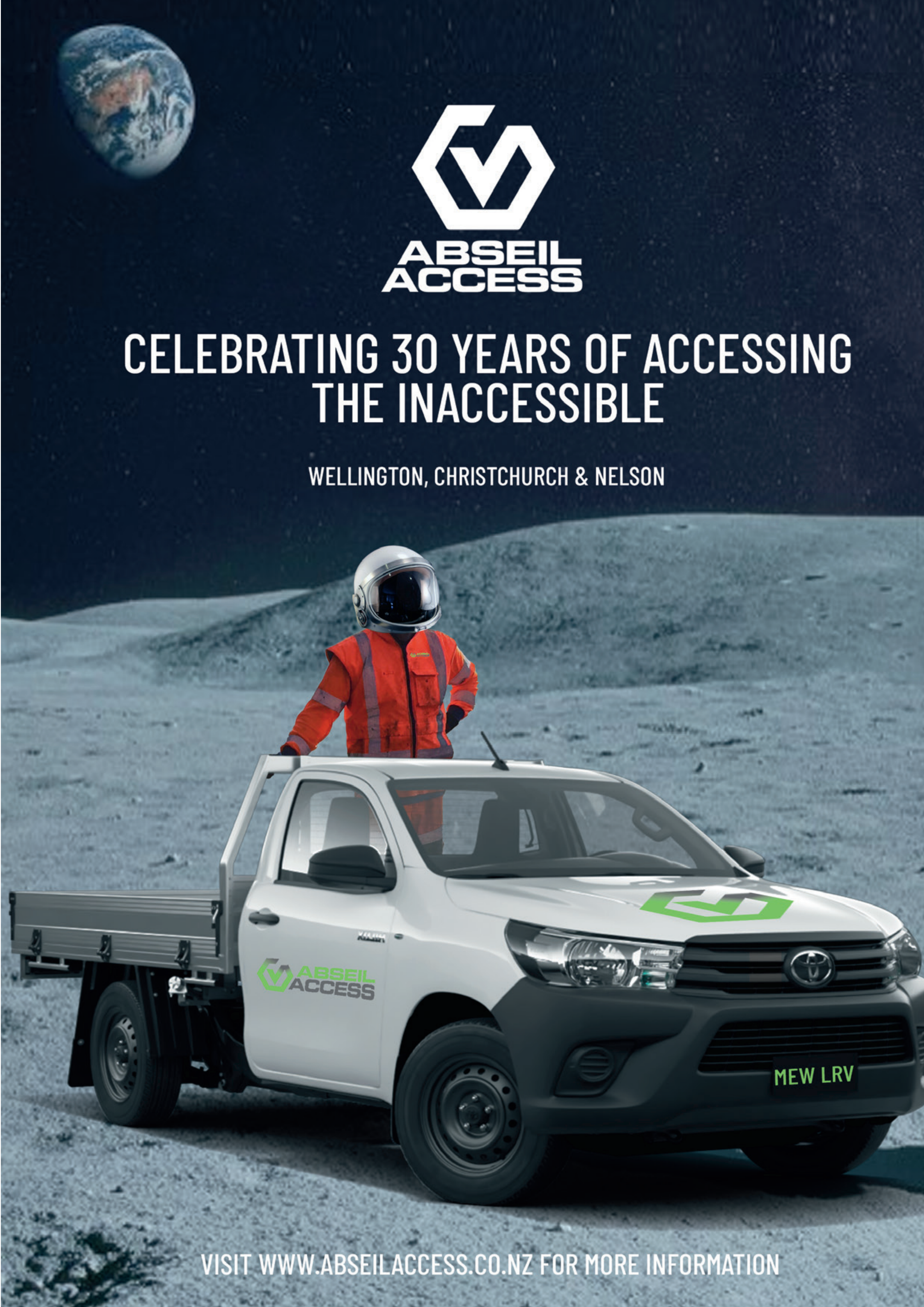
*Camilla Gibbons*

*Camilla Gibbons is a Principal and engineering geologist with Aurecon. She worked in the UK before moving to New Zealand in 2008 "for a year". The Canterbury earthquakes inspired what has now become her real interest in geohazards and rockfall in particular and she has since enjoyed working on projects combining this with her other interest of improving efficiencies and improving safety by the effective use of digital technology.*

***NZ Geomechanics News*** **co-editor**

*Robert Kamuhangire is a principal geotechnical engineer with KGA Geotechnical Group, based in the Christchurch office. He previously worked in the UK predominantly on large infrastructure projects, prior to arriving in New Zealand in 2012 to be part of the Christchurch Rebuild. In addition to forgetting his "perpetual warm/rain jacket" during his first summer in New Zealand (thanks to the consistent good summer weather), he has been blessed to work on a number of claim assessments, new residential and commercial buildings, subdivisions, retaining walls, deep and shallow foundations, and ground improvement schemes among other things.*

***NZ Geomechanics News*** **co-editor**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# From the Chair

*Eleni Gkeli is an Engineering Geologist with over 25 years of experience predominantly from her involvement in large infrastructure projects in Greece and in New Zealand in the transport, building and water sectors. Eleni has been working in New Zealand since 2012 and has been an elected member of the New Zealand Geotechnical Society since early 2016, holding a variety of positions and roles since that time.*

**Eleni Gkeli**
*Chair, Management Committee*

**WE HAVE ALREADY** flown through the first half of 2022, and despite the challenges of the domestic and international environment, the geotechnical industry in New Zealand continues to do well. Investment in infrastructure remains strong and based on New Zealand's first-ever long-term infrastructure strategy that was released by Te Waihanga / New Zealand Infrastructure Commission in May 2022, it is expected to continue to remain strong to keep up with the country's needs. The strategy highlighted that investment needs to become smarter, better planned, adaptive to change and more focused on resilient and sustainable solutions. This is the message that we need to take as geotechnical professionals and apply these principles in our everyday practice.

As we are gradually moving to normality with respect to the COVID-19 Pandemic, the key challenges that our industry is facing now are associated to supply chain restrictions affecting construction and the workforce shortages. The latter is aggravated by the fact that the borders have re-opened, and travel is restarting. We all have personal experiences of young colleagues resuming their plans to leave for overseas experience. This is a challenge that our industry must respond to, by providing enough support, development opportunities, and prospects for younger geotechnical professionals in order to attract them to either stay in New Zealand or plan to come back.

Responding to the "re-opening of the community" after the softening of the COVID-19 restrictions, we are anticipating that our in-person NZGS activities will ramp up in the next few months. We will continue using the technology for some of our meetings and events, as we are now used to it, and because it provides means for efficient, and environmentally sustainable gatherings. But where in-person attendance provides more value compared to the on-line, we will encourage it, to ensure that the geotechnical community remains strongly connected and engaged.

I have included a brief overview of some of the NZGS activities in the following sections.

## WEBINARS

### REVISED GEOTECHNICAL MODULES

The use of the revised Earthquake Geotechnical Guidelines issued in November 2021 is becoming well embedded into the geotechnical practice. NZGS understands that questions have been generated in the first months of use and decided to organise webinars to assist the industry. These webinars are supported by MBIE.

Priority is the revised Module 1, which includes the interim update of the seismic hazard for six locations across New Zealand. The webinar on Module 1 will be delivered in early July 2022 and will be free to NZGS, NZSEE and SESOC members. I would like to thank the lead authors of Module 1, Misko Cubrinovski (University of Canterbury) and Kevin McManus (McManus Geotech) for offering their time for this webinar, as well as Stuart Palmer (Tonkin and Taylor) for presenting on the joint NZGS, NZSEE and SESOC advisory "Designing for Uncertainty", expected to be issued around June 2022. I would also like to thank the NZGS Education officer Emilia Stocks (Tonkin and Taylor), for taking the lead in this important training for NZGS.

Similar webinars will follow on the other revised Modules later this year. We would like to make them as interactive as possible and tailored to the industry needs. So please send your questions and observations in the links provided in our weekly newsletter announcements. These webinars do not intend to override or supersede any formal and more intense training; on the contrary, your feedback will guide us to target future training to the real needs. NZGS will continue collaborating with MBIE and EngNZ for developing this training.

### OTHER WEBINARS

Rolando Orense (University of Auckland) continues the good work organising online lectures by international experts. In February 2022, Professor Junichi Koseki (University of Tokyo) presented on "Experimental Investigation into Multiple Occurrence of Liquefaction"; in March 2022, Professor Charles W. W. Ng (Hong Kong University and outgoing President of ISSMGE) presented on Design Analysis of debris flow barriers; in April 2022, Gennaro G. Marino (Marino Engineering Associates) presented on Mining Subsidence.

All webinars were very well attended and so far, have covered a variety of geotechnical themes. We will continue organising webinars but seek opportunities to invite international experts for more extended and in-depth in-person workshops, as travel restrictions are lifted.

We also had very interesting webinars from local experts, organised by the local NZGS branches, which were very well attended, such as on the Abbotsford Landslip Disaster by Nick Rogers (Tonkin and Taylor), the 3D Geological Modelling in Wellington presented by Matt Hill (GNS Science), the Hamilton Seismic Hazard, presented by Dr. Max O. Kluger (University of Waikato). I would like to thank everyone involved in organising these presentations and keeping us engaged and connected.

## NZGS GEOMECHANICS LECTURE AWARD

The NZ Geomechanics Lecture is the premier award of the New Zealand Geotechnical Society. It is presented by a person prominent in Geomechanics who can, in the presentation, contribute a statement of significance and value relevant to New Zealand.

I am pleased to announce that the NZGS Management Committee has decided to present this year's Geomechanics Lecture Award to Ann Williams, Manager Geotechnical and Technical Fellow in Beca. Ann had an inspiring career of over 30 years in engineering geology, hydrogeology and managing geotechnical risks in infrastructure projects. She is a Past Chair of the New Zealand Geotechnical Society, and a past Vice President of the International Association for Engineering Geology and the Environment. Ann was instrumental in establishing the register of Professional Engineering Geologists (PEngGeol) in New Zealand and was recently elected as a member of the Engineering New Zealand Board.

The choice of the Geomechanics Award winner was very difficult, and Ann was selected amongst a number of prominent names, each one having made a significant contribution to Geomechanics. The Management Committee is considering presenting this award more frequently in the future, to be able to reward the great achievement in our profession.

Ann will present her Geomechanics Lecture in-person in a number of centres across New Zealand in the coming months, and we hope that this exceptional Lecture for our society will mark the recapture of our in-person gatherings.

## NZGS OPERATIONS MANAGEMENT COMMITTEE

The term of our Australasian Vice President for ISMMGE, Phil Robins (Beca) has come to end, coinciding with the 20th ICSMGE in Sydney. NZGS would like to thank Phil for the excellent representation of the NZ geotechnical community in this international organisation. Alongside this demanding role in ISSMGE, Phil has also been an active member of the NZGS Management Committee. His mentorship will be missed, and we hope that he will remain involved in the NZGS activities.

Following tradition, the ISSMGE Vice Chair role will be handed over to a representative from the Australian Geomechanics Society, Graham Scholey. The role of the NZ representative, to work closely with Graham to support our interests in ISSMGE, will be filled by Rolando Orense (University of Auckland). With his Academic background and international exposure, Rolando is perfectly suited to represent NZGS in this international organisation.

## NZ GEOMECHANICS NEWS

The editorial team of our magazine will be changing in the next few months. Don Macfarlane (Consultant with AECOM), who has had the Editor role since 2018, is gradually stepping back. Don has been serving Geomechanics News exceptionally for the past few years, and NZGS is grateful to him for bringing his experience and wisdom into the role.

Camilla Gibbons (Aurecon) will be stepping up to the role of Editor, and the role of co-editor was kindly taken by Robert Kamuhangire (geotechnical engineer, KGA Geotechnical), who expressed interest in the open NZGS invitation. I would like to thank the other candidates who volunteered for the role.

## YGP ACTIVITIES

The NZGS Young Geotechnical Professionals group led by Helen Hendrickson (WSP), continues to be very active. This year we are looking forward to the 14th ANZ YGP Conference, which will occur in November in Rotorua. This is an event particularly anticipated, following the cancellation of the previous ANZ YGP Conference in Cairns, after several postponing attempts. Organising for the YGP Regional Mini-Symposia has also started. Participation in these Symposia will be free to our members this year, fully subsidised by NZGS.

The YGP Geotechnical Engineering Basics training posters project is also progressing well, with posters issued for the membership feedback and a new expression of interest published for the "Developing an Engineering Geological Model" poster. I would like to thank Miles Buob (Tonkin and Taylor), who following the issue of the draft posters is stepping down from this role. Wendy Weng (Tetra Tech Coffey) will be taking over the YGP Training Coordination role and bringing these posters from draft to final publication.

## CANDIDACY FOR ISRM CONGRESS 2027

NZGS has submitted its candidacy to host the 2027 Congress of the International Society of Rock Mechanics in Christchurch. Paul Horrey (Beca) the NZ Representative for ISRM is leading this fantastic initiative for NZGS, with Stuart Read (GNS Science and Past ISRM Vice-President), Romy Ridl (University of Canterbury) and Christoph Krauss (Beca) working alongside him. The team will support our bid by presenting in the next ISRM Council meeting in Asunción (Paraguay), the venue for the 2022 ISRM International Symposium, later in the year. This NZGS initiative has been strongly supported and sponsored by Tourism New Zealand, whose contribution provided a different weight and calibre in our proposal.

## NZGS PROJECTS

### CLIMATE CHANGE INITIATIVES

NZGS is developing a Climate Change Focus Group, to enable more targeted work in this area. It is beneficial for geotechnical professionals to come together, understand the current legislation in New Zealand and the different milestones and targets that need to be achieved with respect to embodied carbon reduction and other aspects of our work in relation to climate change. The focus group is aiming to develop tools, guidelines or educational material to help geotechnical practitioners adapt to the requirements related to climate change, from a geotechnical point of view.

Based on the feedback from the 2021 Climate change Symposium, there is considerable interest in the geotechnical industry across New Zealand to get involved in this topic. This focus group comes to address one of the biggest hurdles currently existing, the lack of an industry body who offers opportunities to participate and contribute. Martin Larisch (Jacobs) has taken the lead in advancing this NZGS initiative. There will be an open call from NZGS for contributors to this group. Please contact Martin to find out more about this exciting opportunity.

### TECHNICAL DOCUMENTS

The Piling Specification led by Tony Fairclough (Tonkin and Taylor), which is developed in collaboration with SESOC, is progressing well. I am pleased to report that we expect to have a first issue of the document in Q2 – Q3 of 2022.

The Anchor Specification, led by Sam Glue (Beca), is also progressing well, and a draft document is currently with the NZGS Management Committee for review. We expect that this document will also be issued by the end of this year.

NZGS is planning to organise presentations or short seminars in the use of the two documents after they are published, aiming to collect feedback from the membership as well.

NZGS is advocating for the development of a series of guidance documents on Slope Stability, which has consistently been identified as a gap in various NZGS surveys. You will be able to find more details on the NZGS vision for these documents in an article in this issue of the magazine. Please provide your opinion and feedback to the NZGS lead for this project Richard Justice (ENGEO).

### INDUSTRY COLLABORATION

NZGS and SESOC in collaboration with Engineering NZ have issued guidance for structural engineers for decision-making for residential development projects. A template geotechnical report has also been issued in conjunction with the decision-making flowchart, to be used by structural engineers. In the same collaboration scheme, we are in discussions for developing guidance for the design of warehouses, as a joint document between EngNZ, SESOC and NZGS amongst other societies and organisations. I would like to acknowledge the contribution of Martin Pratchett from EngNZ who is leading these projects and Ayoub Riman, the NZGS Management Committee representative on this work.

Stuart Palmer (Tonkin and Taylor) is representing NZGS in the working groups for updating and maintenance of the Earthquake Prone Buildings Guidelines led by MBIE and in the development of the NZGS, NZSEE and SESOC joint advisory on "Designing for Uncertainty", which is expected to be issued around June 2022.

NZGS continues to be represented in the different working groups established by MBIE for the update of the Building Code, following the update of the National Seismic Hazard Model, by Rick Wentz (Wentz-Pacific) and Andreas Ginnakogiorgos (Miyamoto).

### OCCUPATIONAL REGULATION

In March 2022, Cabinet made the decision to progress with the new regulatory regime for professional engineers that was proposed by MBIE, following the public consultation carried out in 2021. The proposed details of the scheme are currently being worked through by MBIE. We understand legislation will be introduced to Parliament later in 2022 or early 2023, and it is expected that after this legislation is passed the new regime will take up to six years to be fully implemented.

We expect that there will be opportunity for NZGS and the wider industry to provide feedback on the draft bill later this year before it goes to the Parliament. NZGS is working closely with Engineering New Zealand to monitor the developments and provide timely updates to the proposed legislation.

We are taking action to advocate for the inclusion of Engineering Geologists in the new registration system, which is uncertain based on current knowledge. We will provide more detailed updates to the membership through our weekly newsletter on our actions on this matter.

### ENGINEERING NEW ZEALAND FELLOWSHIP

Several Geotechnical Engineers and members of NZGS have been recently awarded Distinguished Fellowship and Fellowship by Engineering New Zealand. A special presentation of those individuals "who have made it to the top of the engineering profession and achieved outstanding results" is included in this issue of the *NZ Geomechanics News.* I would like to extend my congratulations on behalf of the NZGS Management Committee to everyone awarded.

### CONCLUSION

As the disruptions and restrictions imposed by the pandemic are gradually easing, our activities are picking up. I would like to repeat the invitation included in my December 2021 Corner: There is a lot happening and the Society needs your contribution to succeed. I would like to invite all our members to participate in the NZGS activities and respond to our frequent calls for various contributions.

Please get in touch with me or the Management Secretary or any other member of the Management Committee, if you would like to support one of the NZGS projects or if you have ideas with respect to our activities (email us at chair@nzgs.org or secretary@nzgs.org ).

# CLL

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# Congratulations to our new Fellows

## Seven of our members recognised by Engineering New Zealand

**CONGRATULATIONS TO OUR** NZGS members below who were recently made Engineering New Zealand Fellows in recognition of their outstanding impact on engineering in Aotearoa. The Fellowship is the pinnacle of the profession and is awarded to the most highly experienced engineers and engineering geologists. Please join us in congratulating the members on the achievement and we wish them an enjoyable time at the upcoming Fellows Dinner in August 2022. The highlights below are with permission from Engineering New Zealand.

### BRENDON BRADLEY

Brendon Bradley is Professor of Earthquake Engineering in the Department of Civil and Natural Resources Engineering at the University of Canterbury and Director of QuakeCoRE: The New Zealand Centre for Earthquake Resilience. His areas of interest include engineering seismology, strong ground motion prediction, seismic response analysis of structural and geotechnical systems, and seismic performance and loss estimation methods.

Brendon has also been a Director of Bradley Seismic Limited since 2010, providing consulting services in several areas of earthquake engineering.

Congratulations Brendon and your cutting-edge research keeps helping shape the profession for the better, thank you.

### DR JAN KUPEC

Dr Jan Kupec is a Geotechnical Principal at Aurecon. His expertise includes deep and shallow footings, retaining walls, slip and rock fall remediation, embankments and dams, as well as risk-based assessment of slope stability. He considers his work with the Canterbury Earthquake Recovery Authority and subsequently with Land Information New Zealand as Chief Geotechnical Advisor on the Christchurch rebuild to be career highlights.

Jan is also an Urban Search and Rescue (USAR) specialist with Fire and Emergency NZ. He responded to Christchurch and Kaikōura earthquake events and deployed with the USAR team to Japan after their 2011 tsunami disaster.

He has published over 70 technical papers and is passionate about resilience, sustainability and innovation.

Congratulations Jan and thank you for being a good ambassador at CERA/LINZ at a challenging time for the Christchurch community.

### STUART FINLAN

Stuart Finlan's career spans over 35 years on three continents in the geotechnical, geo-environmental and civil engineering spheres across a diverse range of projects. These include industrial and commercial developments, horizontal infrastructure, mining subsidence, landfills, land instability and contaminated land assessment and remediation.

Stuart is currently the Lead Advisor-Geotechnical at Waka Kotahi NZ Transport Agency where he sets the strategic direction, policy and standards for geotechnics. He is recognised nationally and internationally for his innovation and technical expertise and has been involved in the development of several national design guides.

Stuart has helped lead and govern a number of industry committees and organisations, as well as project alliance boards within Waka Kotahi.

Congratulations Stuart and thank for your ongoing support to both the engineering community and to Waka Kotahi.  Your drive to improve the industry through innovation and specific design guidance for Waka Kotahi is greatly appreciated.

### DON MACFARLANE

Don Macfarlane has made significant contributions to engineering geology within New Zealand over more than 40 years. During this time, he has led teams working on high profile and geologically complex projects such as the Clyde Power Project, Manapouri Second Tailrace Tunnel Scheme, Project Aqua and the Port Hills.

Don exemplifies industry leadership, and his body of work has been referenced by geologists and engineers throughout the country. He is not one to seek the limelight and instead inspires those around him while quietly contributing to improving the knowledge and practices of engineering geology.

Congratulations Don and also thank you for the exceptional contribution as the *NZ Geomechanics News* editor over the past few years.

### STUART PALMER

Stuart Palmer is a Technical Director–Earthquake Foundation Engineering with Tonkin + Taylor. Stuart is actively involved in the seismic design and assessment of buildings in collaboration with structural engineers. He has authored more than 20 papers in earthquake foundation engineering and was a lead author of the 'Seismic assessment of existing buildings guideline', section C4 'Geotechnical considerations'. He has been providing input to New Zealand's 'Low Damage Seismic Design' handbook and review of the New Zealand Geotechnical Society's earthquake engineering modules. A focus of his work continues to be assessment of liquefaction and lateral spread potential of Wellington's reclaimed waterfront and design of building foundations allowing for this potential.

Congratulations Stuart and thank you for continued contribution via the different technical societies.

### YOLANDA THORP

Yolanda Thorp is a geotechnical engineer with over 35 years' experience. Her current role is that of Technical Director, Project Director and Team Leader at Tonkin + Taylor Ltd.

Yolanda has contributed to the design and construction of major infrastructure roading and rail projects for a major portion of her career. These include Auckland's City Rail Link, the Puhoi to Warkworth motorway, Lincoln Road Interchange and Maioro Interchange.

Yolanda derives great satisfaction from meeting the challenges of design and construction and delivering an asset that improves New Zealand's infrastructure, as well as mentoring and assisting the career development of engineers across the industry

Congratulations Yolanda and thank you for your contribution to the major infrastructure space over the years.

### DR SJOERD VAN BALLEGOOY

Dr Sjoerd van Ballegooy is the Expertise Director at Tonkin + Taylor Ltd for the Geotechnical Group, comprising over 200 geotechnical engineers and engineering geologists. He is responsible for overseeing the technical development and technical mentoring of the engineering staff, technical liaison with the industry and research institutions, incorporating latest research into engineering practice and ensuring that T+T continues to be at the forefront of geotechnical engineering.

Sjoerd has extensive experience in earthquake engineering, including seismic site response, liquefaction, lateral spreading, effects on structures and ground improvement, hazard mapping, earthquake loss modelling, earthquake resilience assessment and stakeholder engagement. His work in these areas has included expert evidence work for hearings, mediations and the courts.

Congratulations Sjoerd and thank you for your contribution to the profession particularly following the Canterbury Earthquake Sequence.

# NEWS

## NEW EQC FUNDED RESEARCH CONFIRMS MAJOR SEISMIC EVENTS IN WEST OTAGO

**A PRESS RELEASE** from EQC on 24 March 2022 indicated that EQC-funded seismic research on the Nevis Fault in western Otago confirms that this low-seismicity area has the potential to create an earthquake similar in size to that of the 2010 Darfield Canterbury event.

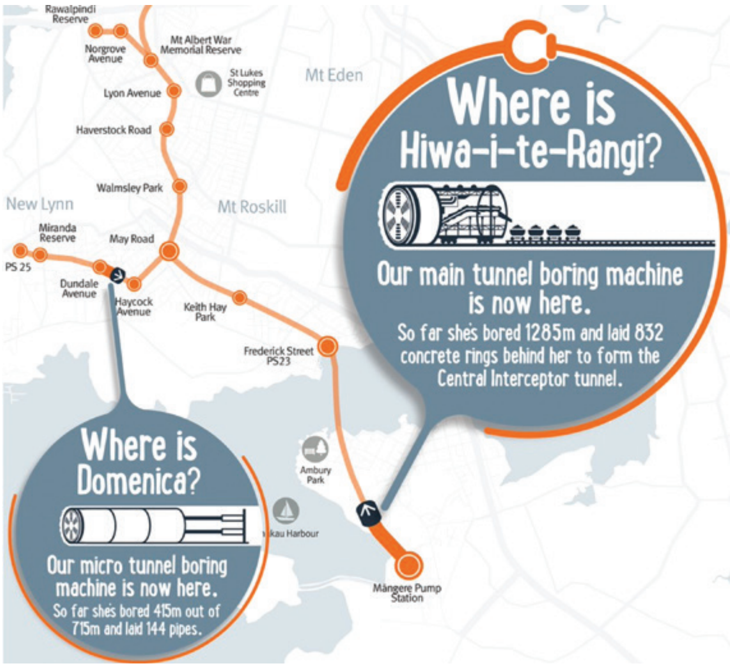
"Professor Stirling, Postdoctoral Scholar Dr Jack Williams and a team of students and colleagues this week [24 March 2022] returned from digging two 25-metre long trenches high in the Upper Nevis Basin, south of Queenstown, to gain a better understanding of the Nevis Fault, and greater Nevis-Cardrona fault system".

"The combined fault system stretches for about 100km from Lake Wanaka to near Garston in Southland, and has the potential to produce an earthquake well into the magnitude 7 range; with the new research finding evidence of at least two major prehistorical events".

"We found evidence of the sediment layers being broken up, warped and thrust over each other, which would have been produced by significant seismic events," says Professor Stirling.

Further details:
https://www.eqc.govt.nz/news/new-research-confirms-major-seismic-events-in-west-otago/

## DEEP NEW ZEALAND SEWER SHAFT COMPLETED




**GHELLA ABERGELDIE JOINT** Venture (GAJV) has completed the deepest open excavation shaft on Watercare's Central Interceptor, New Zealand's largest wastewater project.

The Central Interceptor team and its contractor GAJV have made progress on the $1.2bn(£630M) wastewater tunnel scheme in central Auckland.

The excavation crew has dug the 12.4m diameter May Road Shaft B to its final depth of 73.2m below ground level. The construction shaft to facilitate the main tunnel drive has been supported using secant bored piles, mesh, dowels and shotcrete at various depths.

According to the contractor, it reached the required depth on time and with no lost time injuries. "It is rare for engineers, geologists and operators to see how rock behaves at such depths. An achievement such as this can only be successfully undertaken with a strong team, great communication, and a positive attitude" GAJV said.

GAJV is delivering close to 20km of tunnels, 19 shafts, a pump station and wastewater management and network infrastructure works. The Central Interceptor tunnel will be 4.5m in diameter and run 14.7km from Grey Lynn under central Auckland and the Manukau harbour to Māngere.

From: Ground Engineering June 2022, digital edition https://www.geplus.co.uk/digital-edition/june-2022/

# Topic of the Day – June 1972

## We took a look through the archives of *NZ Geomechanics News* and found this article from 50 years ago. As you read it, just imagine if this was happening today.

3

## A LIGHT-HEARTED LOOK AT METRICATION

P.W. Taylor

When you go home after a tiring day and say to your wife, "I am so exhausted I could sleep for a megasecond", you will have been metricated. Maybe you don't want to be metricated, but metrication is coming whether you like it or not. (To save you working it out, a megasecond is about 11½ days.) As with all new systems of units, great advantages are claimed. It is a much more logical system than we had before; it will simplify our calculations immensely and replace a lot of out-moded illogical units. Some cynics say that it won't replace any of the existing units, it will just add some new ones to those presently in use. This is an unnecessarily pessimistic view. The old units do disappear. Take for example the Roman coin, the denarius. This is no longer with us. Even the d. has disappeared (about a year ago), to be replaced by the p. in the new British currency. Apothecaries' measures, which used to clutter the covers of school exercise books, have been disappearing. One scarcely ever hears of grains, drachms or scruples. (Who has any scruples nowadays?) The carat is still with us, but only for diamonds and gold, and these seldom turn up in soil mechanics.

There is I suspect, an element of international one-upmanship in the adoption of the 'Systeme International', (SI) by British Commonwealth countries as this will enable us to out-metricate the Continentals, where metrication first started. While French engineers may still be working in terms of kilograms per square centimetre, an old-fashioned gravitational unit, we'll be using kilonewtons per square meter, (or kilopascals, if you prefer). In these days when lunar soil mechanics form a topic for serious papers, we'll be using a system which will work perfectly well on the moon - think of the enormous advantages in that! To really go along with the SI system we must drop the confusing idea of 'weight'. We must think in terms of mass. When it comes to force, few will disagree with Newton, who said that force was mass times acceleration, so the unit of force, naturally enough called the newton, is the force exerted when unit acceleration (one meter per second per second) is applied to unit mass (one kilogram). The acceleration of gravity is about 9.8 $m/s^2$ so the force exerted by a body resting on the ground is 9.8 times its mass in kilograms, the answer being in newtons. (If of course, we are working on the moon a different factor has to be used.) How big is the newton? A rough but appropriate approximation is that an apple resting on a table exerts force of about one newton on it. In engineering practice, the newton is rather small, and the kilonewton (kN) will usually be more convenient.

In SI, the multiples and submultiples preferred are in terms of thousands, rather than tens or hundreds. With length, for example, the basic unit is the metre, so the preferred units include the kilometre (1000 m) and the millimetre (1/1000 m). The centimetre (being one hundredth of the basic unit) is not a preferred unit, and we should avoid it in laboratory work, using the mm instead.

Metric engineering drawings rarely state the units used for dimensions - they are either in metres or millimetres and it is never difficult to know which is being used. To employ centimetres would upset the system.

The most common multiples, in SI, are kilo-(k) and mega-(M) for x 1000 and x 1 000 000; milli-(m) and micro-(μ) for 1/1000 and 1/1 000 000. It is preferred, in ratio units, that only one multiple should be used, in the numerator. Taking velocity, as an example mm/s is an acceptable unit, preferable to m/ks or km/Ms which, although describing exactly the same sized unit, do not comply with the recommendation.

It seems to be agreed that we like to use units which result in numerical values in the range 0.1 to 1000. If this is done, we feel that the answers "mean something". They are numbers we can visualize, mentally. By contrast, numbers such as $10^{-9}$ or $10^{11}$ signify very little, other than "very small" or "very large". So, in SI, it is suggested that we choose units which result in values for the things we commonly measure, within the range 0.1 to 1000.

Now, let us try to apply these suggestions to an example in soil mechanics - the coefficient of permeability. A value of $2.0 \times 10^{-6}$ cm./sec. (as currently expressed) might be found from a laboratory test on a compacted silt. This unit (cm./sec.) should not be used in the new system as the centimetre is not a 'preferred' unit. If we want the answer to be between 0.1 and 1000, then the multiple which fulfils the requirements is nanometres/second (1 nanometre = $10^{-9}$ metres) and our coefficient of permeability would be written

20 nm/s

This, however, did not appeal to the British engineers who studied the matter (Ref. 1). They preferred to use the SI unit metres/second, giving, for our compacted silt, a coefficient of permeability

$2.0 \times 10^{-8}$ m/s

Presumably the thought was that a factor of $10^{-8}$ was really no worse than the $10^{-6}$ which we customarily use. Particularly for a quantity, such as permeability, which has in practice an enormously wide range, it is not always desirable to use a unit which gives a numerical value between 0.1 and 1000 as suggested.

For volume, the cubic metre ($m^3$) will be suitable for most practical problems (earthworks etc.) but it is rather large for the laboratory. The next, smaller, recommended unit is the cubic millimetre ($mm^3$) but this, even in the laboratory, is rather small! ($1m^3 = 10^9 mm^3$) The litre(l) and millilitre (ml) will, undoubtedly be used. Although a millilitre is the same as a cubic centimetre, the former name is to be preferred.

When it comes to density, we must describe it as mass per unit volume, and drop the term "unit weight". It may be desirable, for some years at least, to use the term "mass density" so that it is clear that we are not using gravitational units. Now 1000 kg may be called a megagram (Mg) or a tonne (t). While "megagram" is listed as the preferred SI unit, and "tonne" merely as an "other unit which may be used" it is the metric tonne (which differs by less than 2% from our existing "long ton") which is preferred by engineers in U.K. (Ref 1). This would lead to the tonne/cubic metre ($t/m^3$) as the logical unit for density, having the merit that the density of water (for all practical purposes) is 1.0 $t/m^3$, the same number as when using the gram/millilitre. (Let us recall, however, that g/ml, having the multiple in the bottom line will no longer be considered desirable). Despite all this, the British engineers have recommended, in their wisdom, the use of the kilogram/cubic metre ($kg/m^3$) for density. This will put all values of mass density, in soil or rock mechanics, in the range 1000-4000.

The adoption of the newton as the unit of force leads to the newton/square metre as the logical unit of stress. Many quantities, such as bearing pressure on foundations, cohesion and shear strength of soil, etc. are measured in units of stress. A special name, the pascal (Pa) has been adopted as a shorter term for the newton/square metre. For use in geomechanics the kilopascal (kPa) is a suitable multiple. The conversion from tons/sq.ft. (existing Imperial gravitational units) to kilopascals is

1 ton/sq.ft. = 107.3 kPa

For modulus of elasticity, which is also in units of stress, the megapascal (MPa) will be more suitable. In the literature which has appeared before approval of the pascal, we find $kN/m^2$ instead of kPa, and $MN/m^2$ instead of MPa. Don't let that confuse you! What might be confusing is the bar which crops up at this point. Already enshrined in the vocabulary of the meteorologist (that anticyclone with a central atmospheric pressure of 1015 millibars, for example) the bar has the advantage that 1 bar is about normal atmospheric pressure - roughly 1 ton/sq.ft. in our old-fashioned system. While it is a multiple of the SI unit (1 bar = $10^5$ pascals) it is not a preferred unit, but seems likely to remain with us for no better reason than it has "achieved wide use and forms a convenient unit for pressures". (A similar claim could be made for some of the units to be thrown out!) To cap it all, a world-wide aluminium company has announced that it will be expressing the strength etc. of aluminium alloys in hectobars - a non-preferred multiple of a non-preferred unit! At this stage one can be forgiven for entertaining sadistic thoughts about directors (and engineers) in aluminium companies. This is better than the alternative - heading for the nearest mental hospital! Fortunately, aluminium products seldom appear in geomechanics problems.

For coefficient of consolidation ($c_v$ is the usual symbol) which we now express in sq.in./sec. or sq.ft./year, the agreed best choice is $mm^2/s$.

The unit recommended for coefficient of compressibility (symbol $m_v$) is square metres per kilonewton ($m^2/kN$) which only contravenes one of the SI recommendations.

While the symbols used for various quantities will not, in general, be altered, it is necessary to use a new symbol for mass density, as it is dimensionally different from the "unit weight" or "density" in the old system. Unit weight is a measure of gravity force per unit volume, whereas mass density is a measure of mass per unit volume. To continue to use $\gamma$ as the symbol would lead to confusion. The symbol $\rho$ is used in allied disciplines for mass density, and is the obvious one to adopt in geomechanics.

Some items of punctuation are of interest. The full stop is not used to indicate an abbreviation. It is used as a decimal point or it may be used to indicate multiplication in the abbreviation for a compound unit. Torque, or bending moment, for example, might be in kilopascal metres, written kPa.m or kPa m. Commas are not used to split large numbers into groups of 3 digits - spaces are used instead; 1 234 000 for example, not 1,234,000. Units named after men are spelt with a small letter, but the abbreviations are capital letters (N, Pa etc).

Although the megasecond was mentioned at the beginning of this article, minutes and hours will, of course, still be used. Similarly, degrees, minutes and seconds will continue to be used for angles, although the radian is the preferred SI unit.

This article is intended to be an introductory outline of the use of SI units in geomechanics. The list of units which follows is not complete. Neither the formal definitions nor the conversion factors from existing to SI units are given. Ref. (2) should be consulted for such details and many more tables will appear as the new system is introduced. The following list includes the possible units for certain quantities in order of their "status", as based on the recommendations of international and national bodies.

It appears then, that provided we forget about "weight" and think in terms of "mass" and "force" - and double-check our calculations (written, of course on A4 paper) - metrication may prove to be an experience which is not too unpleasant.

7

# SI UNITS IN GEOMECHANICS

Symbols used in column headed "status" -

(a) Base unit in the SI system

(b) Derived unit in the SI system

(c) Recommended decimal multiple in the SI system

(d) Other(not recommended) multiple in the SI system

(e) Other unit which may be used

(f) Supplementary unit in the SI system

(g) Recommended by the Institution of Civil Engineers for use in design offices Ref. (1).

| Quantity | Unit | Abbrev. | Status | Conversion |
|---|---|---|---|---|
| Length | metre | m | (a) | |
| | kilometre | km | (c) | $= 10^3$m |
| | millimetre | mm | (c) | $= 10^{-3}$m |
| | micrometre | μm | (c) | $= 10^{-6}$m = (1 micron) |
| | centimetre | cm | (d) | $= 10^{-2}$m |
| Area | square metre | $m^2$ | (b) | |
| | square kilometre | $km^2$ | (c) | $= 10^6 m^2$ |
| | square millimetre | $mm^2$ | (c) | $= 10^{-6} m^2$ |
| | hectare | ha | (d) | $= 10^4 m^2$ |
| Volume | cubic metre | $m^3$ | (b) | |
| | cubic millimetre | $mm^3$ | (c) | $= 10^{-9} m^3$ |
| | cubic centimeter | $cm^3$ | (d) | $= 10^{-6} m^3$ = 1 millilitre |
| | litre | l | (e) | $= 10^{-3} m^3$ |
| | millilitre | ml | (e) | $= 10^{-6} m^3$ = 1 cubic centimetre |
| Mass | kilogram | kg | (a) | |
| | megagram | Mg | (c) | $= 10^3$kg |
| | gram | g | (c) | $= 10^{-3}$kg |
| | milligram | mg | (c) | $= 10^{-6}$kg |
| | tonne | t | (e) | = 1 Mg |
| Density | kilogram per cubic metre | $kg/m^3$ | (b)(g) | |
| | tonne per cubic metre | $t/m^3$ | (e) | $= 10^3 kg/m^3$ |
| | gram per millilitre | g/ml | (e) | $= 1\ t/m^3$ |
| Force | newton | N | (b) | $= 1$ kg $m/s^2$ |
| | kilonewton | kN | (c) | $= 10^3$N |
| | meganewton | mN | (c) | $= 10^6$N |
| Stress | (also pressure, shear strength, bearing capacity, etc.) | | | |
| | pascal | Pa | (b) | $= 1\ N/m^2$ |
| | kilopascal | kPa | (c)(g) | $= 10^3$Pa |
| | megapascal | MPa | (c) | $= 10^6$Pa |
| | " | | (g) | for modulus of elasticity |
| | bar | bar | (e) | $= 10^5\ N/m^2$ |
| | millibar | mbar | (e) | $= 10^2\ N/m^2$ |

| Quantity | Unit | Abbrev. | Status | Conversion |
|---|---|---|---|---|
| Velocity | metre per second | m/s | (b)<br>(g) | <br>for coefficient of permeability |
| Coefficient of compressibility | square metre per newton | $m^2/N$ | (b) | |
| | square metres per kilonewton | $m^2/kN$ | (g) | $= 10^{-3}\ m^2/N$ |
| Coefficient of consolidation | square metre per second | $m^2/s$ | (b) | |
| | square millimetre per second | $mm^2/s$ | (g) | $= 10^{-6}\ m^2/S$ |

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# Proposed Slope Stability Guidance Modules

## An Update

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## INTRODUCTION

**MBIE and the New Zealand Geotechnical Society have produced six Earthquake Geotechnical Engineering Guideline Modules covering a range of geotechnical topics. However, these do not specifically address slope stability. In 2019 the NZGS management committee agreed to start work on a seventh module for the assessment of slopes under seismic loading. It quickly became apparent that addressing the seismic case without first dealing with the static case would be problematic, given the varied practice currently identified in the industry.**

**This paper provides an update on the development of the proposed slope stability guidance outlined above. It provides some background as to our current challenges as an industry dealing with slope stability hazards, what the state of practice is within New Zealand, member concerns with current practices, and the NZGS management committee's proposed way forward. The majority of the text below is extracted from the Committee's Position Paper, which has been developed to act as a starting point for discussions with Stakeholders.**

## 1 BACKGROUND

Landslides and slope instability are a significant natural hazard in New Zealand, and are more common here than many countries because of our terrain and unstable geology. Residential, water and transport infrastructure in New Zealand has been developed into steeper landscapes, especially in urban areas, due to the lack of level space available. These areas are vulnerable to slope instability, either naturally inherent or created by human development and activity. These issues may be already evident under normal conditions but are expected to be exacerbated and widespread in seismic events. They will become more and more evident in the future and may be compounded by the effects of climate change (Roberts, 2020).

Schuster (1996) noted that landslides are responsible for considerably greater socioeconomic losses than is generally recognised, because they often occur as one element of multiple-hazard disasters and are reported in combination with the triggering process (e.g., earthquake, flood, hurricane, volcanic eruption, and bush fires that expose the soil). The Earthquake Commission in 2006 stated that "...more than 70% of the weather-related insurance claims are most likely related to landslips...".

## 1.1 LANDSLIDES AND SLOPE INSTABILITY IN THE NEW ZEALAND CONTEXT

The Motu Economic and Public Policy Research report in 2020 indicates that between 2000 and 2017 over 15,000 weather-related claims were settled (completed), amounting to approximately $450 million dollars. Of these, about two-thirds (about 10,000) of the pay-outs are because of land damage." The Motu report also states *".... we deduce that more than 70% of weather-related insurance claims are most likely related to landslips."*

The landslide hazard and potential consequences were distinctively demonstrated during the November 2014 Kaikoura earthquake. Over 80 landslides occurred along and immediately adjacent to the coastal transport section of the transport corridor, severely affecting State Highway 1 (SH1) and KiwiRail's Main North Line (MNL). Lengths of around 10 km to the south of Kaikoura, and 14 km to the north, were most heavily affected. Due to the time of the Kaikoura earthquake and the lack of residential development in the area most affected, there were not any fatalities as a direct result of slope instability. The toll could have been very different if the earthquake was during a busy day. However, the socio-economic cost for the region and New Zealand in general was substantial. State Highway 1 was closed for a year while the recovery costs for the transport corridor were on the order of $1.4 billion. If a residential population had been impacted by the earthquake generated landslides, significant life loss would most likely have occurred.

From a life loss perspective, over 600 deaths in New Zealand have occurred since the 1840s because of landslide-related causes (Rosser et al, 2017) – more than from earthquakes and tsunami combined. As an example, the Murchison earthquake of 1929 killed 17 people – 16 because of the landslides it generated, as evidenced by photograph 1.

**PHOTOGRAPH 1 Landslides generated by the Murchison earthquake, 1929**

Despite this, landslides have historically caused few deaths in New Zealand ***relative to the number of events that have occurred*** because there have been few settlements in mountainous terrain and the population density has been relatively low; although it is also possible that some deaths due to slope instability may have been attributed to the triggering event (for example, earthquake or rainstorm).

This is changing. As the society evolves, and as the need of space for residential development in urban areas increase, we are pushing subdivisions and development increasingly into areas that have significant landslide or slope instability potential. Together with the effects of climate change, our existing (and already high) risks from landslides and slope instability are likely to only increase in the future.

**PHOTOGRAPH 2 The Thredbo landslide, Australia, 1997a**

For overseas experience, the Thredbo landslide disaster in Australia in 1997 (photograph 2) is a characteristic example. This landslide killed 18 people and sparked 10 years of work by the Australian Geomechanics Society to improve site investigations and risk assessment processes for slope instability in Australia, resulting in issuing the AGS Landside Risk Management Guidelines.

***New Zealand has not done likewise***, even though there is no doubt that several similar situations exist here (The Priscilla Crescent landslide in Kingston, Wellington in 2014 for example).

> *"It also seems to me that the geotechnical community needs to evaluate the way it conducts its investigations to ensure that investigations whether of conditions of roads or the suitability of building sites, be undertaken having regard to the potential effect of instability on human life and the risk of loss of life or injury"*
> *– Mr Derrick Hand (Coroner to the Thredbo Inquiry)*

## 2 CURRENT NEW ZEALAND STATE OF PRACTICE

A Local Government NZ think-piece on natural hazards in 2014 stated:

> *Natural hazards and associated risks are not managed under a single statute. Rather, their effective management relies on the interplay of many statutes. Most of these statutes are enabling in nature, meaning they provide powers for agencies (mostly local government) rather than prescribing detailed requirements. Under this framework, effective management of natural hazards requires the many players exercising powers and responsibilities to do so in a coherent and co-ordinated way. The legislative picture is, however, a patchwork of laws from different eras and to some extent different philosophies and subject to different legislative purposes. The policy guidance within these statutes remains very high level and hence much is left to the discretion and judgement of those at the sharp end of implementation. Further, the integration of these statutes has not necessarily been thought out in a fully considered way. This is evidenced by (for example) the many different definitions of natural hazards included across the various statutes."*

Little has changed in the eight years since this think-piece was issued.

Since the Ministry of Works was disestablished and privatised in 1988 there is a wide-spread recognition by the geotechnical engineering industry that there has been a systematic underinvestment in geotechnical training and guidance development in New Zealand.This issue was highlighted by the Canterbury Earthquakes Royal Commission (CERC) in their 2012 report with perhaps 50 of the 189 recommendations directly or indirectly related to geotechnical issues. Many of these CERC recommendations were later addressed under the leadership of MBIE and the support of the New Zealand Geotechnical Society (NZGS) by the development of the Earthquake Geotechnical Engineering Practice Series and associated training. However, the focus of these geotechnical guidance documents was largely around earthquake design and liquefaction induced damage, although rockfall hazard mitigation and retaining wall design guidance was also prepared.

It was also apparent early in the CERC guidance project to both MBIE and NZGS in the development of the geotechnical earthquake engineering module series that there was also a need for an updated geotechnical earthquake engineering landslide guidance document that had ***a New Zealand focus***. The large earthquake induced landslides and subsequent heavy rainfall triggered landslides of slopes damaged by earthquakes has only enhanced this viewpoint.

Leaders in the geotechnical industry have noted the assessment of the static assessment of slopes is not consistently done well in New Zealand. In a similar manner to Module 6 retaining wall guidance, it is considered that any guidance document on assessing the stability of slopes under seismic loads should first address good practice in assessing static stability first, then move on to cover the assessment of the stability of slopes under seismic loads.

### 2.1 EXISTING DOCUMENTS

A literature search was undertaken on what national and international guidance exists covering the following general themes:

- planning/land-use
- static assessment of slope stability
- seismic assessment of slope stability
- emergency response
- case studies of landslides
- a variety of action plans

One type of slope instability hazard, rockfall, is a nationwide hazard to dwellings and infrastructure. Following the widespread rockfall that occurred during the 2010-2011 Canterbury Earthquake Sequence in the Port Hills, this hazard has been addressed by the Earthquake Geotechnical Engineering Practice Series. This efficiently addresses the rockfall hazard from slopes and the design of protection structures, however it covers only a small aspect of potential slope instability issues that occur in New Zealand. It is noted this document, similar to the Earthquake geotechnical module series has regulatory status by being issued by MBIE as guidance under Section 175 of the Building Act.

The most recent NZ based document addressing slope instability in general is planning focussed (in particular, Saunders et al, 2013). This document does not have regulatory status. With some exceptions, NZ based guidance is generally nearly 20 years old and is not up to date with the latest scientific and technical knowledge with respect to co-seismic assessment of stability. The literature search suggested that there are many good practice documents available, both NZ based and internationally, ranging from land use planning, investigation, model development, stability assessment risk assessment and mitigation that can form a good basis for an updated and modern guidance document.

The literature search raised the question about what an updated NZ based guidance document should cover that is not already covered in readily available national and international textbooks and guidance, and whether the production of a new slope stability guidance document would have a significant impact. The authors believe reference to a 'single source of truth' for geotechnical practitioners and consenting authorities would provide clarity around both good practice and the minimum standards required to address slope instability risks, without stifling innovation. It would improve consistency and prevent confusion caused by information spread across different sources and documents, as well as ensure that the latest science and knowledge is incorporated through regular maintenance and update of this single source of truth. Such guidance would also bring in New Zealand specific advice and knowledge to ensure suitability for our regulatory regime and geology.

## 3 CURRENT INDUSTRY PRACTICE

There are many examples of excellent – even world leading – practice in New Zealand. Our flexible regulatory regime and many international engineers and geologists means we have a strong skills base and we can be innovative in our approach. However, not all practice is at this level. Many slope stability assessments, particularly at the more basic level, lack technical rigour and are sometimes significantly flawed in ways that are not immediately apparent to consenting authorities. The causes of this inconsistent performance may span from the appropriate education and training of engineers, to ongoing professional development, and the availability of adequate industry standards to ensure a consistent approach to the problem and subsequently relatively uniform results.

It is generally recognised that recent geo-professional graduates need a substantial amount of post graduate training and mentoring in order to be effective in the private sector. Some large consultants have developed formalised systems to achieve training and development of their technical staff.

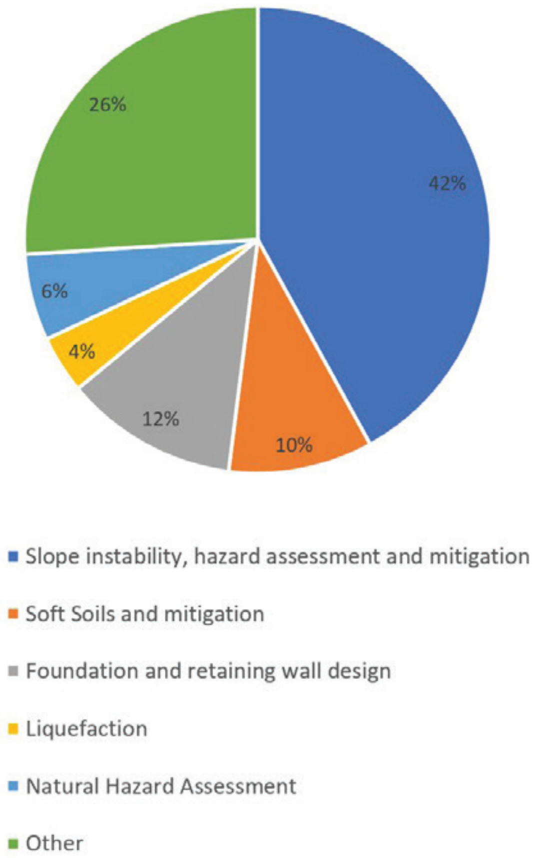
For generalist civil/structural engineers this has caused problems when it comes to recognising the warning signs of unstable terrain. In order to address this, some Councils have put restrictions on which geospecialists and engineers can submit consent applications in higher hazard terrain e.g., Tauranga City Council.

In addition, the CPEng (Geotechnical) and PEngGeol Body of Knowledge (BOKS) documents prepared by NZGS have been developed to outline a consensus view of senior practitioners of the skill considered appropriate to work on higher risk geotechnical projects including slope stability assessment.

In 2019, NZGS sent a survey to its members requesting their feedback on a wider scope document. This survey identified several concerns about slope stability practice in New Zealand that could be resolved with more clear guidance. A few key messages arose from the feedback to this survey, as indicated from responses received to the following three questions (10 questions were asked in total).

**Q1. Which topics in the geotechnical engineering arena do you feel cause most problems in NZ?**

Out of 50 responses received, 42% of respondents concluded that slope stability caused them the most concern

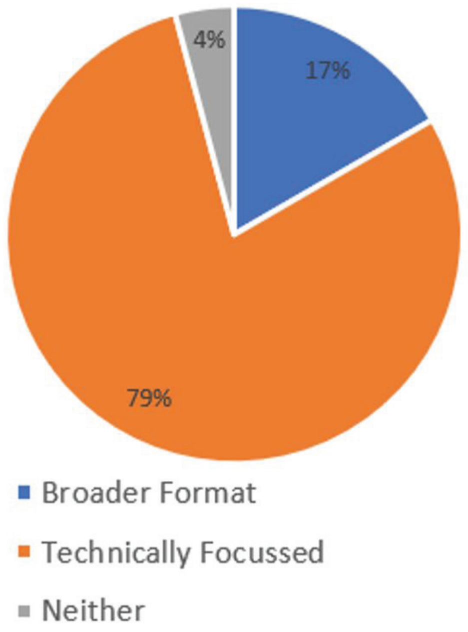



**Q2. Can you describe examples of practices observed in New Zealand related to slope engineering (either seismic or static) that did not give a good result?** (Selected responses)

- ❑ Yes; poor attention to geological conditions, assuming material parameters and blindly modelling.
- ❑ Yes - Inadequate understanding of groundwater conditions (or what causes failures)
- ❑ Yes. I've seen single slopes analysed by different firms/ People all giving considerably different results due to different training/ references.
- ❑ Often the geological and groundwater interpretation is poor. The method of analysis can often be less than optimum or even wrong.

In summary, the responses received showed a general lack of consistency and in some cases a lack of understanding and/or lack of appropriate engineering geological input exists within the geotechnical community in New Zealand.

**Q3. Would a guidance document on slope stability be more useful in the technically focused format of the earthquake geotechnical engineering modules, or in the broader format of the 'Planning and engineering guidance for potentially liquefaction prone land'?**




## 4 WAY FORWARD

The NZGS committee believes that development of a slope stability guidance document for geo-professionals is now essential. It is proposed to develop the guidance as a series of Modules or Units, which replicate the structure used in the Earthquake Geotechnical Engineering Guideline Practice Series.

### 4.1 PROPOSED MODULE 1

#### 4.1.1 Objectives

The purpose of the proposed first project is to produce an overarching Module (Module 1), defining the general principles under which other documents will sit, and to scope subsequent modules to provide more detailed guidance on specific topics that are identified as high priority. Where appropriate, the additional modules will reference existing documents (for example, sections of the AGS Landslide Risk Management Guidelines, or the planned update to the Saunders and Glassey (2007) Guidelines for assessing planning policy and consent requirements for landslide prone land.

The proposed guidance is intended to set a reliable baseline for good practice for different slope instability problems, and to ensure that improvements are implemented into the guidelines after reasonable consultation with the NZGS members.

#### 4.1.2 Stakeholder Engagement

We have had discussions with MBIE and EQC, who have indicated their qualified support for development of the proposed Landslide Guidance Module 1 although MBIE have also indicated it is not their current priority. We will continue to lobby MBIE on this issue as having some form of regulatory support, similar to the Earthquake module series would be the ideal. With a level of support now secured, the committee is starting and continuing discussions with stakeholders with a view to secure funding for development of the Module. Following this, we will need the expertise of the NZGS members to develop and review the Module as it evolves.

NZGS will also give thought to complementary activities such as training as associated with the guidance, promotion of the new landslide data base, pushing to increasing availability to valuable data sets such as the LINZ Section 72-74 land hazard notice dataset and EQC land claim data set.

The recent landslide prediction models developed by GNS with government funding support are also a valuable resource and relevant to this discussion.

The NZGS committee is determined to make this a success. So please watch this space!

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www.SLIDENZ.net

# How did we get here?

## A biased view of the development of engineering geology in New Zealand

Don Macfarlane

**Don Macfarlane**

### BACKGROUND

Engineering geology is the application of geology to engineering for the purpose of ensuring that the geological factors, including hazards, affecting a site or scheme are recognized and taken into consideration during the design, construction, and maintenance of the engineering works.

Unfortunately, in the past misunderstandings frequently created friction between the engineers and geologists. The problem was that all too often the geologists did not have any real understanding of engineering principles and were prone to a lofty, scientific style of report or oral presentation that was foreign (and therefore nearly useless) to the engineer.

In extreme instances, some project engineers minimized the importance of adequate geological investigations or scorned the data and recommendations of geologists as of little practical value. In my own experience, this was exemplified by a quote pinned to an (un-named) engineers office door during the construction of Clyde dam which advised us that the only information required from the geologists was the answer to the question "is it rock of ages, or is it shit?"

Sadly, some of the resulting engineering works ultimately experienced difficulties and/or failure due to not understanding the geological conditions. Well known examples include the St Francis Dam (1928), Baldwin Hills reservoir (1963), the Vaiont landslide (1963), along with the Malpasset (1959) and Teton (1976) dam failures. However, lessons were learned from these experiences and, as expressed by Eggers (2016)[1], *today a mutual interdisciplinary respect exists for the importance of mature, field-experienced geological input for engineering purposes*. Or, more simply, engineers now see the value of appropriately trained and experienced (engineering) geologists, and use them!

1 Eggers, M.J. (2016). Diversity in the science and practice of engineering geology. Geological Society, London, Engineering Geology Special Publications, 27, 1-18, 21 September 2016. https://doi.org/10.1144/EGSP27.1

## A POTTED HISTORY OF ENGINEERING GEOLOGY

Several detailed chronologies of the history of engineering geology history are available in the literature. For example, a detailed historical record of engineering geology beginning with William Smith was prepared by Galster (2004)[2]. Smith (1769-1839) is often considered to be the 'Father of Engineering Geology' (at least by the English) because not only did he contribute to the development of the modern geological map, but he was also involved in a range of engineering work in connection with mines, canals, irrigation and coastal defences through which he observed, documented and realised the significance of understanding the geology, and being able to predict it, for safe and efficient construction.

Arguably, the broad concepts of geology have been known and used in construction for thousands of years: engineering geology has (unknowingly) been an important scientific sub-discipline for as long as people have sought to construct their living environment. People soon learnt which caves were safe, where buildings could be built safely and where poor foundation conditions or the presence of geohazards meant that unacceptable risks were present. However, it was due to the great growth of geology as a major science in the early 19th century that the specific science and practice of engineering geology developed.

The first ever book entitled 'Engineering Geology' (which was published in England in 1880) was written by one William Henry Penning[3]. The Introduction in Penning's book begins with the sentence: *"In the execution of engineering works, however scientific in design and clever in workmanship, failure has frequently usurped the place of success because due attention has not been paid to geological phenomena".*

During the twentieth century the attitude of engineers toward the importance and relevance of geological input into engineering works changed dramatically. Eggers (2016) suggests that the modern development of engineering geology occurred over the period from the 1920s to the 1960s, helped along by lessons learnt from a number of catastrophic engineering accidents (such as those listed above) that showed engineers around the world that the safety and economy of engineering projects required geologists with an appreciation of engineering principles, not just a classical education in geology. But Penning saw this way back In 1880 when he said *"…it is not possible, nor even desirable, for all professional engineers to become proficient geologists… but they may, and with advantage, avail themselves of the labours of others.."*

The development of 'modern' engineering geology was very much driven by large national building projects after the First and Second World Wars. Examples of major projects that promoted the development of engineering geology include the Hoover dam in the USA, constructed during the 1930s, the Snowy Mountains hydropower and irrigation scheme built between 1949 and 1972 in Australia, and the Waitaki, Waikato, Tongariro, Manapouri and Clutha hydropower schemes built between the 1930s and 1980s here in New Zealand. Standards and codes were developed, mostly during the 1960s–1980s, particularly for material descriptions and investigation methods.

Also during that period, particularly in the 1950s and 1960s, courses on geology were introduced into civil engineering undergraduate programmes in many countries and in some universities engineering subjects were incorporated into geology degrees. From this engineering geology emerged as a new discipline, culminating in the development of specialist courses in Engineering Geology in the 1960's and 1970's. Many of New Zealand's senior or recently retired engineering geologists were trained in such courses in the UK, Australia or New Zealand. Others were trained in South Africa, the USA or Europe.

## DEVELOPMENT OF ENGINEERING GEOLOGY IN NEW ZEALAND

The rapid economic expansions of the post-World War II years were the catalyst for engineering geology to become a cohesive profession (Williams 2016)[4]. The development of engineering geology in New Zealand prior to about 1970 was driven by major construction projects, particularly the hydro power stations, and was based largely on very good observations by geologists with no relevant training but experience in the geology of the site areas.

### WAIKATO RIVER DAMS

The development of the Waikato Valley Hydro Electric Power Programme, which was carried out between 1929 and 1970, resulted in the construction of 8 dams and 9 power stations between Karapiro and Lake Taupo. The geology of the dam sites was determined by regional geologists from NZ Geological Survey and limited construction observation reports were prepared based on rare site visits. Most of these reports were prepared by Jim Healy (1940's and 1950's) or Bruce Thompson (from the mid 1950's). The only original investigations/construction geological records for Arapuni were authored/co-authored by J Henderson (later Director of NZ Geological Survey) between 1920 and 1930. None of these people had any engineering background – both Jim Healy and Bruce Thompson were primarily volcanologists.

2 Galster, R. W. (2004). The Origins and Growth of Engineering Geology and its Professional Associations. Association of Engineering Geology Special Publication 19, CD-ROM, P. O. Box 460518, Denver, Colorado.

3 Penning originally trained as an engineer then joined the (British) Geological Survey in 1867

4 Williams, J. (2016). Engineering Geology – Definitions and Historical Development Applications in Life Support Systems. http://www.aegstl.org/wp-content/uploads/2016/10/APPLIED-GEOLOGY-5000-YRS-EOLSS-A.pdf

## WAITAKI RIVER DAMS

Following investigations in the late 1920's, involving Pat Marshall (who wrote a report on the later-developed Aviemore site), Waitaki dam was constructed between 1932 and 1938, with extensions to the powerhouse in the early 1950's. The only known geological information from that site was a mention in a regional mapping paper by Marwick (1935) and reference made to a report by Henderson in 1930. Marwick was a paleontologist; Henderson was primarily a mining geologist.

The Lake Tekapo Control Structure (1940) and the Tekapo A power station (commissioned 1947) were built either side of WW2 and the original Pukaki dam (since flooded) was commissioned in 1951. Ian McKellar and Horace Fyfe did a lot of the 1950s investigations for the Upper and Mid Waitaki projects. Benmore was constructed 1958 – 1963 and Aviemore in the period 1964 - 1968. None of these sites had a resident geologist during investigations or construction. Although occasional visits were made during construction by geologists from Christchurch, in particular Les Oborn and Graham Mansergh (for Benmore and Aviemore), there was little systematic recording of the ground conditions. Les was a graduate of the Otago School of Mines, Graham was a geologist with a special interest in Quaternary geology.

## CLUTHA RIVER DAMS

Roxburgh dam was constructed between 1949 and 1956, and Hawea was constructed from 1956 to 1958. Bryce Wood and Ian McKellar (regional geologists from the Dunedin office of the NZ Geological Survey) made periodic site visits and reported their observations to the Ministry of Works but again there was no full time geologist on site capturing a complete record of the ground conditions at either site.

## MANAPOURI POWER SCHEME

The original Manapouri scheme (completed between 1963 and 1971) brought a change of attitude to the value of on-site geologists, probably largely driven by the experience of the overseas contractor (Bechtel Pacific Corporation) and the complexity of the scheme. Kiwi engineering geologists associated with the Manapouri scheme were Royden Thomson who worked on the underground excavations (late 1960's) and Graeme Halliday who worked on the Te Anau and Manapouri lake control structures from 1973 to 1976. Royden (a graduate of the Otago School of Mines) was employed by the contractor, Graeme was a Ministry of Works employee with a geology degree (who started on site as a chainman). Both of them were subsequently involved with the Clutha Valley Development project[5].

## TONGARIRO POWER SCHEME

TPD represented the first major commitment to engineering geology by the Ministry of Works and really represents the true 'birth' of engineering geology in New Zealand. This scheme, constructed between 1964 and 1983 involved a complex of dams, canals, tunnels and power stations, both surface and underground. By this time NZ Geological Survey had established (in 1965) its Engineering Geology Section led by Les Oborn[6].

The first engineering geologists to provide full-time on site support in a secondment to TPD were Warwick Prebble and John Dow in the late 1960's. They were later joined (or replaced) by Bernard Hegan, Graham Hancox, Brian Paterson, Terry Grammer, Chris Gulliver and (in 1977) Dick Beetham. None of them had specifically trained as engineering geologists; John Dow and Chris Gulliver had engineering degrees that included geology, while the young fella (Dick Beetham) had degrees in both engineering and geology.

## UPPER WAITAKI POWER DEVELOPMENT

Les Oborn and Graham Mansergh provided geological support from Christchurch for the initial investigations for the Upper Waitaki Power Development during the 1960's. The first full time engineering geologist on UWPD was Stuart Read from March 1972 through until September 1976. He was joined by an itinerant Scotsman, Jim McLean, who was there from 1975 until January 1977 when Don Macfarlane joined the team straight out of the University of NSW. Don remained on site until late 1981. All three of these site geologists had engineering geology degrees.

## CLUTHA VALLEY DEVELOPMENT

Royden Thomson joined the Geological Survey and was seconded to CVD in 1974, joined by Graeme Halliday (still with Ministry of Works) in 1976 and then Graham Salt (an engineer rather than a geologist) in 1978. Don Macfarlane transferred to CVD in late 1981 and the team was strengthened by the addition of Dick Beetham in the 1983. Jim McLean returned for a time also, working on Clyde dam, and was subsequently replaced by Mark Foley when he moved to Christchurch.

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5 The Second Manapouri Tailrace Tunnel constructed between 1997 and 2001 to increase the efficiency and output of the power station involved a new generation of engineering geologists.

6 Leslie Eric Oborn (1920-2011) was Chief Engineering Geologist from 1965 until his retirement. Les studied at the Otago School of Mines 1947 to 1950 and then began work at NZ Geological Survey in Christchurch where he became involved in (among other things) investigations for the Benmore dam. This led to an ongoing interest in geology for engineering projects. In 1964, Les transferred to Lower Hutt where he established the Engineering Geology Section of NZ Geological Survey and convinced the Ministry of Works and NZ Electricity Department of the value of engineering geologists to major projects.

**ABOVE** Engineering Geology Section, NZ Geological Survey (at Turangi, 1977)
**BACK ROW:** (L to R) Don Macfarlane, Graham Hancox, Bruce Riddolls, Graham Salt, Dick Beetham.
**FRONT ROW:** Nick Perrin, Royden Thomson, Les Oborn (Chief Engineering Geologist), Bernard Hegan, Brian Paterson. Absent – Stuart Read, Ian Brown (both overseas).

### MANIOTOTO COMBINED POWER AND IRRIGATION SCHEME

Engineering geological support for this late 1970's to early 1980's scheme involving canals and a tunnel was provided by Brian Paterson and Jeff Bryant with some input from Royden Thomson. Kelvin Moody completed his Masters thesis on batter stability in schist along the canal.

## ENGINEERING GEOLOGY SECTION IN LOWER HUTT

As noted above, the NZ Geological Survey (now GNS Science) established an Engineering Geology Section under Les Oborn in about 1965. At the time there were few (if any) specialist engineering geologists in the country. It was Les who convinced the Ministry of Works and NZ Electricity Department of the value of engineering geology to major projects and (in my mind) can justifiably be called the "father of engineering geology in New Zealand".

In the early 1970's the team in Lower Hutt comprised Les Oborn, Nick Perrin and Bruce Riddolls, joined in mid 1970s by Graham Hancox on transfer from TPD and Ian Brown after a period providing full time support for the Auckland Rapid Transit investigations. Stuart Read relocated to Lower Hutt in 1978 after a stint travelling and working offshore but the team otherwise remained complete and unchanged until at least 1980. This group worked on a range of (mostly Government) engineering and research projects all around New Zealand, including providing additional support to those out on the big projects.

## TRAINING OF ENGINEERING GEOLOGISTS

Few of the early engineering geologists in NZ had training in engineering geology, but they understood the relevance of geology to engineering and were able to communicate the important issues to the engineers better than a 'classical' geologist who commonly spoke a completely different language. This ability to communicate the importance of the geological factors was the key to the successful development of engineering geology as a separate profession in New Zealand (and worldwide).

According to Rogers (2002)[7], the American Engineering Council for Professional Development pushed for the inclusion of engineering geology in undergraduate civil engineering courses from the 1950s so that by the mid 1970s nearly 80% of courses in the US included a compulsory engineering geology module. In the UK, Australia, New Zealand and many other countries, degree courses in Engineering Geology were first established in the late 1960's and 1970's, usually led by geologists with practical experience on large projects, most often dams and/or tunnels.

By the mid-1960s it had become obvious that there was a need for the local training of professionals who could provide quality geological input to engineering projects and work alongside engineers. At that time geologists at both Auckland and Canterbury universities had taught a paper 'Geology for Engineers' within the professional BE degree structure for several years but it was clear that more advanced and practical courses were needed.

In the mid 1960's, Canterbury University appointed John Hill an Australian engineering geologist who had worked on dam construction projects in NSW (including Bendora dam, ACT) in the early 1960's. He taught (some) geology to the civil engineering students and established an engineering geology course as part of the BSc (Hons) course: the first two graduates were Stuart Read and Ian Brown in 1971. The BSc (Hons) students did a range of classical geology papers and "Special Topics" (such as Rock Mechanics) that introduced them to engineering geology in their Honours year[8].

David Bell (another Australian) joined the department in late 1972 and took over teaching the engineers about geology, then developed the hugely successful MSc programme in the late 1970's.

Similarly, in the early 1970's a Senior Lectureship in Engineering Geology was established by the University of Auckland and Warwick Prebble was appointed in 1975. He held a BSc in Geology from Victoria University and had worked for six years as combined regional geologist for the Geological Survey (DSIR) and project geologist for the Tongariro power project before moving to Auckland in 1971 to join Beca-Carter.

7 Rogers, J.D. (2002). Disappearing practice opportunities: why are owners and engineers taking increased risks? What can be done to counter this threat? In: Proceedings of a Symposium "Visioning the future of engineering geology: stewardship and sustainability," 26 September 2002, Reno, Nevada at the Joint Annual Meeting of the Association of Engineering Geologists (AEG) and the American Institute of Professional Geologists (AIPG). Special Publication 14 (on CD-ROM). Association of Engineering Geologists, Denver, Colorado, 14p

8 The original students were required to gain entry into Engineering school to participate in the soil mechanics courses

So, between the two universities, NZ was producing a small number but increasing of graduates with specialised engineering geology training during the 1970's. Some of the Canterbury University engineering geology graduates of the 1970's were Jeff Bryant, Guy Grocott and Grant Borrie, while over this period Auckland University graduates with some training in engineering geology included Simon Carryer, Wayne Russell, and Jarg Pettinga.

Also in the 1970's Michael Selby was teaching geomorphology at Waikato University with a few students each year. Graduates in this period included Nick Rogers, Kevin Hind and David Burns. In the 1980's Selby established an undergraduate engineering geology paper with a landform-based approach within the Earth Science degree, and a graduate paper more focussed on rock and soil mechanics principles in relation to slopes.

## OUTSIDE OF NZ GEOLOGICAL SURVEY

It has not been easy to identify people who worked as geologists and/or engineering geologists in other organisations before 1980 and it is probable that this account has missed some, several or most of them!

From 1924 until his retirement in 1940 Pat Marshall was employed as geologist and petrologist by the New Zealand Public Works Department in Wellington. His work was centred on geological problems associated with engineering projects[9]. This probably makes him the first full time engineering geologist in New Zealand.

W.N. Benson (Professor of Geology at Otago University 1917 to 1949) worked within the Dunedin area, and undertook studies that involved investigating new road- and rail-cuts and commercial and domestic excavations. His studies of mass movement, especially in urban areas and coastal areas north of the city, led to statements relating engineering hazards to particular local rock types.

In the late 1960's and through the 1970's the development of Engineering Geology in NZ was rather ad hoc and largely centered around the Geological Survey geologists seconded to the large infrastructure projects. But times were changing... in the universities and in the consultancies.

Tonkin & Taylor was established as a civil engineering testing business in 1959. Both Don Taylor and Ralph Tonkin were engineers. T+T's first engineering geologist was Chris Gulliver (with both a BE and a BSc) in 1970, followed by Nick Rogers (with an Earth Science degree) who joined in the mid -70's. Chris left T+T to join the team at TPD in 1975 but re-joined them in 1978. As noted above, Warwick Prebble moved to Beca from TPD in 1971 before joining the University.

Graham Cookson was a geologist with the Ministry of Works and Development in Dunedin from around 1969 to 1982, mainly supporting roading projects, while after completing his MSc in Jeff Bryant worked at the MWD Central Laboratory 1975-79 testing soils, soft rocks and real rocks. MWD employed Julie McMinn, who had a Diploma in Engineering Geology, in Dunedin in the early 1980's.

In Wellington Tony Mahoney was a geologist with Brickell Moss Rankine and Hill through the 1970s, with particular expertise on assessing the stability of slopes and foundations (that continued into the 1990s).

David Burns embarked on his career in engineering geology (in about 1980) as a soils technician with Worleys. He held a Masters in Earth Science (from Waikato) and was another example of a geologist who learned engineering geology on the job.

Bob McKelvey came to engineering geology via an NZCE (early 1970's) and a Geology degree (late 1970's) and it is likely that others followed a similar path.

## WHERE TO AFTER 1980?

The creation of good training courses, ongoing development projects and increasing awareness of natural hazards and environmental issues led to a rapid growth in the numbers of engineering geologists in NZ from the mid-1980s onwards.

From 1980, the Canterbury MSc in Engineering Geology usually had 5 to 10 students per year – and the University also taught the BSc (Hons) course and offered a postgraduate Diploma in Engineering Geology. Graduates in the early to mid-1980's included Gary Smith, Tim Browne and Mark Foley.

In the early 1980's Bernard Hegan joined the exodus from the NZ Geological Survey to grow the engineering geology expertise of T+T, Ian Brown formed his own company, Bruce Riddolls joined Worleys to establish an engineering geology team, after which he formed his own company. Mark Yetton joined Soils & Foundations in Christchurch (in 1983) and other consultancies retained some of the newly trained engineering geologists emerging from the universities, while others went overseas for experience.

Doug Johnson and Debbie Fellows both worked at CVD as summer students in the early-mid 1980's, and Kelvin Moody spent summers with Jeff Bryant who was then the site geologist on the new highway through the Cromwell Gorge.

The Clyde landslides stabilisation project (1989-1992) saw the largest single concentration of engineering geologists on any single NZ project up until that time, with the on site team peaking at 28 (including summer students Mark McKenzie, Richard Justice and Virginia Cunningham). Graduates on the site included David Barrell, Tim Coote, David Stewart, Paul Horrey and Linda Price.

By the late 1980's our universities were training more engineering geologists than we needed so many of the graduates sought overseas experience. Over the

9 https://teara.govt.nz/en/biographies/3m44/marshall-patrick

intervening years engineering geologists within NZ have become spread across many organisations, including CRIs, regional councils and consultancies. Some of those who went overseas have returned but others have built successful careers elsewhere while overseas-trained engineering geologists are now fairly common in NZ. It is a massive change from the period before 1980!

## FINAL REMARKS

Remember where we came from!

Penning, way back in 1880, first coined the term 'engineering geology'. He had the understanding to realise that *".. the geological conditions which affect engineering and similar works are, mainly, the extent of the various strata, their lithological character, and their order of succession. It matters not what may have been the forms of Life during the ages when the strata were deposited…"*.

It took both the civil engineering and geology professions a long time to realise the truth of this statement, but it happened – and we can thank Penning for setting the career path we have chosen!

**Footnote:**
Penning's 1880 book can be downloaded (free) from https://ia800203.us.archive.org/15/items/engineeringgeolo00pennrich/engineeringgeolo00pennrich.pdf


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All errors and omissions are the responsibility of the author!

# Assessment of Current Practices Around the Use of the Standard Penetration Test

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**KEYWORDS:** SPT, Standard Penetration Test, soil-rock interface, site investigation, guidelines.

## ABSTRACT

**An issue faced by young geotechnical professionals is when to terminate the Standard Penetration Tests (SPT) when advancing a borehole during a geotechnical investigation. This issue commonly arises in the soil-rock interface zone where the ground mass is transitioning from a soil to a rock. Commonly in New Zealand, a cut-off figure of SPT 'N' value >50 is used. However, depending on the end use (what will be constructed), some clients (and colleagues in different disciplines, e.g. structural engineers) want higher 'N' readings to provide stronger evidence of a full transition to rock strength. Some weathering profiles include hard inclusions (core-stones) and recovery in such horizons can often be poor. This paper examines the theories and realities behind what is actually being measured. A soil mass behaves differently from a rock mass, and as such the strength of each is measured and characterised in a different way, applying different criteria. This paper sets out approaches developed to apply SPT results to different situations, drawing examples from both soil strata and very weak rock units. The paper concludes with some simple guidelines that have been found helpful in developing engineering judgement with regard to SPT use in the soil-rock interface zone.**

## 1 INTRODUCTION

Young geotechnical professionals (0-5 yrs) are often sent out to do geotechnical ground investigations, with little or no understanding of how a Standard Penetration Test (SPT) works, or what it actually measures. Without this grounding in the theory, it is difficult to make a decision about the termination of the SPT on site. The SPT is primarily a test for soil and soft rock (UCS 0-20 MPa), and as such the termination of the test is usually in the vicinity of the soil-rock interface. This interface is highly variable in different environments and is primarily a factor of weathering and rock type. Weathering affects the depth and sharpness of the interface, and different rock types can weather in different ways, under a similar climatic regime (Fooks, Pettifer, & Waltham, 2015). The variety in site conditions and engineering requirements means that there can be no universal criteria for the termination of the SPT.

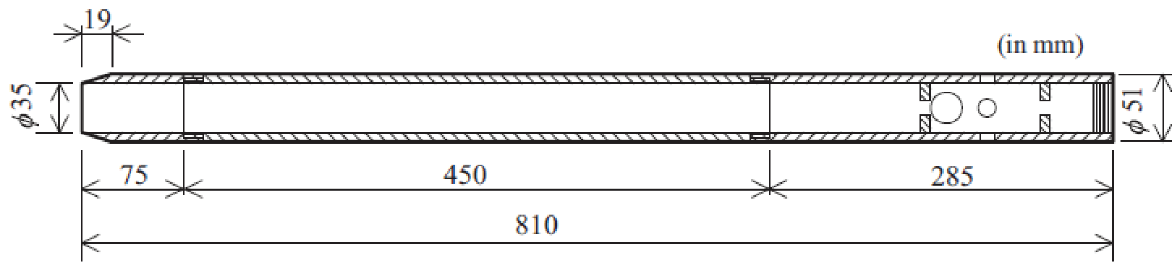


**FIGURE 1:** SPT split spoon sampler (Matsumoto, Phan, Oshima, & Shimono, 2015). Dimensions given in mm

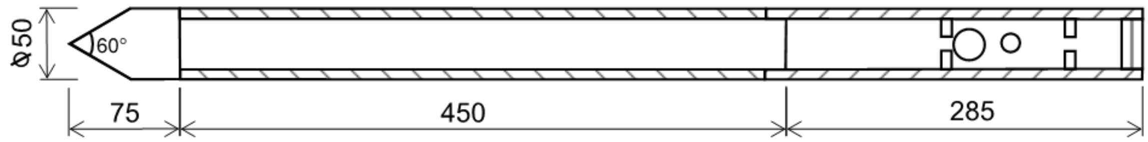


**FIGURE 2:** Solid cone SPT as stipulated in the NZS 4402.6.5.1:1988. Dimensions given in mm, modified from (Matsumoto, Phan, Oshima, & Shimono, 2015)

This paper presents a brief outline of the theory behind the SPT and how it works. Various correlations and corrections have been made over the years to standardise the penetration test and allow comparison between different soil types and measuring apparatus. The importance of understanding the difference between soil and rock is explored and how the strength of each is ascertained for design. This paper aims to provide some simple guidelines that have been found helpful in developing engineering judgement with regard to SPT use in the soil-rock interface zone for a variety of rock types.

## 2 THE STANDARD PENETRATION TEST

The Standard Penetration Test (SPT) is an in-situ dynamic penetration test (Clayton, 1995) that measures the resistance of soils to a split spoon sampler (Figure 1) or solid cone (Figure 2). The SPT is an easy way of gaining additional information from geotechnical investigations at very little additional cost. Because of this advantage the SPT, while reasonably unsophisticated, is likely to remain in use for some time.

In New Zealand the test is conducted in situ down hole by driving a split spoon sampler or solid cone into the ground in increments of 75 mm for a total of 450 mm using a 63.5 kg weight, falling from a height of 760 mm. The test is divided into two parts, the seating drive is the initial 150 mm of the test and the test drive is the remaining 300 mm. The blows recorded for the seating drive are not included in the reported result (N), which is the combined number of blows recorded for the test drive.

The raw N-value or $N_{SPT}$, is the uncorrected blow count and is subject to several influencing factors, including method and equipment used (Clayton, 1995). It is common practice in New Zealand to use the automated method of hammer release, which reduces much of the variability. However, in order to be correlated with any other test, including the equivalent SPT N given in CPT results, the N-value must be converted into $N_{60}$ (Look, 2019). Every rig or apparatus used for undertaking the SPT should have a current calibration certificate which allows the correction to be calculated.

Skempton (1986), published an equation that allowed for the use of hammers of varying efficiencies and corrected for varying field procedures and apparatus.

$$N_{60} = \frac{E_M C_B C_S C_R N}{0.60} \quad (1)$$

Where $N_{60}$ is the corrected SPT N-value, $E_M$ is the hammer efficiency, $C_B$ is the borehole diameter correction, $C_S$ is the sample barrel correction, $C_R$ is the rod length correction, and N is the raw SPT N-value recorded in the field. The corrected N-value is given as $N_{60}$ because the original SPT apparatus had a hammer efficiency of 60%, and all SPTs are referred back to this original test to allow a comparison between results.

In 1986 Liao and Whitman published a correction for the overburden pressure which may influence the SPT result.

$$(N_1)_{60} = N_{60} \frac{\sqrt{2,000}\ psf}{\sigma'_z} \quad (2)$$

In this correction the overburden pressure is removed from the $N_{60}$, where $\sigma'_z$ is the vertical effective stress from the depth of the test.

The solid cone was developed for use in gravelly soil in an attempt to overcome the effect of gravels larger than the mouth of the split spoon sampler. The solid cone is less likely to be damaged by the gravels and is more able to go through or around the gravel clasts. The solid cone is deemed inappropriate to use in anything other than a gravel as the increased surface area can result in up to 100% increase in the penetration resistance (Clayton, 1995). Despite this, it remains common practice in New Zealand to switch to the solid cone when the ground begins to reach SPT N values above 40. The SPT is now one of the most widely used in-situ tests for gauging the density and compressibility of granular soils and is also used to check the consistency of stiff or stony cohesive soils and weak rock (Clayton, 1995).

## 3 SOIL MASS PROPERTIES

The engineering behaviours of soil are governed by many factors including effective stress, true cohesion, and internal friction (Skempton & Bishop, 1950). Soils can be broadly divided into two basic groups, granular and cohesive. The SPT procedure is more reliable in sands than in cohesive soils (Peck, Hanson, & Thornburn, 1953), because silts and clays exhibit different driving resistance when dry or moist (Rodgers, 2006). The strength of the ground is difficult to predict because the soil moisture can change, thus changing the strength of the ground.

### GRANULAR SOILS

The behaviour of granular soils is controlled by the effective angle of friction which according to Clayton (1995) is a function of:

- stress level,
- grain size distribution,
- angularity,
- void ratio (relative density)
- Cementation - Strength can be increased by particle interlocking and/or cementation.
- Ageing – density and interlocking tend to increase with age, and also penetration resistance above and beyond that expected as a result of the density increase.

Even in granular soils, the rate of testing means that excess pore water pressure will be generated and the conditions will be somewhere between ‘drained’ and ‘undrained’ (Clayton, 1995). The action of penetration leads to shearing, dilation and collapse.

### COHESIVE SOILS

The penetration resistance in cohesive soils is largely a function of undrained shear strength (Clayton, 1995) The relationship between the SPT-N value and the undrained shear strength is influenced by a number of factors including (according to Clayton 1995):

- Plasticity
- Sensitivity – SPT values are a combination of side friction and end bearing resistance. The end bearing resistance is undisturbed and represents the undrained shear strength, where the side friction is remoulded. Sensitive soils therefore have a lower N-value due to the lower side friction.
- Fissuring (fabric) – If the fissuring aligns with the direction of penetration, then the soil will offer less resistance.

## 4 ROCK MASS PROPERTIES

The strength of the rock mass is governed by the intact rock strength and by the density and condition of discontinuities. Intact rock strength can be measured with uniaxial compressive tests, and point load tests can be correlated with geology specific test data to gain an unconfined compressive strength. The estimation of the strength and deformation properties of jointed rock masses however is a more difficult and somewhat empirical exercise. Rock mass classification schemes are one such way of predicting the strength and deformation properties of rock masses. This is achieved by degrading the intact rock properties to rock mass properties (Deisman, Khajeh, & Chalaturnyk, 2013). These include but are not limited to, the RQD (Deere & Deere, 1988) the Q system (Barton, Lien, & Lunde, 1974), the RMR (Bieniawski, 1989) and the GSI (Hoek & Brown, 1997).

SPTs are often carried out in weak and weathered rock. The reported N value is affected by fracturing and weathering of the rock and is not a measure of the intact rock strength (Clayton, 1995). Although calculations have been developed to relate the N-value to UCS, it has been pointed out that the N-value varies with the geology (Look, 2004; Look, 2019). This is largely due to the rock mass fabric including jointing (Look, 2004).

The majority of Auckland, New Zealand, is constructed on a suite of soft Tertiary rocks named the East Coast Bays Formation (ECBF). These rocks exhibit soil-like properties throughout the weathering profile, and an empirical relationship is being developed to classify the weathering of the rock mass, where visual clues are difficult to discern. This classification does not use SPT to directly measure weathering, rather, it is inferred through the general strength of the ground mass. It is important to note that while the ECBF is a flysch deposit, it has never been ‘hard’ rock, can be uncemented, and is much weaker (e.g.(Hodgson & St George, 2008)) than other, older sandstones elsewhere in the world (e.g., Triassic Hawkesbury Sandstone, Sydney Basin (Pells, 2004). In weak and weathered rock, many factors influence the SPT and as such, the interpreted results will be uncertain and imprecise (Clayton, 1995).

## 5 SPTS IN THE SOIL-ROCK INTERFACE

As with all robust geotechnical investigation work, a preliminary geological model should be established prior to undertaking site work. SPT N-values cannot define the soil-rock interface alone and must be put into the context of the logged geology. It is important to differentiate if the ground is behaving in a soil-like or rock-like way and make the interpretation of the SPT based on that information. The following sections provide examples of the soil-rock interface in the common rock types found in Auckland.

## BASALT (LAVA FLOWS)

Basalt lava flows in Auckland can come in a variety of configurations and it is important to consider these when interpreting the SPT results. For example, an SPT of N=50+ has different design implications if it is located in the rubbly margin of a flow, or if it is in solid basalt. It is also useful to remember that lava flows can have soft material underneath or between flows (i.e. 'stiff over soft' conditions), and this material may need to be tested too. The weathering profile is often quite sharp, and the ground transitions from soil to rock rapidly. The rock mass can be highly fractured, or can be fully intact, with the intact rock strength usually in the vicinity of 50–100 MPa. As a general rule, SPTs are not undertaken in basalt. This is because conducting an SPT in strong rock damages the equipment and no relevant data is obtained. The rapid transition to competent rock can mean that there is a possibility of not reaching N=50+ at all before terminating the test. The SPT can be terminated if competent rock is identified in the previous run or if the hammer double bounces. Deeply weathered lava flows may leave core stones, so it is important to check the next run or two to confirm the rock head.

## ECBF (SOFT TERTIARY SANDSTONES AND MUDSTONES)

As stated above, ECBF is a soft rock that underlies much of Auckland. The weathering profile is not usually deep and fracture density varies, though is typically low. The unweathered intact rock itself reaches strengths of >20 MPa at its strongest and is a function of the degree of cementation. Caution needs to be exercised when interpreting the N-value in relation to rock strength because the density of the ground does not necessarily translate to the strength of the material. An example of this is the uncemented sandstone which can be found at depths well below the weathering profile. These unweathered sandstones often have an N-value of 50+ but a UCS of <1MPa. SPT refusal is usually around the moderately weathered to slightly weathered interface, and indeed is often used to classify that very transition, where the behaviour becomes rock-like rather than soil-like.

## NORTHLAND ALLOCHTHON (SANDSTONE/ MUDSTONE MELANGE)

The Northland Allochthon is a very general term for a suite of rocks classified as a melange resulting from a submarine landslide or thrust sheet (Winkler, 2003). The Northland Allochthon is contemporaneous with the earlier parts of the ECBF and is not a very strong rock due to its young age and material composition. In these rocks even if an N-value of 50+ is reached 3 times, it can be useful to keep going. There have been examples where 3 x N=50+ was reached in one borehole, yet the adjacent borehole never achieved three in a row. This is due to the highly heterogeneous nature of the rock unit which includes concretions that can reach up to 2 m diameter. It is important to bear in mind the purpose of the investigation and consider whether SPTs will assist or not in determining ground conditions. For example, if the purpose of the investigation is to identify a landslide slip plane, then SPTs would be inappropriate as the crucial information could be obscured by the test.

### GREYWACKE (INDURATED SANDSTONE AND MUDSTONE)

Mesozoic-age Greywacke in New Zealand is highly indurated, usually highly fractured and faulted, and commonly veined with quartz. These rock mass and structural defects lead to an undulating and highly variable weathering profile. It is common to find core stones within this material, typically close to the soil-rock interface. Unlike the ECBF, the soil-rock interface in Greywacke is usually at the completely weathered to highly weathered boundary. It is important to note that even in completely weathered greywacke relict rock mass fabric can still control failure modes, even though the strength and behaviour is soil-like. An SPT in moderately weathered greywacke will show that the rock mass is fractured and that the intact rock strength is weaker than unweathered greywacke but will not quantify the degree to which this has occurred. Degree of fracturing and strength loss are better seen in core that has not had an SPT attempted in it. When terminating the SPT, satisfy yourself that you have reached rock head and not a core stone. If there is moderately weathered rock at the base of the run, do not undertake the test in the following run.

## 6 GUIDELINES FOR TERMINATING THE SPT

In the New Zealand standard NZS4402.6.5.1-1988, it is specified that the sampler be driven until it penetrates 450 mm into the ground or for a total of 60 blows, whichever comes first. The current rule of thumb is to terminate SPTs after 3 successive readings of N=50+, whether in the seating drive or the test drive. While this practice is convenient and easy to remember, it is not always the most appropriate course of action, and engineering judgement must be exercised. As demonstrated in the examples above, there are situations where fewer than three or greater than three N=50+ are required. Figure 3 is a breakdown of the questions to consider when terminating the SPT.

The SPT cannot be used as a predictive model to determine the weathering grade, rather the SPT needs to be interpreted in the context of the geology. It is vitally important to get an accurate understanding of the local geology, because characteristics other than strength are important. In an Auckland case study the rock head was set at SPT N=50+, which is adequate for pile design. However, the geology was misinterpreted, and the foundations were set in an aquifer. This had major implications for the project, that could have been mitigated if the geology had been correctly identified.

## 7 CONCLUSION

The standard penetration test has been in use since the 1920's and is one of the most common in-situ geotechnical tests. Despite this it must be used with caution and within the same scale of accuracy as the test itself. Of fundamental importance is the conversion of N to $N_{60}$. Without this conversion the SPT results cannot be compared to other N-values from other tests.

Soil and rock exhibit different engineering behaviour, and different strengths. The SPT in granular soils can

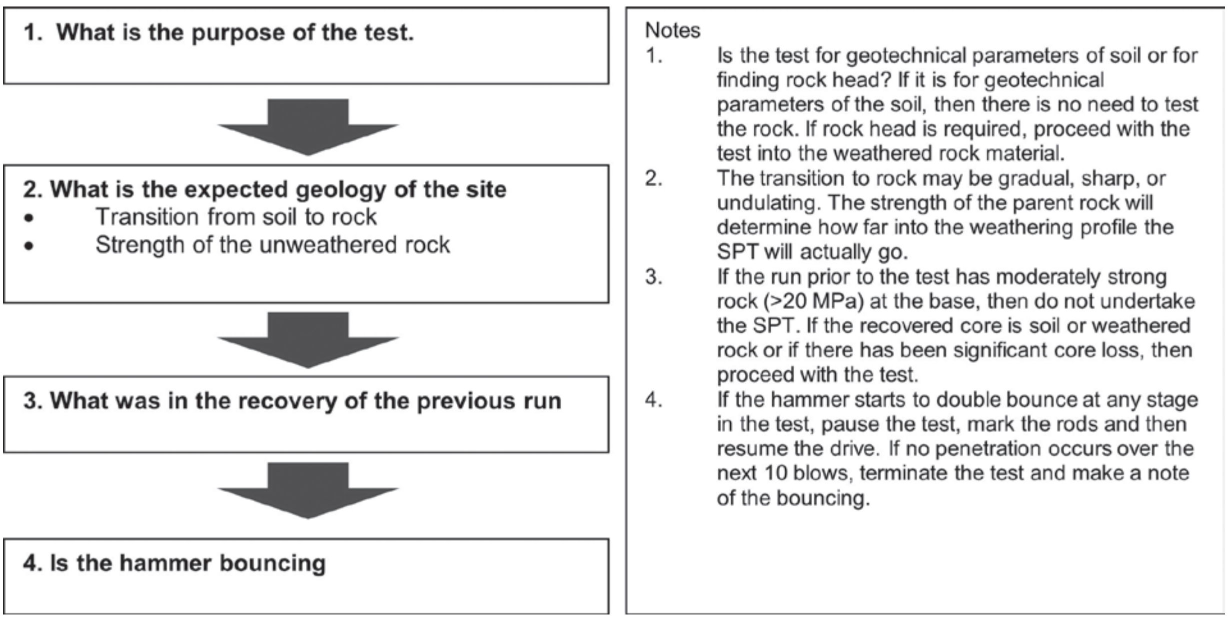


**FIGURE 3:** Considerations when terminating the SPT.

be related with a degree of confidence to a number of empirical equations. With cohesive soil and weak rock, the SPT is more of an indicator of consistency. Examples from Auckland demonstrate the complexity of the soil-rock interface and how the SPT can be interpreted in different situations. A breakdown of the steps to follow and questions to consider when terminating the SPT have been presented. These may prove useful to the young geotechnical professional embarking on ground investigations.

Finally, the SPT must be interpreted in the context of the local geology and not be relied on independently of other contributing factors.

## 8 ACKNOWLEDGEMENTS

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PEACOCKES BRIDGE SLOPE STABILISATION


TRANSMISSION GULLY - WAINUI SADDLE

# Quantifying the pumice content of soil through utilisation of the unique characteristics of pumice

Callum Sands

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**KEYWORDS:** Pumice, Light Weight, Crushable, PSD, Particle Breakage, Hardin, Relative Breakage

## ABSTRACT

**The geotechnical behaviour of natural pumiceous soil deposits is highly dependent on the percentage of highly crushable pumice particles to hard-grained sands, within the soils matrix. There is currently no standardised method to quantify the pumice content of a given soil sample. The research and laboratory testing outlined in this paper was initiated with the purpose of forming a standardised methodology to do just that. This paper explores one method by which the amount of pumice in naturally occurring pumiceous sand deposits can be estimated. The method makes use of the high crushability of pumice grains, compared to hard-grained sands, in correlation with some particle breakage indices based on particle size distributions (PSD) of a soil sample prior to and after testing. In order to standardise the crushing of the soil sample, a modified version of the standard proctor test outlined in NZS 4402.4 was adopted. Extensive testing was undertaken to determine a compaction energy that would result in significant crushing of pumice grains, with minimal crushing of hard-grained sand particles. Testing concluded that a compaction energy of 900 KJ/m$^3$ was optimal for the final methodology. To determine if there is a correlation between some particle breakage indices and the pumice content of a soil, soil samples were prepared by mixing Mercer River sands and pumice sands. The samples ranged from 0 percent pumice to approximately 100 percent pumice, stepping up in intervals of 25 percent. Multiple samples at each pumice percentage were sieved, tested and then sieved again to build a range of data. From the data collected, an analysis was conducted to determine which breakage index would yield the most reliable relationship. The results of the analysis showed that Relative Breakage ($B_R$), theorised by Hardin (1985), was most appropriate for the final methodology.**

## 1 INTRODUCTION

Pumice is a volcanic material that forms during volcanic events. During the initial eruption, magma erupts at such great speeds that it forms a molten froth. As this froth travels through the air it is rapidly cooled. During this cooling gasses get trapped in the froth, forming vesicles, or pores. Due to this rapid cooling, the atoms in the material are unable to form into a crystalline structure.

Pumiceous deposits in the central part of the North Island, New Zealand are generally encountered in river valleys and flood plains; these areas typically coincide with large areas of development and human inhabitants (Orense et al. 2012). These widespread pumiceous soil deposits around the Volcanic Plateau are the product of massive volcanic eruptions occurring in the Taupō Volcanic Zone (TVZ), shown in Figure 1 below. The TVZ is responsible for a majority of New Zeeland's volcanic soil deposits. The zone extends from White Island to Ruapehu and is considered extremely active. Volcanic soils have been deposited over approximately one-sixth of the North Island during the last 1.5 million years by events in this zone. Particularly large amounts of hot ash and pumiceous materials have been deposited by huge eruptions of the Taupō, Okataina and Rotorua volcanoes, over the last few thousand years. During these events, pumiceous soils were initially deposited over the land via airborne transport, and were later transported many kilometres away via soil erosion and river deposit (Kikkawa et al. 2012; Wesley 2001).

Cone Penetrometer Testing (CPT) is a common test that is used to estimate the engineering properties of soil. Testing involves pushing a cone through the soil and measuring it's tip resistance, pore water pressure, and sleeve friction. From these measured data, other engineering properties of the soil are estimated, such as the liquefaction resistance, i.e. cyclic resistance ratio (CRR). Correlations made between CPT data and soil characteristics have largely been modelled from tests involving more common soils, such as non-crushable silica sand deposits. During CPT testing the shear stress transfer from the cone to soil can result in crushing of pumice grains. This localised crushing results in the formation of finer grained material, of which the mechanical properties can vary greatly from the in-situ material. Therefore, the conventional relationship of tip resistance, relative density, and cyclic resistance do not apply to pumiceous soils (Orense et al. 2012). In addition, laboratory testing indicates that the response of natural pumiceous sands is dependent on the pumice sands present in the soil matrix.

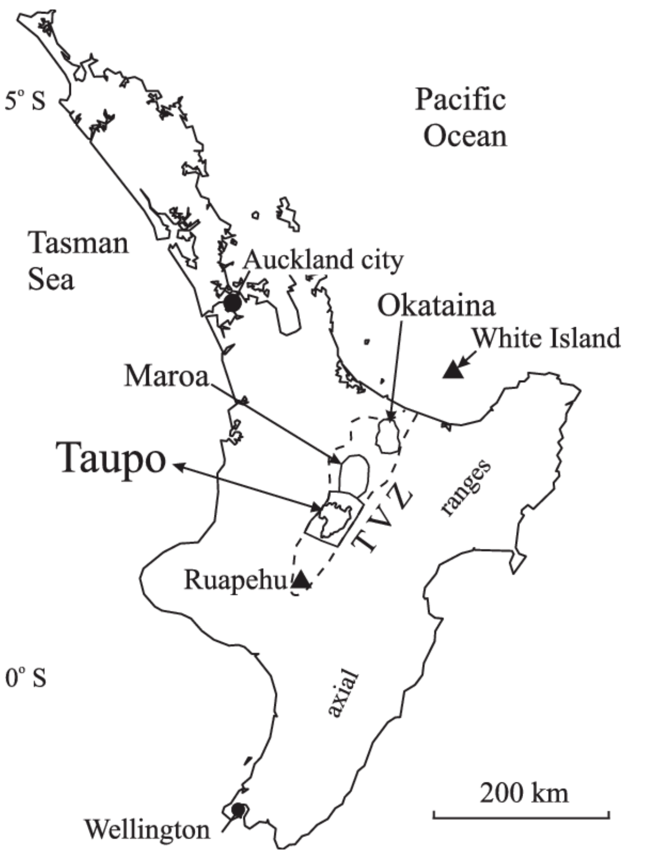


**FIGURE 1:** Taupō Volcanic Zone (TVZ) extent (Blake et al. 2006)

## 2 LITERATURE REVIEW

### 2.1 ENGINEERING PROPERTIES OF PUMICEOUS SOILS

Pumice is typically characterised as lightweight and crushable. These defining characteristics are due to the porous nature of the pumice. Each individual pumice particle has an inter-connected network of external voids/vesicles, as well as internal vesicles. The individual particles of pumice are easily crushed between one's fingers. The high crushability results from two factors: the vesicular and angular structure of the particles. Pumice sands are typically 75% by weight of silica sands. The structure of individual pumice grains can be seen in Figure 2 below.

Wesley (2001) has led investigations into quantifying the specific gravity of pumice sands, and the effects of particle size. He carried out two different methods

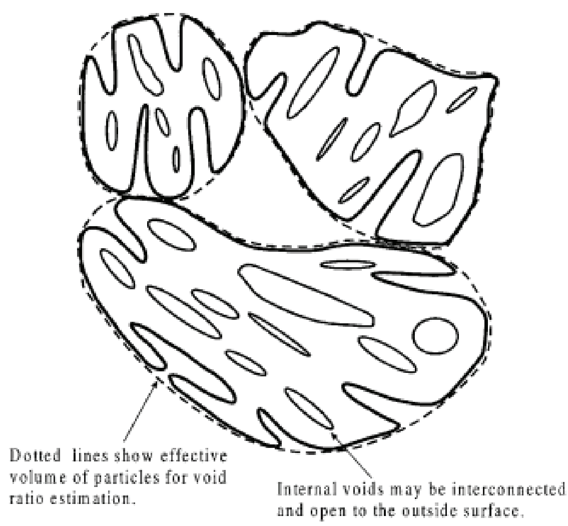


**FIGURE 2:** Schematic representation of pumice particle (Wesley, 2001)a

to determine the soil's specific gravity: (1) using simple displacement methods, and (2) putting the soil under vacuum. He noted a substantial difference in specific gravity results between the two tests for all particle sizes. This likely arises due to the water not penetrating the external voids of the pumice particles when not under vacuum. He observed that even when the particle size was as small as 0.04 mm, there were still internal voids of which water could not penetrate. The average specific gravity from his testing came out at 1.77, which is considerably lower than that of quartz sand at 2.65. He found that pumice particles ranging from 0.0063 mm to 2.0 mm have a relative density of approximately 890 kg/$m^3$, with a max dry density of 955 kg/$m^3$ (Wesley 2001).

### 2.2 RESPONSE OF PUMICEOUS SOILS DURING SEISMIC EVENT

During the 1987 Edgecumbe earthquake, widespread liquefaction was observed. Liquefaction resulted in vertical settlements, lateral spreading and sand boils across the Rangitaika Plains. The Rangitaika Plains soil profile typically consists of alternating layers of pumice-derived alluvial sand and gravel, with interbedded greywacke gravel, tephra, marine silt, and sand. The groundwater in this area is generally around 3 m below the surface. At some of the sand boil sites, the ejected soil consisted of stratified, fine to medium-grained sands with pumice lapilli of up to 20 mm in diameter (Pender and Robertson 1987).

### 2.3 SOIL COMPACTION AND CRUSHING

In the New Zealand Standard NZS 4402.4, there are two compaction tests outlined: the Proctor Test and the Modified Proctor Test. Both tests involve layering a soil sample into an apparatus of a given dimension, to a standard number of layers; followed by a hammer weight dropped from a standard height, weight, and a number of blows. This results in the compaction of the soil.

It was found in research by Luxford (1975) that these tests are not suitable for a cohesion-less material, such as sand or coarse-graded stones. This is due to the inherent angular stability of these materials. Further adding to this, Arcement and Wright (2001) found that compaction of fine soft sands would undergo crushing, whereas weathered silica sands could withstand the test.

### 2.4 SOIL BREAKAGE POTENTIAL INDICES

Soil breakage refers to the physical degradation of a soil particle or grain, through applied force. The deformation response of soil is highly dependent on the soil's ability to withstand high loads, and not undergo significant particle breakage (Hardin 1985). The degree of particle breakage is dependent upon the stress path, time and magnitude of loading. More particles will crush when a large load is applied over an extended period. However, the rate of crushing will gradually lessen. This is due to the grading of the soil particles becoming finer and less crushable under loading. Furthermore, the structure of the particle influences the breakage potential. Pumice particles have an angular, porous structure that results in high crushability, whereas hard grained quartz and silica sands have a round structure and do not crush easily. Simply, the greater the relative density, the less the potential for crushing (Lade et al. 1996).

The degree of particle breakage can be defined by comparing the particle size distribution (PSD) curves before and after loading. The PSD of soil shows the proportion of variable particle sizes contained in a soil sample. Several geotechnical engineers have theorised what is known as a 'particle breakage index'. In all theories, the index makes use of a particle size distribution curve; where the PSD is taken both before and after the crushing of a soil sample. Most of the breakage indices, such as $B_p$ (Marsal 1967), $B_{10}$ (Lade et al. 1996) and $B_{15}$ (Lee and Farhoomand 1967), indicate breakage through a singular index based on PSDs before and after crushing. However, relative breakage ($B_R$) by Hardin (1985) looks at the entire PSD curve (Ghanbari et al. 2013).

Marsal (1967) based his breakage measure around the percent of soil retained in each sieve before and after crushing. The difference in percentage retained is calculated at each sieve size and for every positive percentage change the differences are summed, giving the breakage index $B_p$. Marsal's index has a theoretical lower and upper limit of 0 and 100%. Lee and Farhoomand (1967) proposed a similar measure, where their particle breakage potential index is expressed in terms of the change in particle diameter, specifically the 15% finer curve before and after crushing. This is the ratio of $D_{15i}$ to $D_{15f}$, where $D_{15i}$ represents the diameter for which 15% of the sample was finer initially, and $D_{15f}$ represents the diameter of 15% finer after crushing. Lade et al. (1996) later came up with a similar index, using the percentage change at the 10% passing sieve after crushing.

Comparative to these indices, Hardin (1985) proposes a more global index, looking at the entire PSD curve and the potential for particle breakage to a defined state. He defined three parameters, the first being the Breakage Potential ($B_p$), the area between the initial distribution curve and the number 200 sieve. Breakage potential is representative of the total change in gradation when every single particle is crushed down from its initial grain size to a size lesser than the number 200 sieve. Secondly, he defined the total breakage ($B_T$) as the area between the curve before and after crushing. Finally, he states that the relative breakage ($B_R$) is the ratio of total breakage to potential breakage (Hardin, 1985).

$$B_R = B_T \div B_P \qquad (1)$$

The lower limit of his relative breakage is 0, and the theoretical upper limit is 1, representing the zero crushing and full crushing of all particles, respectively. Potential particle breakage ($B_p$) and total breakage ($B_T$), as described by Hardin (1985).

## 3 TESTING METHODOLOGY

### 3.1 SOILS USED AND TESTING PROCEDURE

The soils used in testing were combinations of commercially available Mercer River sands and pure pumice grains. The pumice used in testing has been sourced from the Waikato River and separated from other minerals using a centrifuge. Five sample combinations were used for testing, ranging from 0% pumice to 100% pumice, increasing at increments of 25%. The division of each sample was based on the soil weights, rather than volume. This was done due to the difference in the relative density of the hard-grained, quartz sands and the pumice. All samples used for testing are that of sand grain sizes, ranging from 0.063 mm to 2 mm.

### 3.2 TESTING PROCEDURE

The testing methodology was designed to find a trend between the particle breakage of a given soil sample, and the percentage of pumice within the sample. Particle breakage of a soil sample was achieved following the proctor compaction test procedures, outlined in the New Zealand Standards for soil testing NZS4402. Due to having a limited supply of pumice particles for testing the smaller, Japanese Proctor mould, sized in accordance with JIS A 1210:2009 was used. This mould was adopted as it has a smaller volume than the standard NZS4402 mould and would subsequently use less soil. This method of crushing was adopted to control the amount of particle breakage, through changing the compaction energy of testing. The compaction energy can be controlled by varying the height of the drop and number of drops of the hammer, using the standard 2.5 kg proctor hammer. To quantify particle breakage, the most reliable breakage index was determined based on the PSD curves before and after soil crushing. The PSD curve of a soil sample is determined following the procedure outlined in NZS4402:2.8, method for dry sieving. Wet sieving was not adopted due to time constraints for testing.

## 4 RESULTS

### 4.1 COMPACTION ENERGY

The final testing method was chosen based on a compaction energy that would result in minimal crushing of the sand, and significant crushing of the pumice. Multiple tests were conducted on pure pumice and sand samples to find the optimal compaction energy. Compaction energy, per unit volume, was calculated using drop height (*H*), hammer weight (*W*), number of layers ($N_l$), number of blows ($N_b$), and the volume of the sample (*V*), shown in equation (2).

$$Compaction\ Energy\ (Ec) = (H \times W \times N_b \times N_l) \div V \tag{2}$$

It was found that for compaction energy of 900 KJ/m$^3$ there was some sand crushing, however, the crushing was negligible in comparison to that of the pumice grains. This compaction energy is considered 'optimal' for the testing methodology of this paper, with the amount of sand particle breakage considered inconsequential. This is shown in Figure 3, where it is seen that as the compaction energy increases, the relative breakage of pure pumice increases at a significantly higher rate than that of the silica sand the optimal compaction energy was achieved using a 2.5 kg hammer from a drop height of 305 mm with 3 layers of soil with 3 blows of the hammer.

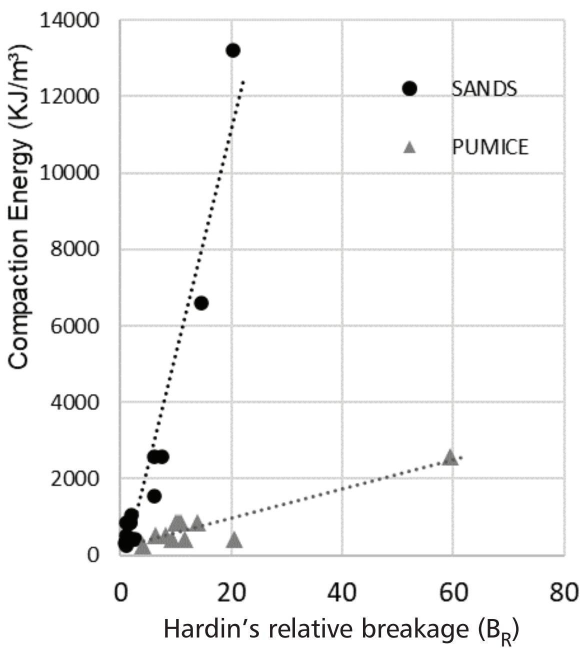


**FIGURE 3:** Hardin's relative breakage with varying compaction energy

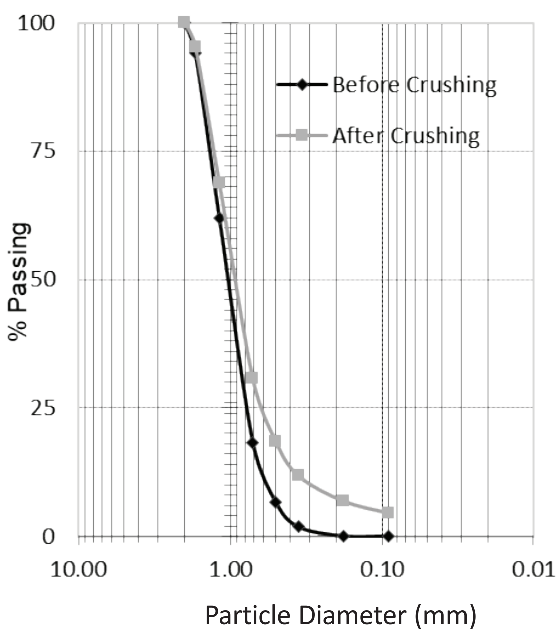


**FIGURE 4:** Particle size distribution curve for 100% pumice sample

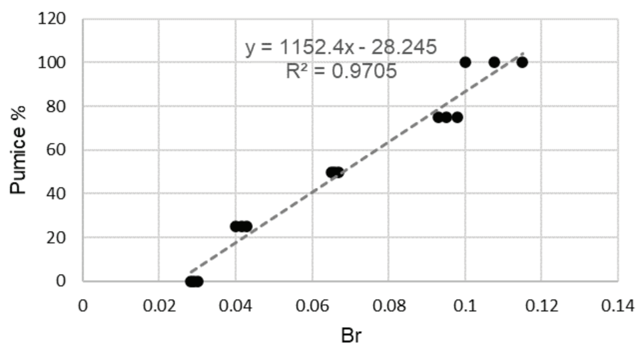


**FIGURE 5: Hardin's relative breakage ($B_R$) compared with pumice content, performed by independent parties**

## 4.2 PARTICLE BREAKAGE INDEX

Once the final testing methodology had been determined, testing was conducted on samples of known pumice contents. As expected, when using the optimal compaction energy there was significant crushing of the pumice grains relative to the crushing of the hard-grained sand particles. To visualise the degree of particle breakage, PSD curves were used. Figure 4 above shows the change in PSD of a soil sample consisting of 100% pumice grains.

## 5 DISCUSSION

Using the 'optimal' compaction energy, physical degradation of pumice particles relative to hard-grained sands was visualised on the PSD curves from before and after crushing. Of the several breakage indices investigated, only Hardin's $B_R$ was deemed accurate and reliable for quantifying a relationship of pumice content and particle breakage. The relationship of $B_R$ to pumice content, as percent by weight, can be seen in Figure 5. A total of 15 soil samples, three at each specified pumice content, were tested using the final methodology to produce this trend. Tests at each percentage were performed by independent parties in order to negate any user bias. It can be observed from Figure 4 that samples of 100 percent Mercer sand experienced some crushing. However, this is minor relative to the crushing of 100 percent pumice, resulting in insignificant change to the overall trend.

It was observed during testing that the change in particle distribution after crushing was minor, as shown in Figure 4 above. As a result, the breakage indices that focus on singular point changes of the PSD plot did not yield strong relationships. These indices, comprising Lee and Farhoomand (1967) and Lade et al. (1996), do not consider the entire PSD curve, hence problems can arise when a soil with poor PSD is tested. The effect of poor PSD grading can result in significant crushing of the larger particles with insignificant crushing of the smaller particles. Therefore, when using both the Lee and Farhoomand (1967) or Lade (1996) index, there is the potential to ignore a region of considerable particle breakage. These regions arise from the large particles having a higher potential to crush comparative to smaller particles (Hardin 1985).

Another breakage index that returned a strong relationship between pumice content and particle breakage was Marsal's (1967), breakage potential($B_p$). This refers to the breakage potential of the soil and is the sum of all positive changes in percentage passing at each sieve (Marsal, 1967). Although this index produced a strong trend, due to the dependency on particle distribution and sieve size, it was not chosen for the final methodology. Therefore, Hardin's $B_R$ was chosen for the particle breakage analysis. By using this index, it provides higher accuracy when conducting the test with sieve sizes that differ from those used during experimental testing. Furthermore, variation in results due to samples varying in size, grain size and distribution will be eliminated, as $B_R$ is independent of the PSD (Hardin, 1985).

Hardin defines $B_R$ upon the assumption that after some period of loading all soil particles will be crushed to the point there is 100% passing the number 200 sieve. In a more recent study by the Einav (2007), he postulates that Hardin's theory of 100% crushing is inaccurate. He proposed that the particles would not undergo full crushing, but rather would crush until a certain point and after this would not degrade any further, no matter the extent of force applied. This is due to the cushioning effect of surrounding smaller particles and the lack of concentrated stress from force dispersion amongst the smaller particles. Through his research, he was able to define an equation for a boundary line to which all particles would crush. This boundary is comparable to the PSD profile of the pre-crushed soil and is defined as the ultimate breakage ($F_u$). Similar to Hardin's method, using this boundary line one could determine the relative particle breakage through the ratio of total breakage and breakage potential (Einav, 2007).

## 6 CONCLUSIONS AND RESEARCH RECOMMENDATIONS

There are widespread pumiceous soil deposits through the central north island of New Zealand. These soils are problematic for engineers due to the difficulty in accurately measuring the in-situ mechanical properties of the soil. Particularly the cyclic resistance ratio (CRR) derived from data collected during CPT testing. This in-accuracy is due to the high crushability of the pumice particles within these soils.

Through research and laboratory testing, a reliable methodology was made. The methodology accurately determines the pumice content, as percent by weight, of naturally occurring pumiceous deposits comprising hard-grained sands and pumice particles.

Testing showed that the presented method could be completed in a reasonable timeframe, with standard geotechnical laboratory equipment and minimal training. A high degree of repeatability was observed during testing.

Pumice sands were proven to be highly crushable, comparative to hard-grained, silica sands. By crushing samples of pure silica sand and pure pumice sands, an 'optimal' compaction energy of 900 KJ/$m^3$ was determined. This energy resulted in significant crushing of pumice particles, with only minor crushing of sand particles. The degree of sand crushing was proven to have minimal effect on the overall accuracy of the testing methodology.

Hardin's relative breakage index ($B_R$) proved to yield a strong relationship to the percentage of pumice in a given soil sample. Hardin's $B_R$ was deemed to be the most reliable over others investigated due to the independency from the PSD of the soil.

The relative breakage theory proposed by Einav (2007) appears to have higher theoretical practicality over Hardin's index. Further research should be carried out into this theory and its application to the methodology proposed in this paper.

It was observed during testing that some of the finer soil particles following crushing would get stuck in the sieves. This may cause some irregularities in the test results, as a PSD curve is very sensitive to weight change in the lower sieve sizes. Wet sieving would likely alleviate any indiscretions.

## 8 ACKNOWLEDGEMENTS

I would like to thank A/Prof Rolando Orense for his guidance and enthusiasm over the duration of this project. A special mention should be made for the late Dean Botica, who was a great leader and mentor, imparting a great deal of knowledge and experience in the short time he worked with the author. He had encouraged the author to further his research and pursue the YGPC 2020.

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# A Multifaceted Approach to Mapping in Northland, New Zealand

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**KEYWORDS** geohazard, mapping, Northland, survey, GIS, geology

## ABSTRACT

**A multifaceted desktop and field based geotechnical survey and geohazard mapping project was completed for over 25,000 ha of mixed rural and urban land identified for potential future urban growth and development within the Kaipara District in Northland, New Zealand. The project included desktop reconnaissance, field mapping, interpretation and presentation of data.**

**An initial high-level desktop study was completed to identify areas of focus within each of the eight "Indicative Growth Areas" (IGAs). The desktop study utilised multiple methods to develop preliminary hazard maps and identify focus areas within the IGAs.**

**Once the focus areas had been identified, the field mapping program was carried out to ground-truth the preliminary desktop maps, and to identify geomorphological features and geotechnical hazard potential. The field mapping program utilised land-based mapping and photography, as well as drone photography and videography where possible.**

**The data collected from the desktop and field studies was then used to determine risk ratings for various hazards within the IGAs. ArcGIS was then used to produce various layers which were collated to generate combined geohazard maps for IGA.**

## 1. INTRODUCTION

The importance of robust desktop reconnaissance tools prior to embarking on a field mapping exercise for large areas of land was illustrated during a mapping project for over 25,000 ha of mixed rural and urban zoned land identified for potential future urban growth within the Kaipara District (KDC) in Northland. The project commenced with a high-level desktop study identifying focus points across eight "Indicative Growth Areas" (IGAs) in the region, which were defined by Kaipara District Council.

Stereoscopic aerial photographs, published geology maps, historical aerial photographs, borehole records from the New Zealand Geotechnical Database, Google Earth and in-house GIS were all used during the desktop study to identify focus areas within the IGAs. The field investigation

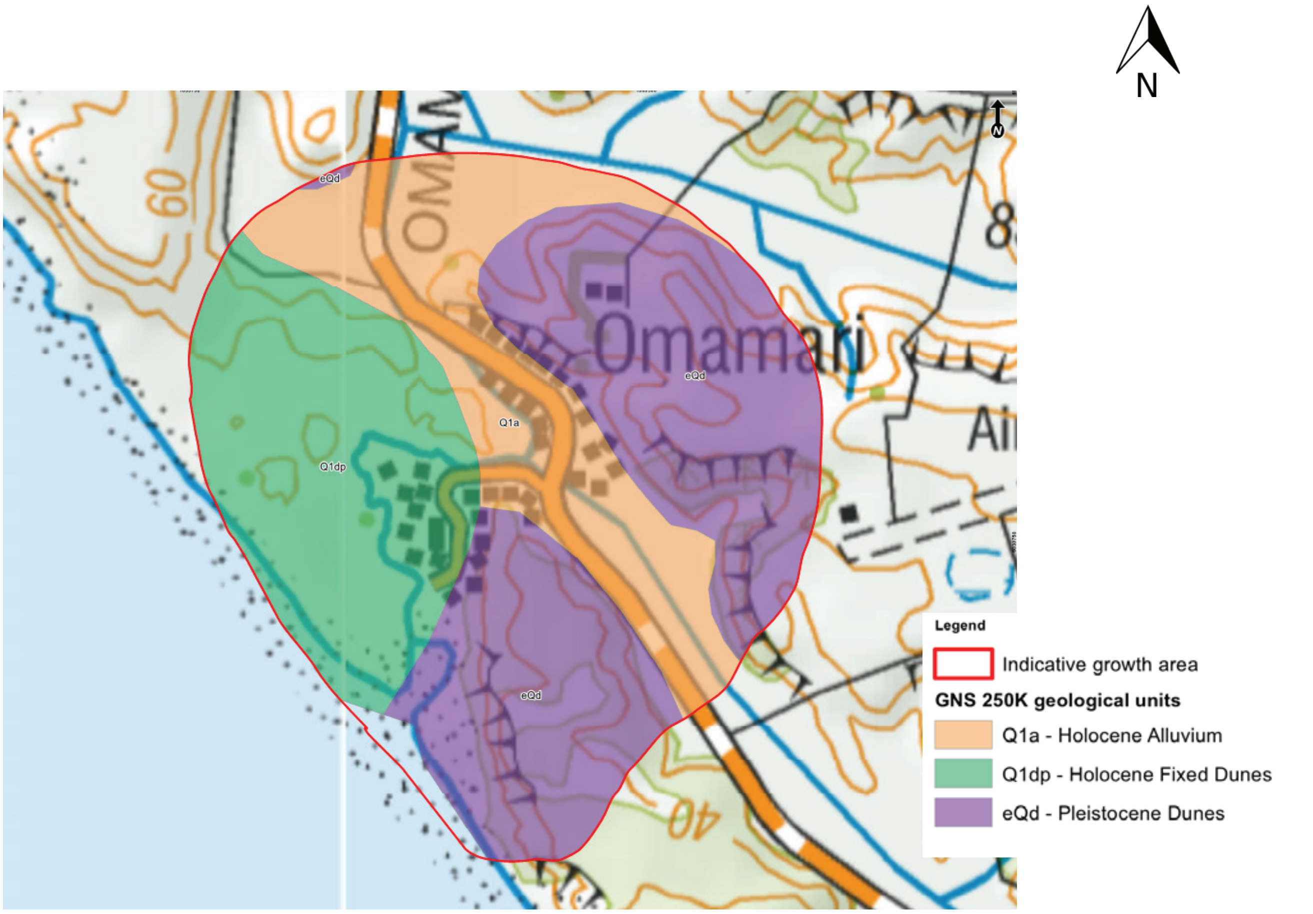


**FIGURE 1: Geology Plan for the Omamari IGA.**

concentrated on identification of geomorphological features and potential geohazards within the IGAs. Land-based mapping and photography, as well as specifically targeted oblique angle drone photography was used to gain variable points of view of features and areas of focus. GIS was used to produce maps showing slope angle, inferred slope stability, liquefaction potential, consolidation settlement potential, acid sulphate soil potential and onsite effluent disposal potential, resulting in combined geohazard maps for the study areas. A robust desktop study utilising various tools and information sources allows for enhanced efficiencies and focus during the field mapping exercise. This holistic approach to mapping projects consequently allows for enhanced data and risk interpretation which, when presented appropriately, will assist in informing policy, planning and development.

## 2. DESKTOP STUDY

As given above, the first step was to complete a high-level desktop assessment utilising a range of methods to assess each of the IGAs and identify focus areas for the field mapping phase. The various methods for the desktop study were undertaken collaboratively within the project team to produce preliminary hazard maps so that field geologists were able to spend more time assessing features and areas of interest during the limited available time in the field. The various desktop study tools are summarised as follows.

### 2.1 GEOLOGICAL MAPS

The geological settings of IGAs were established through a review of the Institute of Geological and Nuclear Sciences (GNS) 1:250,000 map (Edbrooke and Brook 2009), and supplemented by a site walkover to observe the landform and outcrops, where accessible. Although the QMAP series are a widely accepted account of the surface expression of geological units across the country, at a regional 1:250,000 scale, the detail and accuracy of unit boundaries and structural features are indicative only. Although Northland Regional Council 1:100,000 soil and rock type maps (Sutherland et al 1980, Crippen and Markham 1981) are available for some of the IGAs, the QMAP was considered more suited to assist with identification of hazard focus areas across the IGAs due to their focus on the broader geological setting rather than high detail soil and rock settings

The IGAs were underlain by many different geological formations including Tauranga Group Alluvium, Kariotahi Group Dunes, Kerikeri Group Volcanics, Waitemata Group sedimentary formations, Northland Allochthon and Whangarei Limestone; all of which exhibited variable characteristics.

The geological maps (example from of the Omamari IGA is shown in Figure 1) were integral to the desktop study in that they provided a base map and framework to identify various features that are synonymous with different geology types, such as accepted stable slope angles, low lying alluvial areas which may be prone to liquefaction and/or formations that are prone to instability.

### 2.2 AERIAL PHOTOGRAPHY

Historical aerial photographs from Retrolens New Zealand, stereo-paired aerial photos and Google Earth dating back to 1940 were reviewed as part of the desktop study. The photographs were viewed under the context of identifying general changes to the landform and development within the IGAs, and to refine the geological mapping based on the topography. Aerial photographs were useful for studying large areas, however were limited by a lack of georeferencing within the Retrolens historical aerial photographs and the physical stereopairs. It was therefore sometimes difficult to locate and focus in on target features.

For the stereoscopy, a mid-range scale of 1:25,000 was selected to provide project coverage. Images were assessed to identify geomorphic features such as headscarps, hummocky and irregular-shaped landscapes, displaced blocks and debris lobes that may be indicative of recent or historic landslide activity. Approximate extents of alluvium and colluvium deposits in hillside gullies and valley areas were also mapped, as they are more likely to be susceptible to liquefaction and consolidation settlement.

Google Earth allowed for interactive observations and assessments of the wider IGAs. By utilising a combination of exaggerated elevation, three-dimensional terrain and oblique angled navigation it was possible to identify and target areas with potential geohazards and geomorphological features to focus on during the field study.

### 2.3 SURVEY DATA

LIDAR and survey maps were used to form the base for many of the GIS map outputs and allowed for widespread slope modelling.

### 2.4 BOREHOLE LOGS

Borehole data from the KDC groundwater bore database and NZGD were used to identify representative standing groundwater depths across the IGAs.

The datasets were often limited due to lack of publicly available boreholes. This is likely due to the lower density of development in the Northland region compared to other more populated regions in New Zealand. It also reflects the fact that the NZGD is a relatively young tool and a large amount of historical information will not therefore be publicly available.

## 3. FIELD STUDY

A review of the available data was undertaken to identify key areas within each of the IGAs where additional, site-specific data collection would be targeted. The field study phase generally consisted of geomorphological mapping on the ground in those areas where access was possible and use of the drone for more inaccessible areas.

### 3.1 LAND-BASED MAPPING

The mapping was not intended to provide a detailed geomorphic map of the area, but to ground-truth the preliminary desktop maps and to observe geomorphic features that could not easily be interpreted from aerial photographs. Geohazard data was gathered at various scales from areas such as road cuttings (Figure 2 - Left), elevated viewpoints (Figure 2 – Right), dune cliff faces, low-lying alluvial plains, rolling farm land and estuarine areas.

**FIGURE 2:** **Left - Road Cutting in Mangawhai, Right – Hummocky Farm Land at Baylys Beach.**

**FIGURE 3:** Drone Photograph Facing North Over Glinks Gully.

### 3.2 DRONE PHOTOGRAPHY

The use of drone photography and videography facilitated alternative points of view of focus areas and allowed for high level observations of features in wider geographical settings (Figure 3). This allowed for observations of areas that were unable to be accessed by land, as well as providing high resolution oblique aerial images which assist in refining the nature and extent of geomorphic features. However, drone surveys were often hindered by high winds in coastal and exposed areas.

## 4. REPORTING AND GIS OUTPUTS

The data gathered from the desktop and field studies, along with soil and slope parameters from published sources and previous experience with the present geological formations, was analysed and collated to form conclusions on groundwater, active faults, slope angles, slope instability, liquefaction, consolidation, volcanic hazards, acid sulphates, on-site effluent disposal and on-site stormwater disposal.

In order to quantify the geotechnical hazard potential for land planning, a broad framework based on a three-level hazard profile was developed. This system defined potential hazard areas as Low, Medium and High, relative to the level of impact they may potentially have on future development. Risk ratings for various hazards were generally determined from published data, previous experience in the region and field observations. Hazard risk assessments included:

- Published slope data (primarily from GNS), previous studies in the region, our experience with slope stability assessments in similar geology, our field observations of stable slope angles and instability features for slope instability risk (as shown in Figure 4);
- Soil/rock type (e.g. likely presence of compressible organic soils), age of formation, and topographical setting for consolidation settlement risk;
- Age of geological formation, topographical setting, groundwater depth and soil/rock type (e.g. clay soils vs. loose sands) for liquefaction potential;
- Published data (Acid Sulphate Soil Risk maps (Opus 2017) provided by KDC), Soil/rock type (e.g. likely presence of organic soils) and topographical setting (e.g. low lying areas that may have been influenced by seawater) for acid sulphate soil risk; and
- Soil/rock type (e.g. likely soil permeability), topography and groundwater depth for on-site effluent disposal potential maps.

| Geologic Unit | Slope Instability Potential based on Slope Profile Ranges | | |
|---|---|---|---|
| | Low | Medium | High |
| Tauranga Group Alluvium | <10° | 10-23° | >23° |
| Kariotahi Group dunes | <14° | 14-26° | >26° |
| Kerikeri Volcanic Group | <18° | 18-45° | >45° |

**FIGURE 4:** Slope Instability Profile for Formations within the Mangawhai IGA.

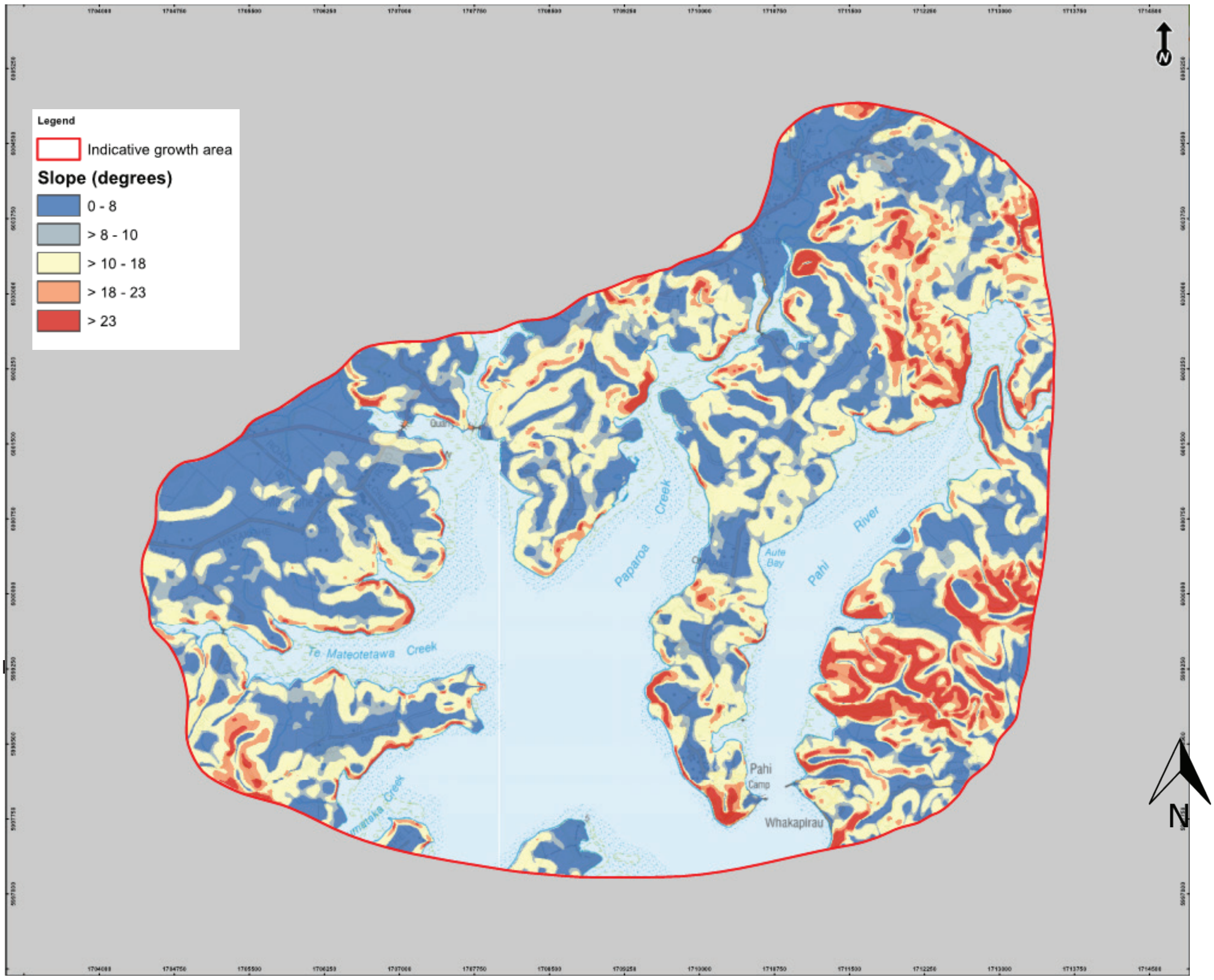


**FIGURE 5:** Slope Classification Map of the Matakohe IGA.

Due to the limited coverage of LiDAR data over the study area, the LINZ Topo50 20 m contours (vertical accuracy ≤ 10 m) were used to create digital elevation models, and then slope models of the IGAs (As shown in Figure 5).

The assessed risk ratings for each hazard were then used as inputs for Arc GIS to produce various maps for each of the IGAs, including maps for:

- IGA Vicinity showing the IGA regional location and boundary;
- Geology showing the geological setting and formations within the IGA (Figure 1);
- Geomorphological Interpretation showing the mapped areas identified as susceptible to geohazards during the field mapping;
- Slope classification showing the slope angles across the IGA (Figure 5);
- Slope instability profile showing the assessed instability risk across the IGAs;
- Settlement susceptibility showing areas of low to high consolidation settlement risk;
- Liquefaction hazard showing areas assessed to have risk of seismically induced liquefaction;

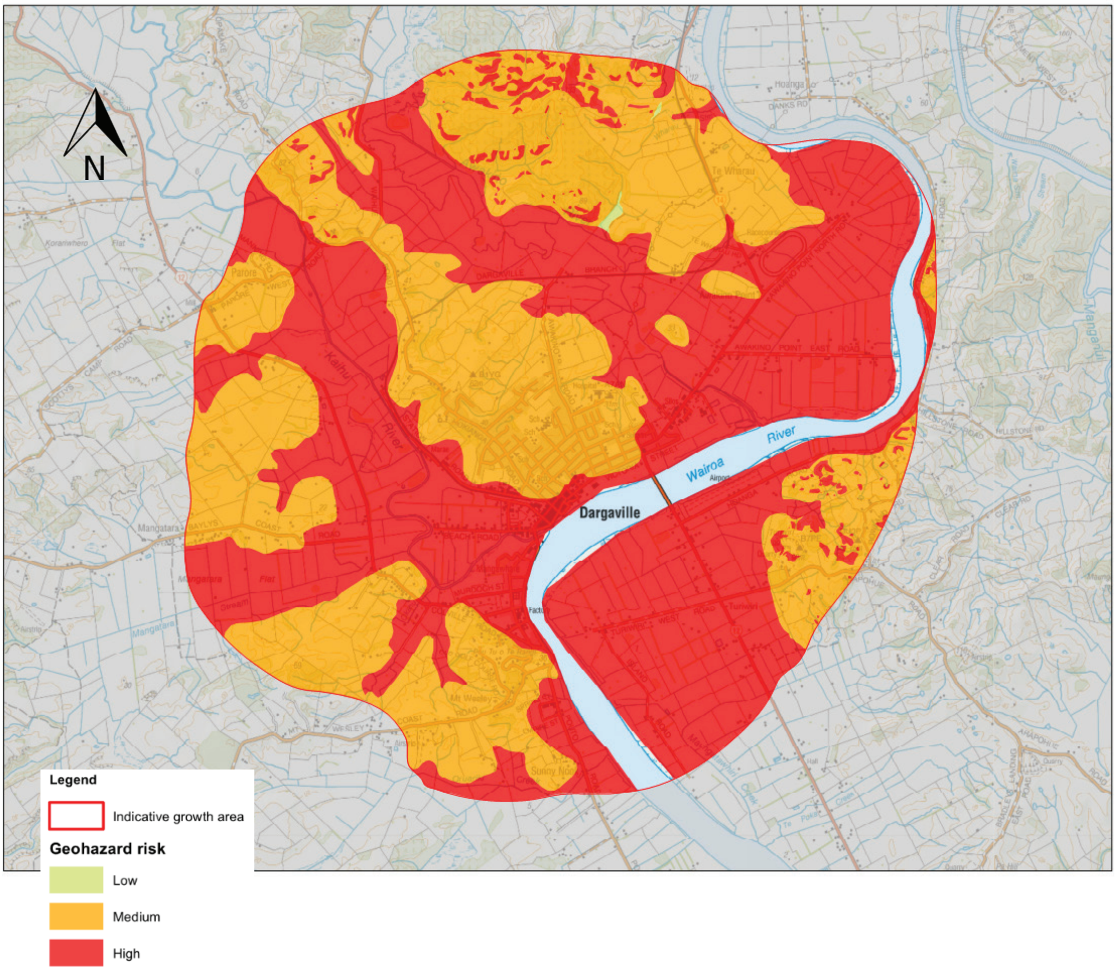


**FIGURE 6:** Combined Geohazard Map for the Dargaville IGA.

- Acid sulphate soil risk showing areas with mapped geology types considered to have potential risk of acid sulphate influence;
- On-site effluent disposal potential maps showing areas assessed to be potentially suitable for on-site disposal and those that are unlikely to be suitable.

Combined geohazard maps based on a summation of the primary geotechnical constraints considered for each IGA was then produced (Example for the Dargaville IGA shown in Figure 6). These combined Geohazard maps depicted the minimum risk level shown.

## 5. CONCLUSION

The importance of a multi-faceted approach to mapping was illustrated during a geotechnical survey and geohazard mapping project which covered approximately 25,000 ha over eight IGAs in Northland, New Zealand.

A robust desktop study was carried out which utilised a variety of tools to formulate a high-level understanding of the geological setting, geomorphological features and potential geohazards within the IGAs, which allowed for focus areas to be identified and preliminary hazard maps to be produced prior to embarking on the field aspect of the investigation. Publicly available aerial photographs (both recent and historical) played an important part in the desktop study as they allowed for a high-level assessment of the wider IGAs and identification of focus areas, with opportunity to review the development of specific features over time.

The field study placed significant reliance upon the findings of the desktop study and focussing on the target areas identified and assessing changes to landform and associated geomorphological features/geohazards.

The source and detail/limitations of available data was a key consideration for the project. The small scale of the available geological maps and aerial photographs meant risk area boundaries were considered approximate only. It should also be noted that the majority of contour data was from LIDAR surveys which can sometimes be very low resolution in remote areas. The significance of utilising multiple tools to study the IGAs and back-checking earlier assumptions against results of various assessment methods became apparent throughout the project lifecycle.

## 6. ACKNOWLEDGEMENTS

Thank you to the extraordinarily supportive and encouraging team at ENGEO for their assistance in completing this project, preparing this paper and sponsorship to attend this conference.

Thank you also to the Kaipara District Council for approval to report on this exciting project and for having the foresight to invest in gaining a deeper understanding of the characteristics of their region.

Thank you to the New Zealand geotechnical community for providing the tools and support for young geotechnical professionals to better their knowledge and training.

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# Site Behaviour and Management of Sensitive Volcanic Soils

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**KEYWORDS** volcanic soils, Whangamarino Formation, halloysite, allophane.

## ABSTRACT

**A residential development in the northern Waikato requires the excavation and placement of 700,000m$^3$ of earth fill to generate suitable building platforms for domestic dwellings and associated infrastructure. During the first season of earthwork operations the highly variable and sensitive nature of the volcanically derived alluvial silts and silty sands on site have presented unique challenges putting pressure on construction deadlines and the conventional earthwork methodology adopted. A high in-situ moisture content combined with the extremely sensitive nature of the soils requires careful planning and soil management. Excessive disturbance during excavation, transport and compaction of these soils could render initially very stiff silts to near liquid conditions. Conventional practices such as lime stabilisation to stiffen these soils are generally uneconomical and research suggests that over time the soils can revert back to the original strength due to their complex volcanic mineralogy. In this paper, I discuss the engineering characteristics of these soils, from onsite observations during the early stages of the earthworks operations. Along with the alternative methodologies employed to overcome the associated challenges to achieve the specified fill characteristics.**

## 1 INTRODUCTION

Large-scale developments generally consist of large cut to fill operations which rely on the reuse of site won material in order to generate suitable building platforms. Developers and contractors are therefore required to make the most of the soils available on site and do so within tight construction timelines.

In this paper, I describe a development in the northern Waikato where CMW Geosciences has conducted construction observations of the earthwork operation that consists of cut and fill of up to 4.0m and 8.0m in height respectively.

## 2 GROUND MODEL

### 2.1 REGIONAL GEOLOGICAL SETTING

The northern Waikato Basin consists of up to 110m thick sequence of soils of the alluvial Tauranga Group (Nelson, et. al 1988) which is separated from oldest to youngest into the Whangamarino, Puketoka and Karapiro Formations, the Kauroa and Hamilton Ashes and the Hinuera Formation.

Deposits of the Tauranga Group as stated by Nelson et. al (1988) are characterised by fluvial systems and silica rich volcanic parent material sourced from the nearby Coromandel Peninsula and the more recent central north island volcanic source. Changes in sea level and volcanism created an ever-changing depositional environment ranging from high energy braided river systems to low energy brackish lakes resulting in a complex sequence of soil types and interbedding.

From the late Pleistocene the landscape has experienced a long period of erosion, sculpting the present-day rolling hill topography which the more recent volcanic ashes mantle.

## 2.2 PROJECT GEOLOGY

The published geological map (Institute of Geological & Nuclear Sciences Limited, 2001) for the area shows the rolling hill topography where the deep cuts were undertaken to be underlain by alluvial pumiceous clays, silts, sand, gravel and lignite of the Late Miocene to Mid Pleistocene aged Whangamarino Formation. Nelson et. al 1988, describes the depositional environment for these soils to be controlled by meandering rivers which has led to a complex soil profile with high vertical and lateral variability. On the elevated areas these soils are typically overlain by several metres of deeply weathered younger volcanic ash beds which mantle the rolling hill topography.

This geology was confirmed by the findings of the site investigation with special mention of the localised concentrations of particularly halloysite rich soils throughout the site. The presence of a layered soil profile with contrasting permeabilities had created perched water tables and localised areas of increased moisture content.

## 2.3 SOIL MINERALOGY

Pumice rich alluvial soils common across the Bay of Plenty and Waikato regions are generally more difficult to work than other alluvial soils due to weathering and alteration of volcanic glass to allophane and halloysite. The typical weathering process first forms allophane to which further chemical weathering occurs to form halloysite (Wesley, 1973). As shown in Figure 1 this process also has an effect on the water holding capacity of the soil.

These minerals have unique structures which when present as a small fraction of the soil mass can control the overall soil behaviour. In this case halloysite mineralogy is more prevalent in the Whangamarino Formation soils due to the poorly drained and layered soil profile.

### 2.3.1 Allophane

The allophane minerals consist of aggregations of very small spherical shapes generally formed in well drained environments with vertical water seepage (Wesley, 2010). Wesley reports void ratios within these soils to range from 1.5 to 8 therefore allophanic soils are typically characterised by extremely high-water contents although they do not display “quick” behaviour when disturbed. It is noted that this research had been carried out in Indonesia. Upon drying, these soils can undergo irreversible changes resulting in previously plastic soils to become non plastic due to the breakdown of the mineral structure and loss of permeability (Wesley, 1973).

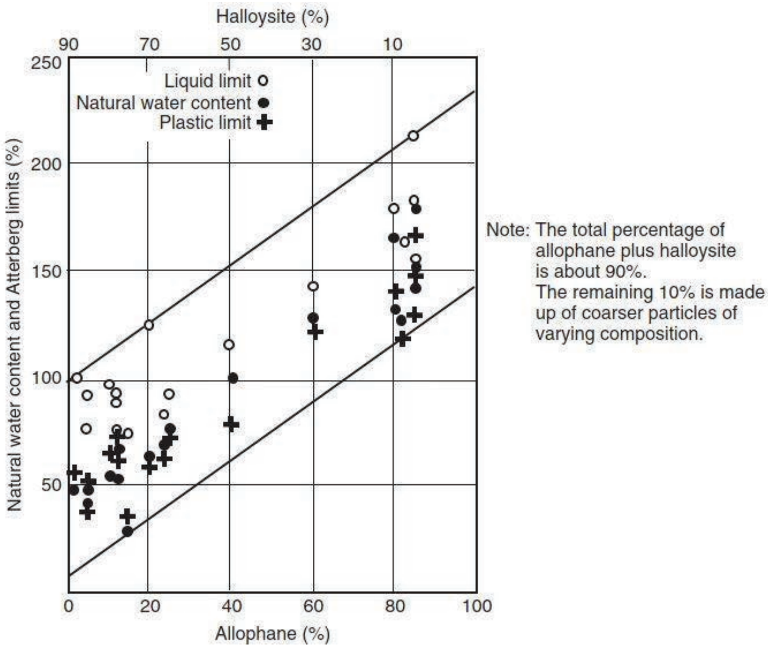


**FIGURE 1:** **Allophane and Halloysite content relationship of weathered volcanic soils in Indonesia (Wesley 1973).**

**FIGURE 2:** Remoulding of sensitive soils at design level through the movement of large machinery.

#### 2.3.2 Halloysite

Halloysite minerals are typically formed in poorly drained environments with microstructures such as hollow tubes, spheres, plates and books. The fine grain size and high porosity of these structures has been stated to create a strong capillary force holding large volumes of water within the clay structures leading to a high moisture content of the soil (Wyatt, 2009).

Upon disturbance of these soils the microstructures break down expelling the water. The expelled water dilutes the clay bonds causing a large reduction in strength resulting in high sensitivity and a loss of plasticity. In some cases documented by Wyatt, the water content exceeded the liquid limit of the soil following the disturbance of the soil.

### 2.4 EXPECTED GEOTECHNICAL ISSUES

Based on the known depositional setting of the soils on the site, and the known weathering processes in the region, the geotechnical risks associated and anticipated with soils of the Whangamarino Formation included high sensitivity to disturbance and remoulding, and rapid vertical and horizontal variation.

## 3 ON SITE OBSERVATIONS AND TESTING

### 3.1 EARTHWORKS METHODOLOGY

On large developments time and cost are important driving factors behind the construction methodology that is adopted. In this case the project required excavation and transport of up to 700,000m$^3$ of material within tight construction deadlines. The contractor initially chose to adopt the use of large motor scrapers to excavate and transport material with heavy bulldozers to spread the fill and sheepsfoot rollers to compact the fill. The slower but more precise machinery such as excavators and graders were to be retained for trimming to design levels where precision was necessary.

### 3.2 OBSERVED SOIL BEHAVIOUR AND SITE OPERATIONS

#### 3.2.1 Excavation of Sensitive Soils

Investigations identify the in-situ soils of the Whangamarino Formation to be generally stiff to very stiff. During the early stages of the earthworks operation, it became evident that the earthworks construction methodology and size of the machinery being used was too large for the highly sensitive soils.

The motor scraper excavation and transport were

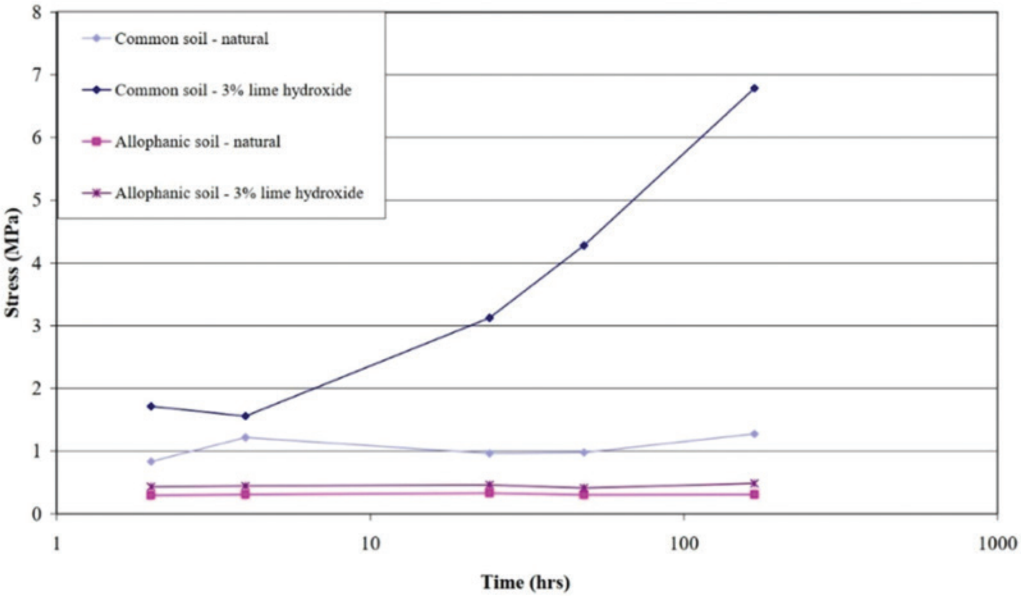


**FIGURE 3:** Stress required for a needle to be pushed into a low allophane soil (13.5%) compared to a 'common' soil (Kett et. al, 2010)

observed to remould the soils to a depths of up to 0.8m, greatly softening them and making excavation and placement difficult. On site testing demonstrated remoulded shear strengths up to 30 times lower than peak strengths in the sensitive soils of the Whangamarino Formation causing plant to get bogged down and severely rutting the soils in the cut areas posing a threat to the quality of the soils at final level as shown in Figure 2.

This was discussed within the project team and it was decided that the lost time and risk of disturbing the final foundation soils outweighed any increases in efficiency gained by the use of motor scrapers in these sensitive materials. Therefore, an alternative excavation methodology was recommended where motor scrapers would cut to approximately 0.5m above design level and the remaining material would be top loaded using excavators and dump trucks working on the raised surface to minimise disturbance of the final foundation soils.

Where areas of sensitive material had been identified during the site investigation and/or encountered during the construction process they were entirely block cut to avoid time lost retrieving bogged machinery.

### 3.2.2 Placing Fill Over Sensitive Soils

In low-lying areas of the site where fills of up to 6.0m were proposed the underlying impermeable lignite caused high groundwater tables. Excess pore water pressures generated during heavy plant operation could not dissipate, 'pumping up' the underlying soils causing large areas of soils to effectively liquefy making it impossible for the movement of heavy machinery.

With large areas of problem soils present the use of imported granular material to form a drainage blanket and act as a 'starter layer' proved to be too expensive.

An alternative method was employed whereby networks of subsoil drains were installed in the natural soils underlying the fill area in order to help dissipate excess pore water pressures developed during trafficking and fill placement. This was combined with a 0.5m thick clay starter layer which was given nominal compaction creating a raised working platform to reduce further disturbance of the natural soils.

### 3.2.3 Variability of Soil Types

The vertical and lateral distribution of the highly sensitive halloysite rich soils required specific investigation to delineate and quantify them, with remediation works assessed on a case by case basis. This reminds us that a ground model is never complete and requires to be constantly updated throughout the construction process.

## 4 FILL STABILISATION

### 4.1 LIME STABILISATION

A common method to aid the compaction of soils with high moisture contents is the addition of a binder such as lime and/or cement. The binder is mixed with the material as it is placed, with the binder and soils undergoing a chemical reaction forming cementitious bonds strengthening the soil.

The results of testing in natural and lime stabilised allophanic soils carried out by Kett et. al, 2010 is illustrated in Figure 3.

The results indicate that the addition of lime to a "low allophane soil" makes little difference to the soil strength compared to that described by the authors as a "common soil". The authors suggest that by adding higher percentages of lime it may be possible to cause a strength increase although it is deemed this would not be economical. Investigations conducted by

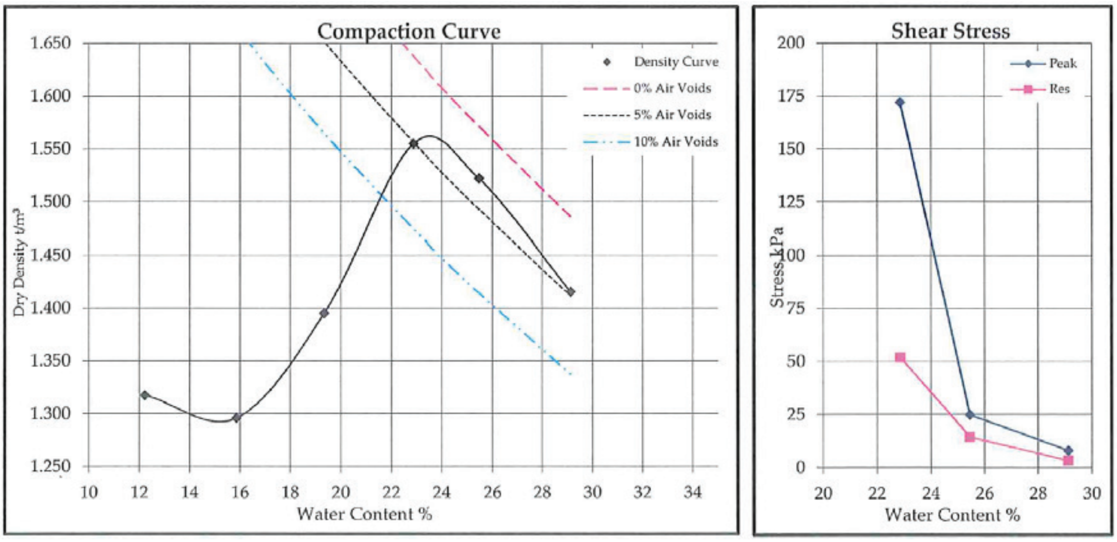


**FIGURE 4:** Laboratory Compaction Curve results for one of the cut soil blended and used as fill.

McNamara, 2003, on allophanic soils encountered in Fiji has indicated that soils stabilised with low proportions of lime can revert back to original soil behaviour between 6 to 12 months following treatment, likely due to the unique mineral structure.

Literature on the effectiveness of lime/cement stabilisation on halloysite soils is not readily available, although based on the similar mineral structures it is expected that halloysite soils would behave similarly. This hypothesis combined with the specialist equipment required and high costs made lime/cement stabilisation uneconomical for the large scale bulk earthworks operation.

### 4.2 SOIL MIXING

The more cost-effective method of using these difficult soils is to blend multiple soil types and allow them to air dry prior to compaction. Due to the high moisture contents of halloysite and allophane soils this needs to be done carefully and in thin layers, with only a minor proportion of these soils in comparison to the other constituents. In this case halloysite rich soils were mixed with other clay-rich and sandy layers of the Whangamarino Formation and the clay-rich volcanic ashes. The mixing of multiple fill sources takes great skill and over working of the soil can still lead to the degradation of the soil structures.

Figure 4 indicates the compaction curve results for one of the cut soils blended and used as fill on this site. Note the defined peak in the dry density curve around 23% and the sharp decrease in peak vane shear strength of the soil sample tested at 25%.

This indicates the small margins that the contractor worked within and highlights the need for careful management of excavated materials within the fill area in order to meet the required fill compaction criteria of 120kPa undrained shear strength.

## 5 CONCLUSIONS

Following the first season of earthwork operations within a large-scale residential development in the northern Waikato, conventional excavation methods have struggled with the highly sensitive soils of the Whangamarino Formation, resulting in lost time and disturbance of soils exposed at design level. An alternative method using a combination of motor scrapers and excavators has since successfully been used reducing further remediation works at finished level.

Due to the unique mineralogy of these soils and scale of the earthworks operation, lime/cement stabilisation was not economical and carried significant technical risk. Soil blending/mixing was therefore adopted to re-use the sensitive soils. This relied on having large volumes of soil to mix problematic soils with and a skilled contractor. The soils available on this site have a small margin of error requiring careful management and planning in order to meet the required compaction criteria.

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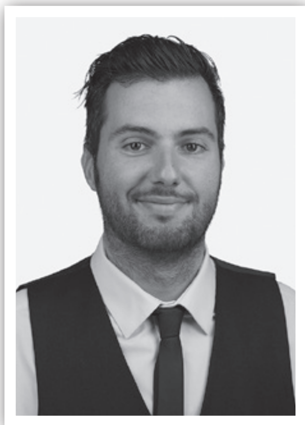
# PARAWEB MSE ABUTMENTS
# TARAMAKAU RIVER, WESTLAND

# Construction in Christchurch's Soft Peat Deposits

L. Janett and T. Magalhaes



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**KEYWORDS:** Peat, Consolidation, Bearing Capacity, Constructability, CNC, Cranford Basin

## ABSTRACT

**The Christchurch Northern Corridor is a scheme involving the construction of a new 4-lane highway connecting the City of Christchurch to the Northern Arterial Motorway. The scheme's site is predominantly underlain by up to 15m of alluvial deposits including very weak, highly compressible peat and organic silt materials, as well as dense sand horizons, overlaying the Riccarton Gravel. These materials are associated with the historic swamp and overbank deposits of the Yaldhurst Member, Springston Formation and are up to 10,000 years old. Of note, the new highway's southern extent crosses the Cranford Basin: a large and isolated pocket of peat, approximately 5 m deep, overlying interbedded horizons of sand, silt (organic in places) and peat. Since construction began in Early 2017, new embankments and temporary surcharge areas have experienced significant consolidation settlements, bearing capacity failures and heave. This paper provides details regarding (a) risks and challenges associated with constructing on soft peat deposits; (b) adopted mitigations; and (c) the settlement back-analyses undertaken to verify long-term performance.**

## 1 INTRODUCTION

This paper presents the risks and challenges encountered during the design and construction phases of the Christchurch Northern Corridor project (CNC), associated with constructing on soft peat deposits which are prevalent throughout the site. The main risks and challenges comprised excessive settlement, inadequate bearing capacity, for both structures and road foundations, and effects of seasonal groundwater variations.

This paper also summarises the back-analysis to show compliance with design specifications and project long-term Minimum Requirements (MRs). This approach involved a robust settlement, deflection and porewater pressure monitoring regime and the back-analysis of the design settlement parameters.

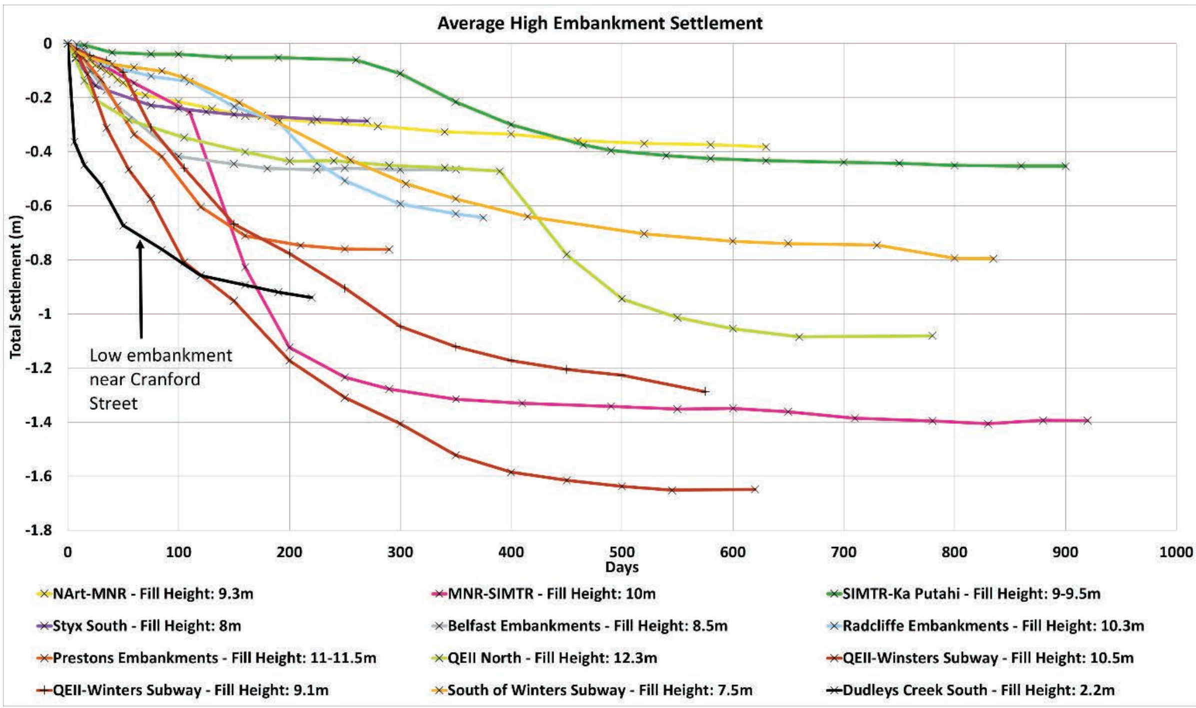


**FIGURE 1: Average total settlements across sections of high embankments where the largest settlements were observed (note - the Cranford Street line is a single instrument, not an average).**

## 2 RISKS AND CHALLENGES

Compressible peat and organic silt deposits vary in thickness, consistency, depth and compressibility across the site. The prevalence of this material combined with variable embankment heights and typically high groundwater also made for challenging design and construction. The staging of embankment construction and surcharge fill meant sixty-six (66) distinct earthworks areas were defined, each with their own construction staging and design for settlement mitigation.

### 2.1 ROAD EMBANKMENT CONSTRUCTION CHALLENGES

#### 2.1.1 Settlement and Groundwater

As shown in Figure 1, the averaged total settlement and consolidation time of high embankments varied significantly across the project. The heights presented in Figure 1 are the combined embankments design heights plus surcharge thickness. Most high embankments (4-8.8 m high) settled between 0.6 to 1.6 m with primary consolidation typically complete around 6-12 months. The low embankments (0.5 - 4 m high) settled between 0.1 to 0.5 m, typically performing as per design. However, in discrete locations, residual settlements and post-pavement construction settlement predictions exceeded the MRs. Where reduced, observed and predicted, creep settlements were recorded / predicted, this is believed to be due to the effectiveness of the applied surcharge thickness.

The embankment construction also resulted in lateral movements of soils, which was a significant design challenge around piled structures due to the potential for pile bending due to consolidation.

The largest thickness of surcharge to design embankment ratio on the project of 0.5 was applied to the low embankments around the Cranford Street Southern Tie-In fills, one embankment (as shown in Figure 1) settled 0.95 m with an applied fill thickness of 2.2 m. Heave between 10-30 mm was typically observed across the project following surcharge removal.

Groundwater typically varies between 0.5 to 1.5 m below ground level (bgl) along the alignment. In locations such as the Cranford Basin, groundwater is at ground level. In addition, there are artesian conditions, to sub-artesian at certain locations, at depth, at the top of the Riccarton Gravel. High groundwater and a prevalence of springs made construction difficult and had a notable effect on the settlement of embankments in some areas, particularly in the southern zone of the project. Approaching Summer, as the groundwater level started to drop, the embankment settlement occasionally accelerated, making surcharge back-analysis more difficult and causing programme delays.

#### 2.1.2 Bearing Capacity Failure

The Cranford Basin has up to 5 m of highly compressible peat and organic silt with little to no stiff crust to provide bearing resistance to embankment loading. For each of the Cranford Tie-In embankments the settlement was 25% to 50% of the placed fill thickness including surcharge. This typically resulted in bearing capacity and global stability failure of the embankments as shown in the photos in Figure 2. This occurred despite the rate of filling being generally controlled. The undrained shear strength of these soils was back-calculated to be as low as 5 kPa.

**PHOTO 1:** Global instability of sidling fill surcharge caused rotation of light poles and poplars.

**PHOTO 2:** Global instability and settlement of sidling fill caused the road to break apart at service trench.

**PHOTO 3:** 0.6 m working platform ~4 m away, caused 0.5 m of heave.

**PHOTO 4:** Adjacent fills (~2 m high) caused 0.5 m heave of timber drain.

**PHOTO 5:** ~1.6 m fill placed ~3 m from drain,buckled timber in the base of the drain and deflected drain walls by 0.18 m.

**PHOTO 6:** ~1.3 m of fill placed 3 m from swale, caused 0.6 m of heave as shoving failure.

**FIGURE 2:** Photos of bearing capacity and circular failure of soils around the Cranford Street South Tie-In.

## 2.2 NETWORK DRAINAGE CONSTRUCTION CHALLENGES

The project has ten major cross culverts including three large post-tensioned box culverts and many network drainage lines, at or below existing ground level. The main risks associated with these structures included differential settlement between the structure and the embankment and construction on soft soils and below groundwater level. In areas worst affected by compressible soils, such as the Cranford Basin, minor structures such as manhole chambers, culvert head walls or drainage lines, recorded settlements of 50 - 125 mm, even where little net load was applied to the foundation soils. These settlements were likely due to primary consolidation as these occurred generally within weeks.

## 2.3 SURVEY MONITORING CONTROL POINTS

One of the biggest challenges to the project was the consistency of settlement monitoring survey data. Though control point calibration was undertaken regularly, the movement of control points often resulted in variability in the data, fluctuating as much as 20-40 mm, outside of an adopted survey tolerance of 35mm. This made it challenging to assess trends and provide settlement predictions for low embankments with small total settlements (as shown in Figure 3A).

For embankments in the Cranford Basin, large fluctuations created uncertainty as to the completion of 90% primary consolidation and compliance with the MRs (as shown in Figure 3B). This caused programme delays during the approval process for surcharge removal.

# 3 MITIGATIONS

The design and construction challenges on the CNC project required innovative engineering solutions, methodology changes and at times a risk-based approach to decision making. This section outlines the mitigation solutions for the risks and challenges presented in Section 2.

## 3.1 ROAD EMBANKMENTS AND CONSTRUCTION CHALLENGES

### 3.1.1 Settlement and Groundwater

To mitigate settlement risk, settlement analyses were completed for all earthwork areas and where required, surcharge and wick drain schemes were designed. Laboratory testing and trial embankment data were used to derive settlement parameters during design. As the project progressed, the parameters were refined by back-analysis based on the actual embankment performance, as design settlement predictions both over and under-predicting the actual field settlements.

During construction, back-analysis was undertaken to assess compliance with the project's MRs for each

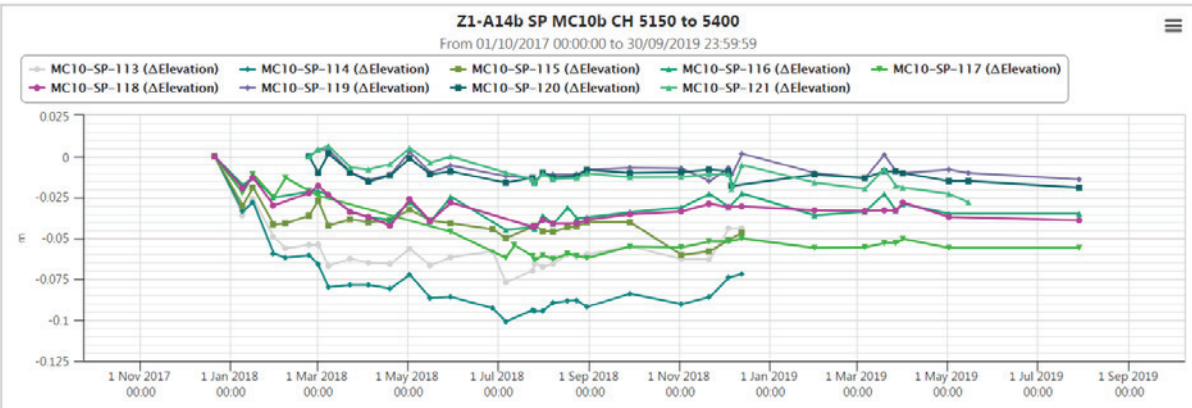


**FIG. 3(A)**

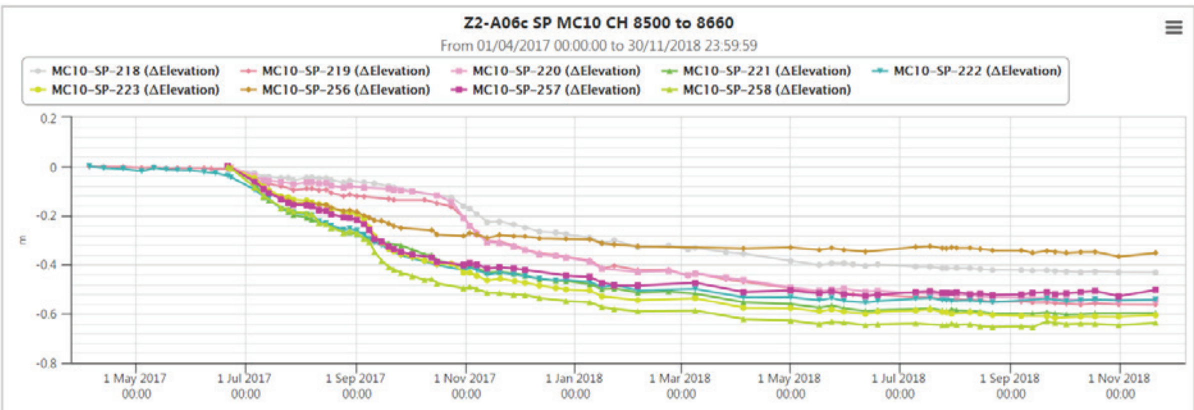


**FIG. 3(B)**

**FIGURE 3:** (A) Low embankment settlement fluctuations. (B) Settlement fluctuation in Cranford Basin (Trimble, 2019).

earthworks area before surcharge was removed and to extend the surcharge hold period if necessary. This approach is outlined in Section 4. In addition to the MRs, the CNC Alliance instated a 2-month 'Pre-Paving Hold Period' so that the effectiveness of the surcharge could be confirmed, or additional action taken if settlement was still on-going.

In some locations, the back-analysis of surcharge could not prove compliance with the long-term settlement MRs; for these areas a risk-based approach was adopted. The Project Alliance Board (PAB), consisting of Alliance partners and clients, reviewed risk assessments comparing the consequences of the predicted ongoing settlement, against the costs and risks of potential mitigation measures. For some areas, it was concluded that there was little benefit in maintaining the surcharge. For other areas, evidence of significant creep settlement meant that the reputational risk and expected damage to infrastructure outweighed the programming and project cost implications. Therefore, in these areas, the surcharge hold period may have been extended by several months to reduce the risk of long-term settlement.

The consequences of ongoing total and differential settlement to the road safety and performance ranged from loss of road crossfall, reducing the effectiveness of the network drainage, to damage to kerbs, barriers, pavement and ride quality.

To mitigate the effects of groundwater and excess pore pressure build up, wick drains were installed to 7-8 m bgl at 1.5-2 m spacings in areas where soft and compressible cohesive soils were present and where the programme for construction was relatively short. In several locations where wick drains were not installed, such as the Cranford Basin, increased settlement duration and potential non-compliance with long term settlement MRs were observed due to slower than expected pore-pressure dissipation.

### 3.2 NETWORK DRAINAGE

To mitigate the risk of total and differential settlement of cross culverts and network drainage lines, a risk assessment was completed across the project for every individual pipeline. A combination of ballast raft construction and/or surcharging of pipe alignments was implemented as required to mitigate long-term settlements. Surcharging was undertaken by excavating the trench and placing the pipe foundation so that the heavier trench backfill was in place for the surcharge period. Once the settlement had slowed the trench was re-excavated to lay the pipe, and the backfill replaced. Pipe trench settlements exceeding 150 mm were observed just due to the net increase in weight of the trench backfill, with no fill placed above original ground level. The success of this methodology will be proven with time, however, as discussed in Section 2.2 in locations where infrastructure was constructed outside of the alignment and were not surcharged, much greater post-construction settlement was observed than surcharged sections. Pipe rafts were not observed to reduce total pipe settlements, although they did mitigate differential settlements and provide a stable base for the pipe to be laid on.

For major culverts along existing drainage lines, ballast rafts and temporary corrugated steel culverts, to be subsequently replaced with permanent culverts, were installed prior to embankment/surcharge filling. The temporary culverts deformed significantly due to the embankment settlement and construction loading. This made it difficult to assess the true culvert settlement and to determine whether the risk of long-term differential settlement had been effectively mitigated.

Piled culverts were avoided as they would result in localised differential settlements either side of the culvert, resulting in poor ride quality and drainage inefficiency due to any ongoing settlement. The permanent culverts were over-sized, so that they could settle with the road embankment while still maintaining sufficient capacity and ride quality. As a secondary mitigation, to reduce the effects of differential settlement at culverts, two layers of geogrid were installed in subgrade and pavement layers at the transition between embankment fills and fills over some major culverts.

### 3.3 SURVEY MONITORING CONTROL POINTS

Approximately midway through the project, following issues with consistency in monitoring results, a mitigation measure was implemented to improve accuracy of readings. The original control points which were installed in a range of locations near the embankments, from ground points to ancillary structures generally founded at existing ground level, were replaced by UC steel piles installed into dense sand layers around the site. There has been no quantitative assessment comparing the accuracy of monitoring before and after the control point upgrade, however, from a qualitative perspective, there was a clear reduction in large monitoring fluctuations that had been hampering the project.

## 4 COMPLIANCE AND LONG-TERM PERFORMANCE

The MRs, defined to assess satisfactory behaviour, focused on long-term ground displacements and degree of consolidation. Limits were specified in the MRs and are summarised in Table 1. To verify compliance with the MRs, a back-analysis supported with numerical verification was adopted by the following methodology:

1. Settlement, deflection and porewater pressure monitoring.
2. Back-analysis of design soil parameters.
3. Long-term settlement prediction against MRs (based on back-analysed soil parameters).

Long-term settlement prediction was undertaken considering two distinct analysis methods to provide robustness and confidence to the predictions: 1) Terzaghi's one-dimensional consolidation theory, assuming that primary and secondary consolidation

**TABLE 1: Minimum Requirements (MRs) adopted in the CNC project.**

| CRITERIA | TIME | LIMIT |
|---|---|---|
| **Total predicted Long-term Static Settlement** | 50 years following Practical Completion | 150mm |
| **Degree of Primary Consolidation** | Prior to building pavement and surface features | >90% |
| **Settlement Rate** | The month preceding pavement and surface features construction | <20mm per year |
| | Over the 6 months prior to the end of the defects liability period | <5mm per year |
| **Static Settlements** | End of defects liability period | 35mm |
| **Angular distortion from the design surface profile** | End of defects liability period | <1/300 over a 3m distance |

settlements occur separately; and 2) a non-linear consolidation method, assuming that primary and secondary consolidation settlements occur simultaneously. The results of these assessments were independently peer reviewed.

This approach provided a robust risk-assessment based on real-life, site-specific data. Despite some challenges regarding monitoring accuracy, as discussed in Section 2.3, it is considered that this approach was successful. It allowed the key stakeholders to evaluate the risks and identify when mitigation measures were required, resulting in significant overall risk reduction.

## 5 CONCLUSION

The soft, compressible peat deposits underlying much of the CNC alignment presented challenges manifesting in low bearing capacities of embankment foundations, excessive construction and consolidation settlement, and excessive movements of structures and foundations.

These challenges were overcome via ground improvements in the form of embankment surcharging, use of basal geogrid, wick drains and raft foundations.

A robust back-analysis and verification process was adopted to assess the effectiveness of the adopted mitigation measures and extend or reduce the surcharge periods where required.

It is considered that the mitigation measures adopted, verification analyses and on-site control mechanisms, enabled the successful delivery of a major highways scheme underlain by variable and challenging ground conditions.

## 6 ACKNOWLEDGEMENTS

The authors would like to acknowledge the contribution of Waka Kotahi and the CNC Alliance in the provision of the design and construction information presented in this paper.

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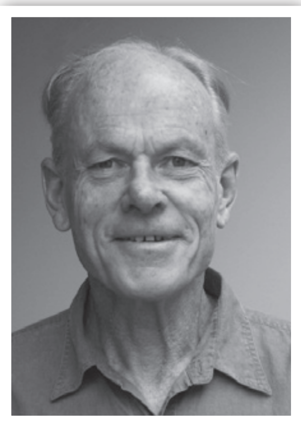
# Cantilever Pole Walls on Sloping Ground and in Cohesive Soils with Bearing Capacity Varying With Depth

John Wood, Retired Civil Engineer



John Wood


## ABSTRACT

**In the paper on cantilever pole walls presented in the June 2021 issue of *NZ Geomechanics News* (NZGN) a summary was presented of analysis methods for determining the lateral load capacity of the embedded pole foundations (Wood, 2021). The testing of rigid timber poles of similar dimensions used in cantilever pole walls in Auckland clays were discussed by Marchant and Wood in the December 2021 issue of NZGN.**

**The present paper provides additional information on the analysis of cantilever pole walls in both cohesionless and cohesive soils with particular reference to walls located near the crest of a down slope in front of the wall. In the June 2021 paper only the load capacity of the poles located on level ground was considered. For cohesive soils the analysis methods considered were based on the bearing capacity factor being constant with depth. An adjustment for depth variation was made by assuming a depth of ineffective soil below the ground surface with a constant bearing capacity factor below this depth. In the present paper two methods that are based on a depth variation of the bearing capacity factor are described. One of these methods (Georgiadis et al, 2013) addresses the effect of slopes on the lateral load capacity of piles in clay soils.**


## 1. INTRODUCTION

Cantilever timber pole walls are one of the most commonly used forms of retaining wall construction used in New Zealand for low to moderate retained heights. Pole walls in New Zealand have mainly been designed using the Broms, 1964a and 1964b isolated pile theory or by continuous embedded wall theory. These commonly used design analysis methods have limitations and do not accurately represent the pole-soil interaction and displacement behaviour. In particular, the Broms method for estimating the pole lateral force capacity in cohesionless soils assumes rotation of the pole about the toe instead of the correct depth at a significant height above the toe. This simplification gives an unconservative lateral load capacity. Simplifying the analysis by assuming that the poles behave as a continuous wall is unnecessary and may introduce errors that are difficult to quantify.

Although soil strength parameters are unlikely to be accurately known for design of small wall structures it is nevertheless desirable to eliminate analysis errors as far as possible and deal with the soil uncertainty by adopting moderately conservative soil strength parameters.

Pole foundation soil-interaction design should be based on pile theory that has been verified by testing. Pile design methods suitable for cantilever wall design have been presented by Guo, 2008 for cohesionless soils, Motta, 2013 for cohesive soils and by Zhang, 2018 for the mixed friction and cohesion case. The Guo, 2008 and Motta, 2013 methods are based on elastic-plastic soil response and give force versus displacement equations for loading from zero up to the ultimate hyperbolic loads. These allow serviceability displacements and the pile top lateral force capacity based on soil yield at the toe to be determined.

Application of these more correct soil-interaction pile analysis procedures to the design of cantilever pole walls is presented in Wood, 2021. Walls are often constructed in moderately steep terrain but the analysis procedures presented by Wood did not address the effect of a slope on the lateral load capacity of the pole foundation. The effect of down sloping ground in front of a wall on the lateral response of the poles and the influence of variation of the bearing capacity factor with depth in cohesive soils are discussed in the present paper. Evaluation of the governing equations of the methods investigated for slope effects and bearing capacity factor depth variation have been carried out to give load capacity and deflection results in graphical form that can be applied directly in wall design.

## 2. RADIAL STRESSES IN SOIL FROM PILE LATERAL LOADING

To assess slope effects under lateral loading of piles, it is important to investigate the decrease in the radial stress in the soil with distance from the pile since a downslope of the ground surface partially removes the soil support at the top of the pile reducing the lateral capacity. The radial stress change in an elastic soil surrounding a horizontally loaded pile can be approximated by (Sun and Pires, 1993):

$$\sigma_r = 2G_s u \frac{\gamma_b}{r_o} \frac{K_1(\gamma_b\ r/r_o)}{K_o(\gamma_b)} \cos(\theta) \qquad (1)$$

Where $\sigma_r$ is the radial stress; $G_s$ is the soil shear modulus; $u$ is the lateral displacement of the pile, $\theta$ is the angle between the direction of the loading and the line joining the centre of the pile cross-section to the point of interest; $r$ is the radial distance from the pile axis; $K_i(\gamma_b)$ ($i$ = 0, 1) are the modified Bessel functions of the second kind of order $i$; $r_o$ is the outside radius of the cylindrical pile. The factor $\gamma_b$ in Equation 1 is given by:

$$\gamma_b = k_1(r_o/L) \qquad (2)$$

Where $L$ is the pile embedded length; $k_1$ is 2.14, and 3.8 for pure lateral loading (eccentricity, $e$ = 0), and pure moment loading ($e = \infty$), respectively. $k_1$ increases hyperbolically from 2.14 to 3.8 as the free length $e$ increases from ground level to infinitely large (Guo, 2008).

The radial stress evaluated using Equation 1 for $\theta = 0$ and $k_1$ = 2.14 is plotted in Figure 1 as a function of the diameter ratio $r/B$, where $B$ is the diameter of the pile. $k_1$ was taken as 2.14 ($e$ = 0) and the $L/B$ ratio as 5. Also shown on the plot are results from a 3-D finite element analysis (FEA) of a typical timber pole encased in concrete and embedded in an elastic soil. This analysis was part of the present study and set-up to specifically investigate the stresses in the concrete encasement surrounding the pole. The element mesh modelling the soil was not ideal for calculating the soil stresses, and the soil boundary distance from the pole was not sufficient to completely remove its effect on the radial stresses from the pole. The $e/L$ ratio for the FEA analysis was 0.2 so the $k_1$ value would have been greater than 2.14 assumed in the theoretical analysis which would have increased the radial stress at distances less than 1.5$B$ from the pole by perhaps 5% and reduced the stresses at greater distances. Nevertheless, agreement between the elastic theory and FEA results is good indicating that the prediction of the radial stress in the soil by either method is satisfactory.

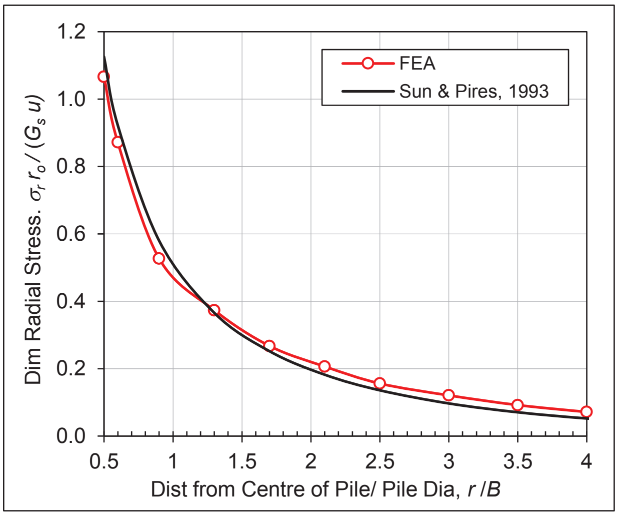



**FIGURE 1. Radial stresses in elastic soil.**

Theoretical analysis of the nonlinear interaction of piles laterally loaded in cohesionless soils is complex. Finite element analysis can be used to study this problem but numerical results for the range of pile geometries relevant to the present study have not been published.

Lin et al, 2015 carried out an experiment on a fully instrumented short, stiff pile laterally loaded at the head to investigate the nonlinear soil-structure interaction. The pile had a steel pipe section with diameter 102 mm and a length of 1.52 m. It was installed in well-graded sand and subjected to increasing lateral load. The pile and surrounding soil were fully instrumented using advanced sensors, including flexible shape acceleration arrays, thin tactile pressure sheets, and in-soil null pressure sensors. The null pressure sensor measurements were used to develop the distribution of horizontal stress changes in the soil around the pile as the lateral pile displacement increased.

Soil pressures measured in the soil at a depth of 352 mm were presented in contour plots for a number of load levels up to the ultimate load reached at the end of the test (Wood, 2021). For the present study the radial stress in the soil along a diameter at $\theta = 0$ were scaled from these contour plots and are shown in Figure 2 for load levels of 1.0, 0.7 and 0.23 of the ultimate (3.8 kN and head displacement of 86 mm).

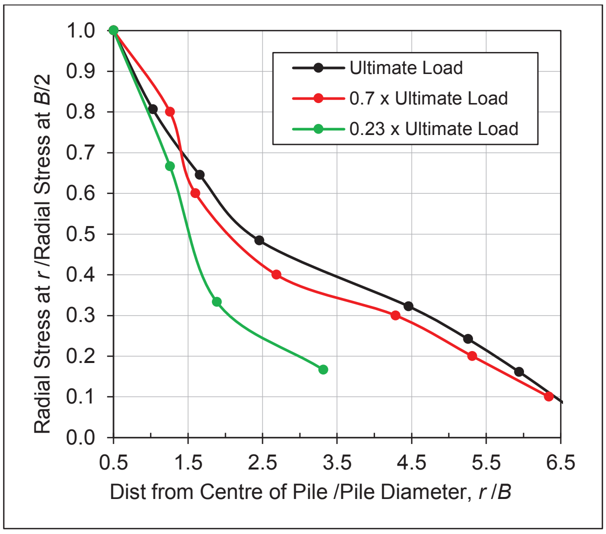



**FIGURE 2. Radial stresses estimated from Fig 16 in in et al, 2015.**

Figure 2 shows that nonlinear soil response results in higher radial stresses with increasing distance from the pile than predicted by elastic theory. However, at a distance of 6*B* the radial stress is approximately 15% (or perhaps less) of the peak stress at the pile surface. This indicates that the soil properties at distances greater than 6*B* are unlikely to have a significant influence on the lateral load capacity of the pile.

## 3. INFLUENCE OF GROUND SLOPE ON VERTICAL GRAVITY STRESSES

In a cohesionless soil, the limiting force on a laterally loaded rigid pile is given by:

$$P_u = A_r z B \tag{3}$$

Where $A_r z$ is the pressure on the pile surface ($FL^{-2}$ units) contributed by radial and shear stresses around the pile surface, and $z$ the depth below the surface. Based on the tests carried out by Prasad and Chari, 1999 (and other experimental work), Guo, 2008 assumed that $A_r$ was given by:

$$A_r = \gamma ' K_p^2 \tag{4}$$

Where $\gamma '$ is the soil effective unit weight. The lateral load capacity of a rigid pile will therefore be dominated by the vertical gravity stress ($\gamma ' z$) down the length of the pile.

The reduction in lateral load capacity of the poles of a cantilever wall resulting from the ground sloping down from the face of the wall (slope normal to the face) was estimated by calculating the vertical gravity stress profile down the slope at distances out to 6*B* from the wall face. Gravity stresses were calculated using 2-D plane strain elastic FEA models. A wall height of 3 m was modelled with 2:1 and 4:1 (horizontal: vertical) down slopes commencing at the wall face. A soil unit weight of 20 kN/m$^3$ was used.

Details of the meshes used in the two FEA models are shown in Figures 3 and 4. Results for the gravity load analyses are shown in the figures as contours of vertical stress at 10 kPa intervals. The contours show that within about 3 m downslope of the wall face there are significant variations in the vertical stress. Just in front of the wall on the model with the 4:1 slope the vertical stress at a depth of 3 m is approximately 90 kPa and on the model with the 2:1 slope the stress at the same depth is about 85 kPa. These stresses are approximately 50% greater than calculated using the depth from the ground in front of the wall, indicating that the backfill soil height behind the wall face has a significant influence on the vertical stresses below the toe of the wall face.

Plots of the variation of the vertical stress versus depth below the toe of the wall for distances of 0.1, 1.1, 2.1 and 3.1 m from the face of the wall are shown in Figures 5 and 6 for the 4:1 slope and the 2:1 slope models respectively. Three vertical stress profiles are shown at each distance from the wall; the stress based on the soil weight below the slope surface at the section ($\gamma z$), the stress from the FEA analysis, and the gravity stress for an assumed ineffective depth of horizontal ground in front of the wall. The ineffective depths were based on the slope surface depth at three pile diameters (3*B*) from the face of the wall assuming a 0.5 m pile diameter. For the 4:1 slope this gave an ineffective depth 0.375 m below the toe of the wall and for the 2:1 slope 0.75 m below the toe of the wall. Figure 7 defines the ineffective depth, $\Delta z$ which is used in the following section to calculate the limiting force profile (LFP) above the rotation point on a laterally loaded pile.

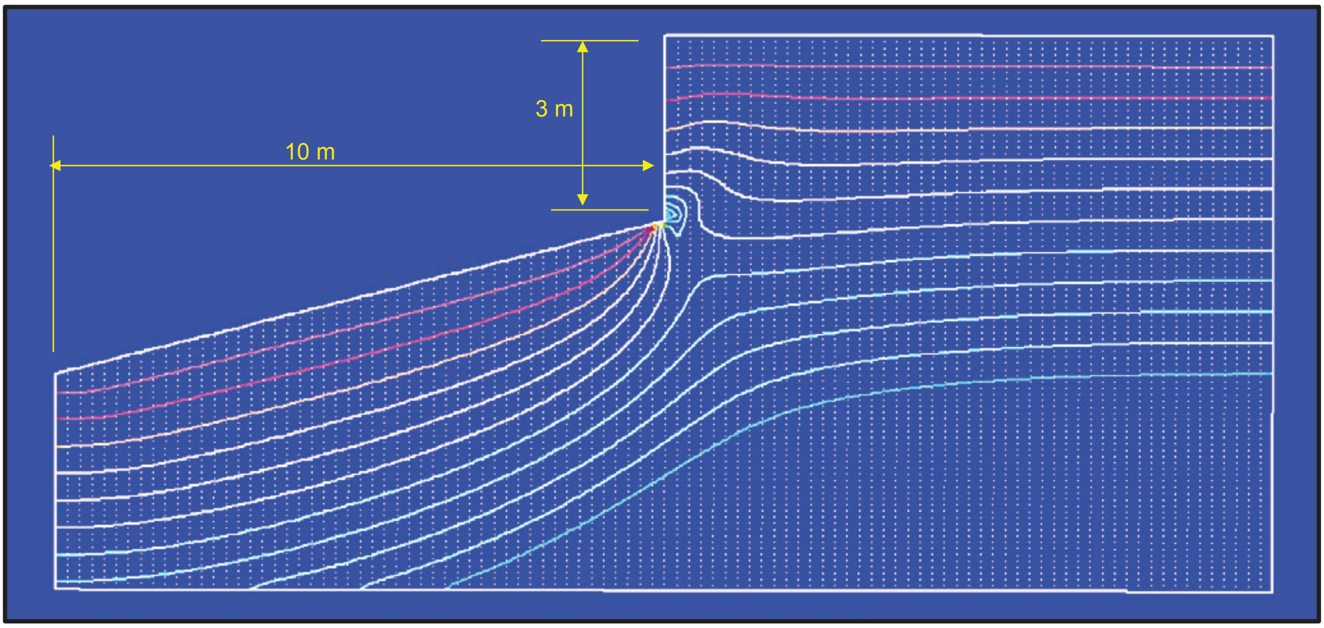



**FIGURE 3.** FEA model for 4:1 slope in front of 3 m high wall. Contours for vertical stress at 10 kPa intervals.

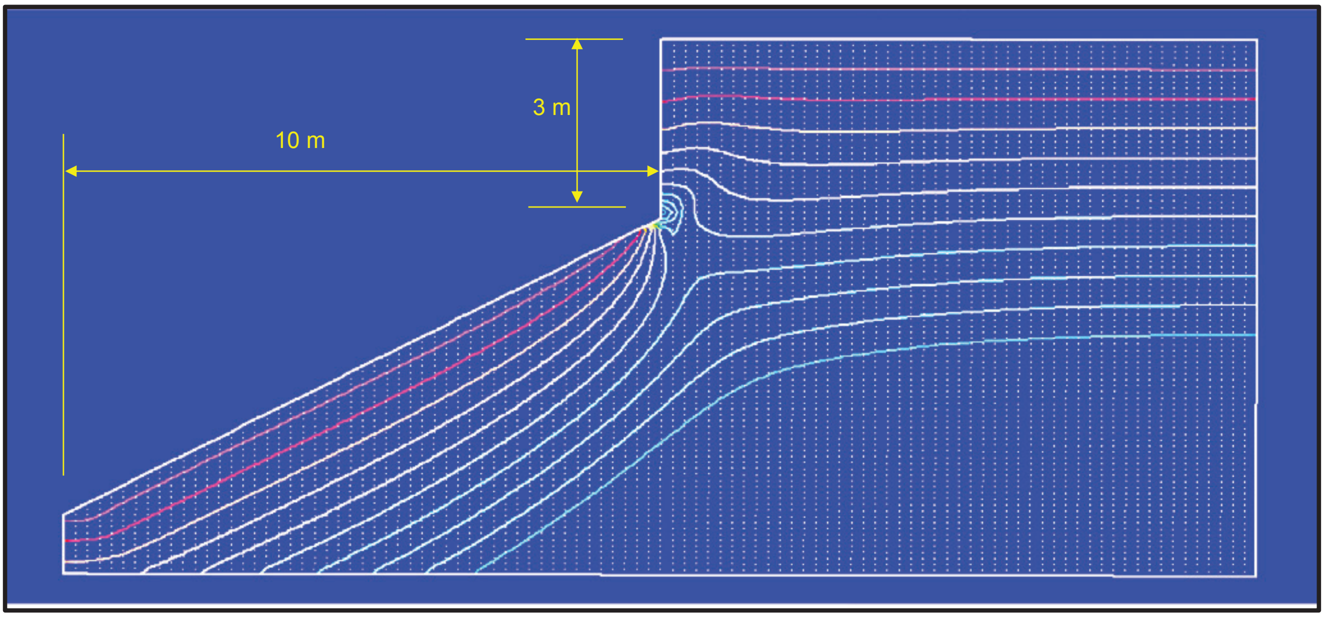



**FIGURE 4.** FEA model for 2:1 slope in front of 3 m high wall. Contours for vertical stress at 10 kPa intervals.

The FEA vertical stresses calculated by both models are greater than the stresses calculated from a simple analyses based on the unit weight of the soil and the depth below the slope surface. The differences exceed 50% near the wall face and reduce as the distance from the wall increases. Differences become less than 15% at a distance of 3.1 m from the wall. The weight of soil retained behind the wall face clearly has a major influence on the vertical stresses below the toe of the wall for a significant distance in front of the wall.

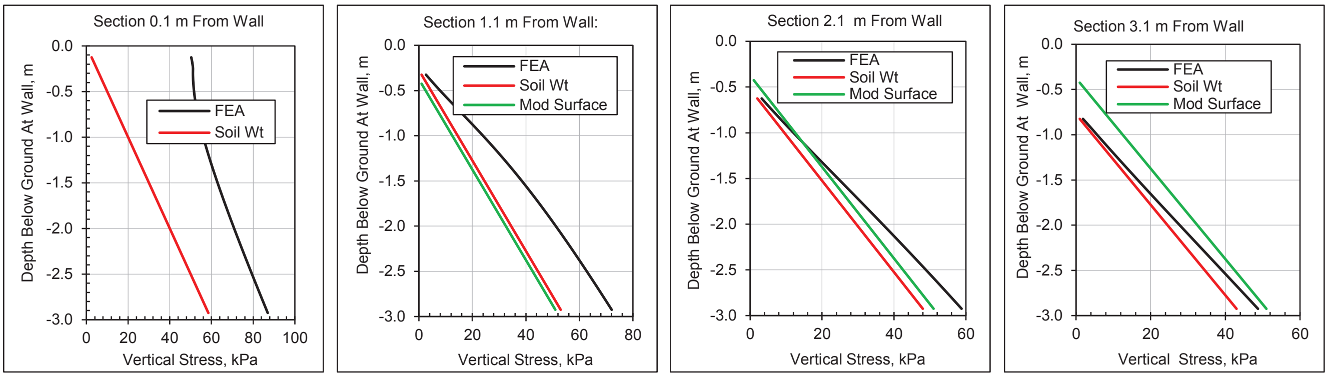



**FIGURE 5.** 4:1 Slope model. Gravity stresses calculated by FEA and theory based on soil depths and $\gamma$

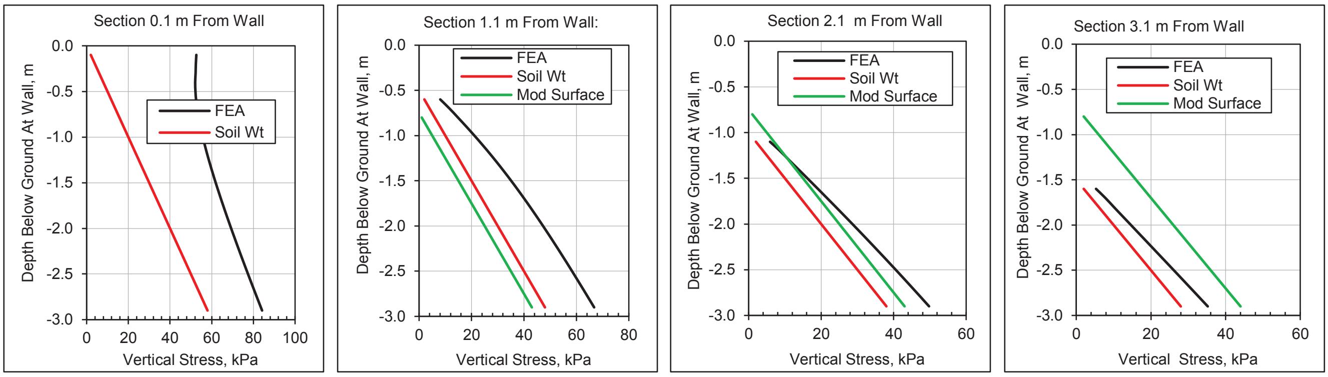



**FIGURE 6.** 2:1 Slope model. Gravity stresses calculated by FEA and theory based on soil depths and $\gamma$

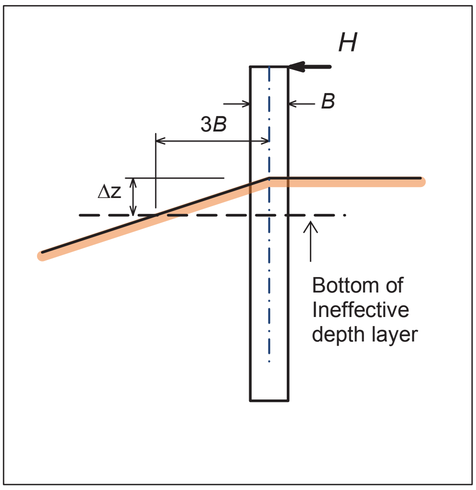



**FIGURE 7.** Ineffective depth definition.

Stresses for the ineffective depth assumption are lower than the FEA stresses out to a distance of 2.1 m or about four pile diameters (4*B*). At greater distances, the ineffective height assumption gives vertical stresses greater than the FEA results. As shown in Section 2, the radial stresses from lateral loading of a pile diminish quite rapidly and are not very significant at distances of 4*B* or greater from the pile centre. This suggests that an ineffective height assumption based on the depth of the surface of the slope at a distance of 3*B* from the pile centre is likely to be satisfactory for estimating the LFP above the rotation point of a laterally loaded rigid pile. At distances closer to the pile than 4*B* the gravity stresses are likely to be greater than given by the ineffective height assumption and this will compensate for any reduction in gravity stresses at larger distances. Because of the complexity of the pile radial stress distribution and its interaction with the gravity stresses below the slope surface, it is not possible to determine an ineffective height by simple analysis. The assumption of basing the ineffective depth on the depth of the slope surface below the wall toe at a distance of 3*B* from the wall face or pile centre is likely to err on the conservative side.

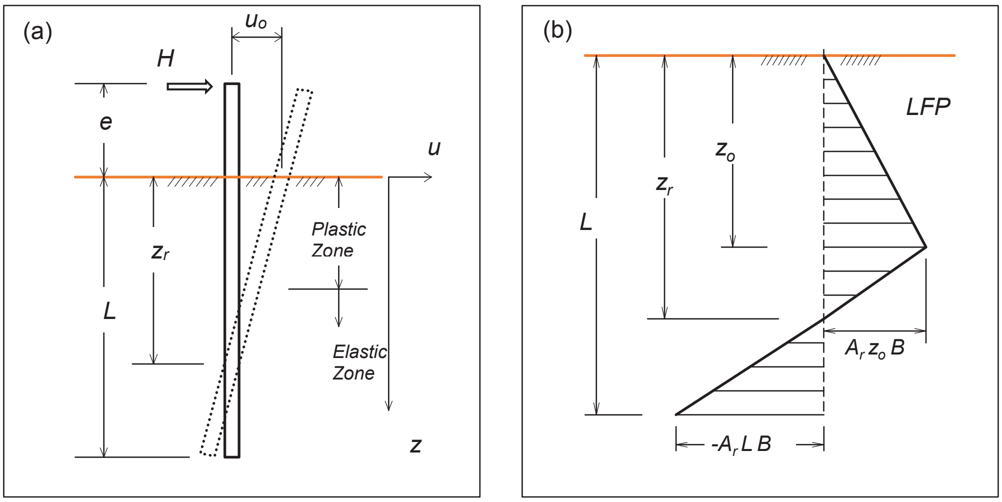



**FIGURE 8.** Guo, 2008 analysis. (a) Displacement profile. (b) Limiting force profile at yield of soil at the pile toe.

## 4. PILE LATERAL LOAD CAPACITY ON SLOPING GROUND - COHESIONLESS SOIL

The Guo, 2008 method for calculating the lateral load capacity of rigid piles located in cohesionless soil on level ground can be modified to give an estimate of the reduced capacity when the pile is located at the crest of a downslope. Figure 8 summarises the level ground analysis method.

The soil pressure forces acting on the pile at the point when the horizontal load is sufficient to initiate soil yield at the toe of the pile can be separated into three component; a plastic force zone from $z$ = 0 down to $z_o$, an elastic force zone between $z = z_o$ and $z_r$, and an elastic force zone below the rotation point depth of $z_r$.

The Guo, 2008 method was modified for the sloping ground case using the following two procedures.

a) Increasing the eccentricity $e$ by the ineffective soil depth of $\Delta z$ to give an effective eccentricity $e_e$ and reducing the embedded length $L$ by $\Delta z$ to give an effective length $L_e$. The lateral load capacity at toe soil yield was then calculated using (Equation 38 from Wood, 2021):

$$H_{yd} = H_y/\gamma' B L_e^2 K_p^2 = -0.046(e_e/L_e)^3 + 0.1236(e_e/L_e)^2 - 0.1447(e_e/L_e) + 0.1126 \quad (5)$$

As shown in Figures 3 to 6 the gravity stresses near the wall face and under the backfill region are increased significantly by the weight of the wall backfill. Below the rotation point the pile is displacing towards the backfill into the higher gravity stress region resulting in an increase in the limiting pressures on the pile below the rotation point. $H_{yd}$ computed using Equation (5) was increased by an enhancement factor, $R_e$ of 1.2 to adjust for this effect. The magnitude of the factor was determined by the more detailed analysis method described in the next paragraph.

b) The force equilibrium and moment equilibrium about the pile toe describing the soil pressures shown in Figure 8 (b) can be written as (Guo, 2008 Equations [A1] and [A2]):

$$H - \int_0^{z_o} A_r B\, z dz - \int_{z_o}^{z_r} k_o\, Bz(z\omega + u_o)dz + \int_{z_r}^{L} k_o Bz(z\omega + u_o)dz = 0 \quad (6)$$

$$H(e+L) - \int_0^{z_o} A_r B(L-z)\, z dz - \int_{z_o}^{z_r} k_o\, Bz(z\omega + u_o)(L-z)dz + \int_{z_r}^{L} k_o Bz(z\omega + u_o)(L-z)dz = 0 \quad (7)$$

Where $k_o$ is the modulus of subgrade reaction [$FL^{-4}$] for a modulus increasing linearly with depth. $u_o$ is the ground level displacement and $\omega$ the pile rotation. Simplifying the notation, Equations 6 and 7 can be written as:

$$H - F_1 - F_2 + F_3 = 0 \quad (8)$$

$$M - M_1 - M_2 + M_3 = 0 \quad (9)$$

Forces $F_1$ and $F_2$ were calculated assuming no pressure force on the pile over the ineffective dept of $\Delta z$. A factor of 1.5 was applied to $F_3$ to allow for the increase in gravity stress below the rotation point. Corresponding adjustments were made to $M_1$, $M_2$ and $M_3$. Equations 6 and 7 were integrated directly and then solved simultaneously by iteration (Solver in Excel) and by applying the relationship (Guo, 2008 Equations 8 and 9):

$$u^* = u_o + \omega z_o = A_r/k_o \quad (10)$$

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Where $u^*$ is the threshold displacement at which soil yield occurs ($u^*$ is negative at the pile toe.)

Results from the two analysis procedures are shown in Figure 9. The analyses were carried out for $e_e/L_e$ ratio of between 0 and 0.5 and the plotted results are for an average over this range. The toe yield capacities were found to be relatively insensitive to the $e_e/L_e$ ratio. Results from the ineffective depth analysis are shown for values of $R_e$ = 1 and 1.2. A factor of 1.2 is recommended for design since using this factor gives a good fit to results from the more correct analysis using integration of the Guo, 2008 LFP forces and moments.

Also shown in Figure 9 are slope reduction factors calculated using equations given in Reese et al, 2006. The method is based on earlier work by Reese et al, 1974 who computed the ultimate lateral resistance of the piles near the ground surface by assuming a passive wedge failure in the soil. The Reese method was used to derive *p-y* curves for more detailed analysis of long piles and was verified using results from pile tests in clean fine to silty sands. The Reese ultimate wedge failure method gives ultimate capacities for *p-y* curves as a function of *z/B*. For the results shown in Figure 9 the ultimate capacities were averaged over a *z/B* range of 0.2 to 1.0. Results from Reese et al, 2006 which assume level ground beyond the crest of the slope, are in reasonable agreement with the ineffective depth method with $R_e$ = 1. This value of $R_e$ is appropriate in applications where there is no wall at the crest of the slope increasing the gravity stresses.

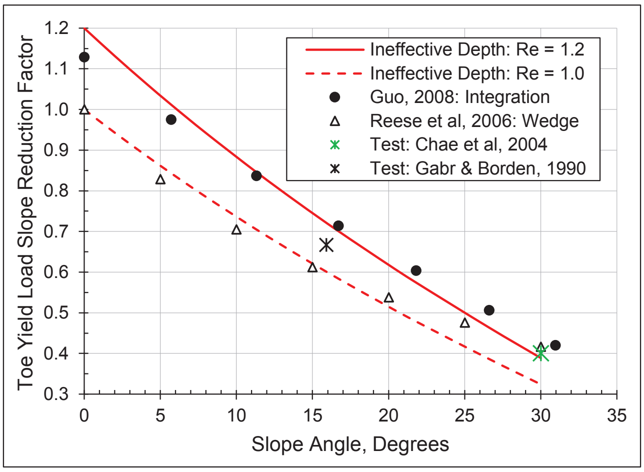


**FIGURE 9. Comparison of toe yield load slope reduction factor for investigated analysis methods.**

A number of published papers on pile lateral load tests on cohesionless slopes were sourced but only two of these were for rigid piles. Results from the Gabr and Boden, 1990 and Chae et al, 2004 tests are plotted in Figure 9 and are in reasonable agreement with the analytical methods. The Chae et al tests were carried out using smooth aluminium model piles with a diameter of 100 mm, an embedded length of 500 mm and a load eccentricity of 100 mm. The test slope was 30° and was formed from a dense sand with a peak friction angle of 47.5°. The Gabr and Borden test was carried out on a concrete pile installed to support a light standard at the crest of a 15.9° slope adjacent to a highway. The pile had a diameter of 0.76 m and an embedment depth 2.13 m. The soil was a fine-to-coarse silty sand with a friction angle of approximately 40°.

There was level ground beyond the crest in both the Chae et al and Gabr and Boden tests carried out for piles located on the slope crest. The test results would therefore be expected to lie closer to the ineffective depth $R_e$ = 1.0 curve than the $R_e$ = 1.2 curve which was derived for the case of a wall at the crest and the associated increase in gravity stresses.

For comparison with the reduction factors shown in Figure 9 for the test on short rigid piles; Baker, 2012 reported a load reduction factor of 0.7 for a long pile (*L/B* of 30) located at the crest of a 26.6° slope and Mirzoyan, 2007 a reduction factor of 0.77 for a long (*L/B* of approximately 42) located at the crest of a 30° slope. These reduction factors are significantly less than indicated by the tests and analyses for short rigid piles.

The reduction factors for the ineffective depth and Guo, 2008 integration methods shown in Figure 9 were calculated assuming a 0.5 m diameter pile with an embedded length of 3 m. They are plotted versus the slope angle since this enabled the reduction factors to be compared directly with the Reese et al, 2006 analysis and the pile test results. The pile length and diameter were typical values that might be used for a pole wall near the crest of a slope. However, to eliminate the influence of the assumed pile dimensions on the results a better approach for design is to use the slope angle and pile diameter *B* to determine the ineffective depth $\Delta z$ as shown in Figure 7 then to use Figure 10 which shows the reduction factor plotted as a function of $\Delta z/L$. (Alternatively, Equation 5 can be used with $H_y$ increased by a $R_e$ factor of 1.2.)

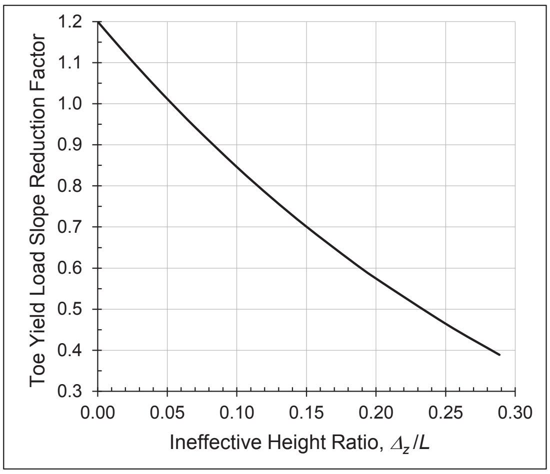


**FIGURE 10. Toe yield load slope reduction factor based on $\Delta z/L$.**

## 5. PILE LATERAL LOAD CAPACITY – COHESIVE SOIL WITH VARYING BEARING CAPACITY FACTOR

Wood, 2021 summarized the methods most commonly used to determine the ultimate lateral load capacity, $H_u$ for a short rigid pile assuming level ground and a cohesive soil. These methods, including the method by Broms, 1964b, were based on assuming simplified distributions of the limiting soil reaction along the pile length. Wood recommended for design that the variation of the reaction near the surface could be approximated by assuming a depth of zero soil resistance and that below this depth the limiting soil reaction was uniform. These assumptions were similar to those adopted by Broms, 1964b but the depth of zero resistance was taken as 0.1 to 0.2 times the embedment length, *L* rather than 1.5 times the pile diameter, *B*.

Georgiadis et al, 2013 developed analytical equations to determine the undrained lateral bearing capacity of rigid piles in cohesive soil. Piles placed in level ground and piles placed at a distance from the crest of a slope were examined, taking account of the effect of the adhesion at the pile-soil interface. Their analytical equations give the lateral load pile capacity as a function of the pile length/diameter ratio, the pile-soil adhesion, the distance of the point of load application from the ground to the pile diameter ratio. For piles near sloping ground, the inclination of the slope and the ratio of the distance of the pile from the crest of the slope to the pile diameter were parameters in the capacity equations.

The ultimate lateral soil reaction per unit pile length can be expressed as:

$$P_u = N_p \, s_u \, B \qquad (11)$$

Where $N_p$ is a bearing capacity factor which varies with depth, $s_u$ is the undrained shear strength and *B* the pile diameter.

Georgiadis et al, 2013 examined a number of $N_p$ versus depth below ground, *z* relationships obtained experimentally, analytically, or numerically including those of Stevens and Audibert, 1980; Murff and Hamilton, 1993; Matlock, 1970; Reese and Welch, 1975; Bhushan et al., 1979; Reese et al., 1975; Broms, 1964; Hansen, 1961; Jeanjean, 2009; and Georgiadis & Georgiadis, 2010. Plots of some of these relationships are presented in Figure 11 for piles located in level ground.

There is large scatter in the $N_p$ versus depth relationships plotted in Figure 11. Georgiadis and Georgiadis, 2010 indicated that the dependence of $P_u$ on the pile-soil adhesion is very significant and one of the main reasons for the scatter. Based on the results of 3-D finite element analyses and comparisons to pile load test results, they proposed the following $N_p$ versus *z/B* relationship for piles located at the crest of a slope:

$$N_p = N_{pu} - \left(N_{pu} - N_{po}\cos\theta\right)e^{-\lambda(z/B)/(1+\tan\theta)} \qquad (12)$$

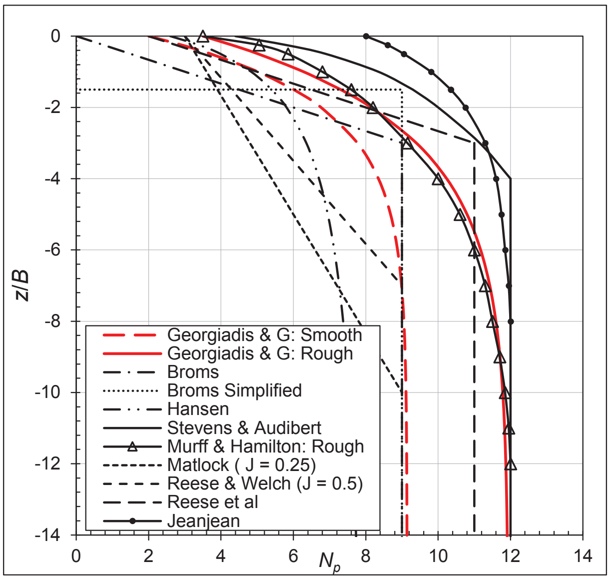


**FIGURE 11. Variation of lateral bearing capacity factor with depth. From Georgiadis et al, 2013.**

Where $\theta$ is the ground slope measured from the horizontal, $\lambda = 0.55 - 0.15\,\alpha$ ; $\alpha$ is the pile-soil adhesion factor that ranges from 0 for a smooth pile to 1.0 for a rough pile. $N_{pu}$ is the ultimate bearing capacity factor for deep lateral flow of soil, $N_{po}$ = 2 + 1.5$\alpha$ is the bearing capacity factor at the ground surface.

To study the effect of the distance between the pile centre and the crest of a clay slope, on the lateral pile behaviour, Georgiadis et al, 2013 performed a detailed 3-D finite element study which led to the modification of Equation 12. The resulting variation in $N_p$ with *z/B* is equal to the bearing capacity factor for horizontal ground up to critical depth $z_c$. Below this critical depth, $N_p$ is determined by the following equation:

$$N_p = N_{pu} - \left(N_{pu} - N_{pc}\right)e^{-\frac{\lambda\alpha_\theta(z-z_c)}{B}} \qquad (13)$$

Where $\alpha_\theta$ is a factor depending on the inclination of the slope and is given by:

$$\alpha_\theta = 1 - \sin\theta(1+\sin\theta)/2 \qquad (14)$$

$z_c$ is the critical depth which depends on the distance between the pile and the crest of the slope, *b* and is given by:

$$z_c/B = 8.8 - 10\log_{10}(8 - b/B) \qquad (15)$$

$N_{pc}$ is the bearing capacity factor at the critical depth, obtained from Equation 12 for $z = z_c$ and $\theta$ = 0.

The Georgiadis et al, 2013 bearing capacity factor for level ground is given by Equation (12) with $\theta$ = 0.

The analysis lateral earth pressure definition diagram

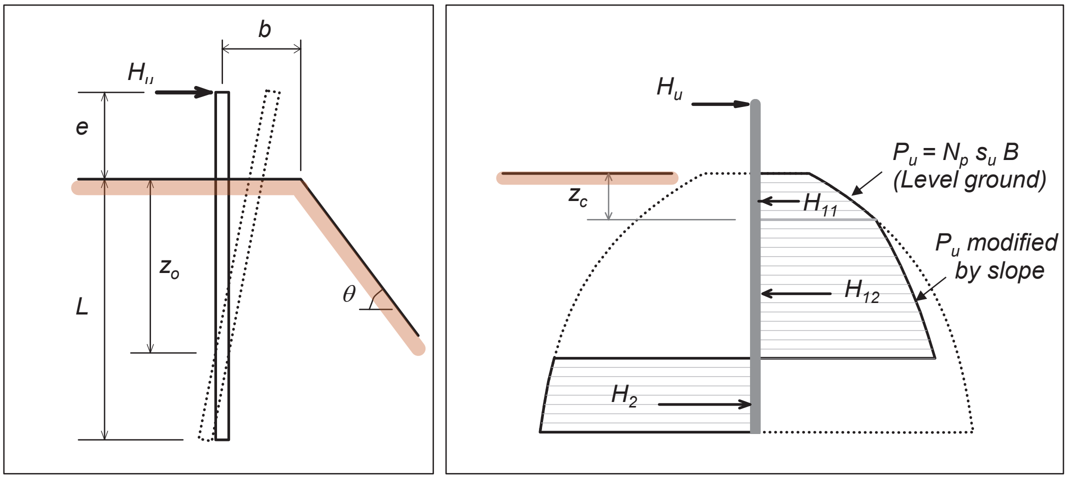


**FIGURE 12.** Ultimate lateral earth pressure on piles near sloping ground. From Georgiadis et al, 2013.

given in Georgiadis et al, 2013 is shown in Figure 12. The horizontal force and bending moment equilibrium equations contain two unknowns; the rotation depth $z_o$ and the ultimate lateral load capacity $H_u$.

The two equilibrium equations are:

$$H_u = \int_0^{z_c} p_u\,dz + \int_{z_c}^{z_o} p_u\,dz - \int_{z_o}^{L} p_u\,dz \tag{16}$$

$$eH_u = -\int_0^{z_c} zp_u\,dz - \int_{z_c}^{z_o} zp_u\,dz + \int_{z_c}^{L} zp_u\,dz \tag{17}$$

The solution of these two simultaneous equations gives:

$$\alpha_1 z_d^2 + \alpha_2 z_d + \alpha_3 e^{-\lambda z_d} + \alpha_4 z_d e^{-\lambda z_d} + \alpha_5 e^{-\lambda\alpha_\theta z_d} + \alpha_6 z_d e^{-\lambda\alpha_\theta z_d} + \alpha_7 = 0 \tag{18}$$

Where $z_d = z_0/B$ and the coefficients $\alpha_1$ to $\alpha_7$ are functions of $N_{pu}$, $N_{po}$, $N_{pc}$, $B$, $e$, $\lambda$, $\alpha_\theta$, $L$ and $z_c$ and are given in the Appendix.

The horizontal force equilibrium Equation 16 for the $z_d$ value derived from the solution of Equation 18 (by iteration) gives the dimensionless lateral force capacity as:

$$H_u/s_u B^2 = c_1 z_d^2 + (c_2 + c_3 z_d)e^{-\lambda z_d} + (c_4 + c_5 z_d)e^{-\lambda\alpha_\theta z_d} + c_6 \tag{19}$$

Where the coefficients $c_1$ to $c_6$ are functions of $N_{pu}$, $N_{po}$, $N_{pc}$, $B$, $e$, $\lambda$ $\alpha_\theta$, $L$ and $z_c$ and are given in the Appendix.

There is no slope effect on the lateral load capacity of the pile when the depth of rotation derived from Equation 18 is less than the critical depth $z_c$. In this case the solution equations for $z_d$ and $H_u$ are simplified.

However, for typical pole wall foundations it is very unlikely that $z_o$ will be less than the critical depth. For example, for a $L/B$ ratio between 4 and 6 and for $b/B$ between 0 to 4 the $z_c/L$ ratio is less than 0.62 and the $z_o/L$ ratio greater than 0.68 for slopes from 0 to 30°.

Zhang and Ahmari 2013 presented a method for the nonlinear analysis of laterally loaded rigid piles in a cohesive soil on level ground. As with the Georgiadis et al, 2013 method, a nonlinear variation of lateral resistance with depth was considered. The Zhang and Ahamari, 2013 method extended the Georgiadis et al, 2013 approach by considering the nonlinearity of the modulus of subgrade reaction with the pile displacement at ground level. The modulus of subgrade reaction was assumed to be constant with depth.

Zhang and Ahmari, 2013 assumed that the bearing capacity factor was given by:

$$N_p = C_1 + C_2\left(\frac{z}{B}\right)^{C_3} \tag{20}$$

Where $C_1 = 2.5, C_2 = \frac{\gamma' B}{s_u} + 5.5$ and $C_3 = 0.1$

A comparison of the Equation 20 relationship with the level ground bearing factor relationship used by Georgiadis et al, 2013 is shown in Figure 13.

The $N_p$ variation with depth used by Zhang and Ahmari, 2013 is clearly very different from that proposed by Georgiadis et al, 2013. Before deciding on the best method of analysis for piles located at the crest of sloping ground, a comparison of the ultimate and toe yield lateral load capacities obtained by using the Zhang and Ahmari, 2013, Georgiadis et al, 2013 and the recommended methods given in Wood 2021 (based on Motta, 2013 and Zhang, 2018) for level ground was carried out.


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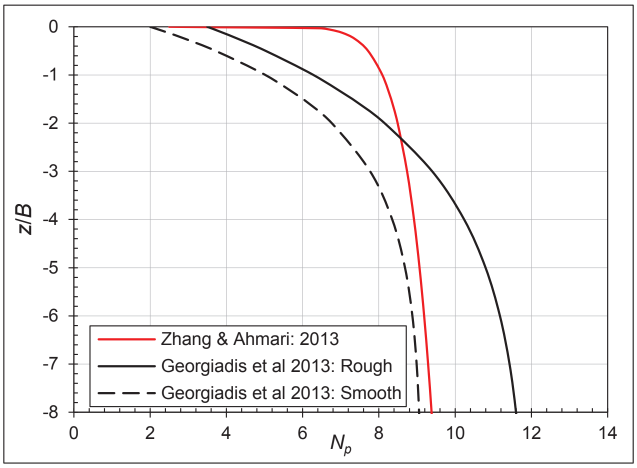


**FIGURE 13. Variation of lateral bearing capacity factor with depth.**

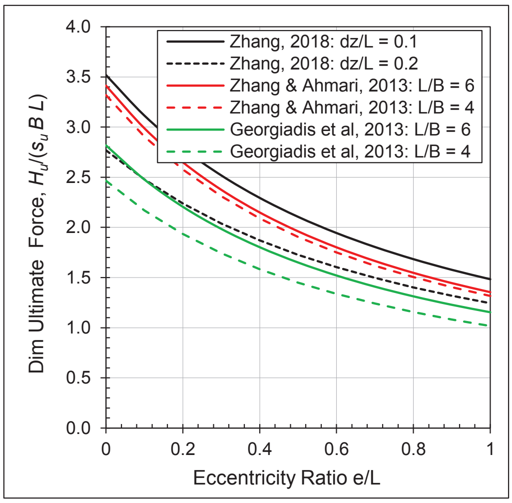


**FIGURE 15. Comparison of ultimate capacities.**

Comparisons of capacities calculated by the three methods for the toe yield and ultimate load cases are shown in Figures 14 and 15 for *e/L* ratios of 0 to 1.0. The Motta, 2013 (toe yield) and Zhang, 2018 (ultimate) capacities are based on a $N_p$ value of 10.8 assumed constant with depth below an upper soil layer assumed to be ineffective with an $N_p$ of zero. An $N_p$ of 10.8 assumes that the pile is intermediate between smooth and rough (deep failure $N_p$ values of 9.14 and 11.94 respectively). The Georgiadis et al, 2013 capacities are also based on a pile with intermediate roughness using $\alpha$ = 0.5. The Motta, 2013 and Zhang, 2018 capacities are shown for ineffective depth values of $\Delta z/L$ = 0.1 and 0.2. Because of the variation of $N_p$ with depth assumed in the Zhang and Ahmari, 2013 and Georgiadis et al, 2013 methods, the capacities are a function of the *L/B* ratio although the capacities are not very sensitive to this parameter. Capacities are shown for *L/B* values of 4 and 6 which covers the typical range for cantilever pole walls.

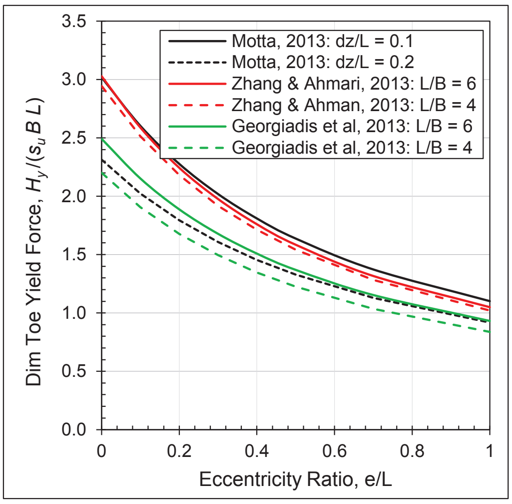


**FIGURE 14. Comparison of toe yield capacities.**

The toe yield capacity is recommended for design (Wood, 2021) and this capacity is about 15 to 20% lower than the ultimate capacity for all the methods. Figure 14 shows that the toe yield capacity from Georgiadis et al, 2013 agrees reasonably closely with the Motta, 2013 value for $\Delta z/L$ =0.2 and is about 20% to 15% lower than the capacities given by Zhang and Ahmari, 2013 for *L/B* values of 4 and 6 respectively.

Zhang and Ahmari, 2013 show good agreement between their force versus displacement predictions and a number of test results. Georgiadis et al, 2013 show good agreement between their ultimate capacity predictions and test results.

## 6. PILE TOP DEFLECTIONS IN COHESIVE SOIL

From the comparisons made above it appears that the Zhang and Ahmari, 2013 method may give unconservative lateral load capacities. However, Zhang and Ahmari, 2013 provide Young's and subgrade modulus values for clay that can be used for estimating the pile top deflection. Table 1 lists the Young's modulus values that they recommend for clays.

**Table 1. Elastic Modulus for Clays.**

| Clay Consistency | Shear Strength, $s_u$ kPa | Young's Modulus, $E_s$ MPa |
|---|---|---|
| Very soft | 2-25 | 0.5-5 |
| Soft | 25-50 | 5-20 |
| Medium | 50-100 | 20-50 |
| Stiff | 100-200 | 50-100 |

The low strain modulus of subgrade reaction can be estimated from:

$$k_{hmax} = \frac{22.4E_s(1-\nu)}{(1+\nu)(3-4\nu)(2\ln(2L/B)-0.443} \qquad (21)$$

Where $L$ and $B$ are the pile length and diameter respectively and $E_s$ and $\upsilon$ the Young's modulus and Poisson's ratio respectively of the soil. Zhang and Ahmari, 2013 indicate that the relationship in Equation 21 is affected by the fixity of the pile head.

The variation of $k_h$ with the pile ground surface displacement $u_o$ can be represented by (Zhang and Ahmari, 2013):

$$\frac{k_h}{k_{hmax}} = 0.058 \left(\frac{u_o}{B}\right)^{-0.5} \qquad (22)$$

Dimensionless ground level displacements ($u_o\, k_h / s_u\, B$) resulting from the lateral load that initiates yield in the soil at the pile toe calculated using Georgiadis et al, 2013, Zhang & Ahmari, 2013, and Motta, 2013 are compared in Figure 16. Although all three methods are based on quite different $N_p$ variations with depth, they give ground level displacements that agree within about 25%. All three methods are based on the assumption that the modulus of subgrade reaction $k_h$ is constant with depth. The deflection analyses for Georgiadis et al, 2013 and Motta, 2013 assume that the modulus of subgrade reaction varies linearly with displacement up to the soil yield level (elastic-plastic response).

In the Motta, 2013 analysis no reduction in the modulus was made for the upper soil layer. The Georgiadis et al, 2013 and Zhang and Ahmari, 2013 displacements were calculated for $L/B$ ratios of 4 and 6. Small differences in displacements occur over this typical range for pole walls because of nonlinear bearing capacity variations with depth.

The ground level displacement at toe yield can be estimated by an iterative procedure using Figure 16 together with Equation 22. Adopting this approach an estimate of $k_h$ (based on Equation 21) allows the actual displacement to be estimated from the dimensionless value shown in Figure 16. $k_h$ can then be revised using Equation 22.

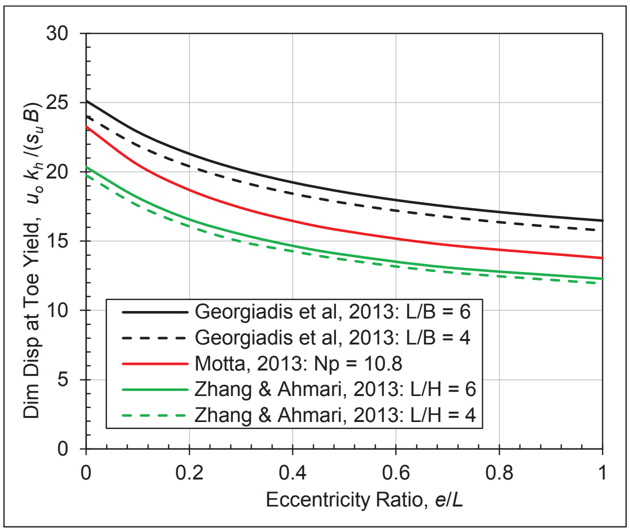


**FIGURE 16.** Ground level displacement at toe yield load.

## 7. REDUCTION IN LATERAL LOAD CAPACITY ON SLOPING GROUND - COHESIVE SOIL

Equations 13 to 19 from Geogiadis et al, 2013 were used to calculate the lateral load capacity reduction from the level ground capacities for poles located at the crest of a slope ($b$ = 0) and for poles located on level ground at a distance of $b = 2B$ from the crest. The pile adhesion factor $\alpha$ was taken as 0.5. Analyses were carried out for eccentricity ratios $e/L$ of 0 and 0.5, $L/B$ ratios of 4 and 6 and for slopes varying from 0° to 30° (measured from the horizontal). The reduction factor $R_f$ = lateral capacity on sloping ground capacity/lateral capacity on level ground is shown in Figures 17 and 18 for $b$ = 0 and $b = 2B$ respectively.

For typical $e/L$ and $B/L$ ratios used in pole walls Figure 18 shows that the lateral load capacity reductions from the level ground case are less than 5% at distances of $2B$ or greater from the crest for slopes up to 30°. These capacity reductions are small and this is partly due to the relatively small pole embedment depth. Even with the piles at the crest of the slope, reductions in lateral capacity are less than 25% as indicated by Figure 17.

Stewart, 1999 used an upper bound plasticity method to estimate the undrained collapse load of laterally loaded piles in clay soils. Figure 19 summarises the analysis assumptions. It was assumed that a deforming conical wedge of soil forms near the surface, and below the base of the wedge the soil is assumed to flow horizontally around the pile. It was also assumed that a gap develops behind the pile above the base of the wedge. The pile was allowed to rotate about a point that is located below the base of the wedge. Resistance to lateral displacement and rotation of the pile is assumed to be comprised of energy dissipation due to:

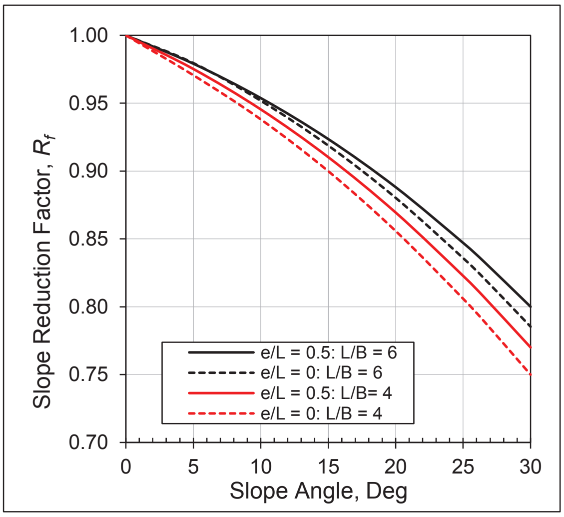


**FIGURE 17.** Slope reduction factors for $b$ = 0.

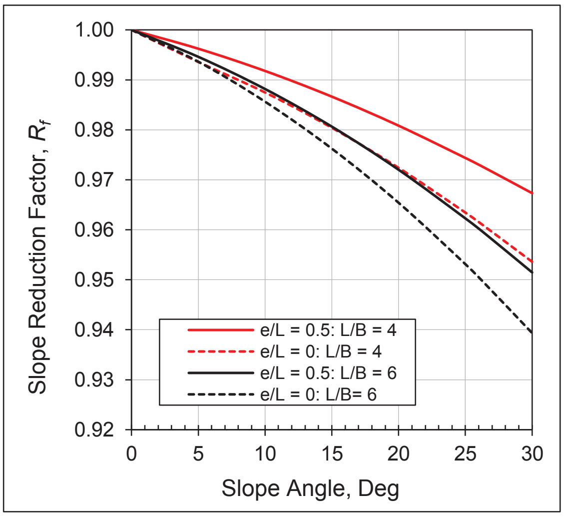


**FIGURE 18. Slope reduction factors for b = 2*B*.**

1. Deformation of soil within the wedge
2. Work done by the soil weight within the deforming wedge as it moves upward
3. Shear along the wedge-soil interface
4. Shear along the pile-soil interface as the wedge moves upwards
5. Flow of soil around the pile below the base of the wedge with a limit pressure of 9 su
6. Shear of soil over the base of the pile

The numbers in the above list refer to the components shown on Figure 19.

The effect of a slope adjacent to the pile was incorporated by truncating part of the assumed collapse mechanism (Figure 19). Stewart concluded that the slope reduction factor depends largely on the distance of the pile from the crest of the slope, the slope gradient, and the length to diameter ratio. Stewart indicated that for the case of level ground, the method yielded results that were similar to those from Broms, 1964b.

Stewart's results for the pile located at the crest of the slope and with the load eccentricity ratio, $e/L$ = 0 are compared in Figure 20 with those of Georgiadis et al 2013 for an $L/B$ ratio of 4. Limited results were given by Stewart for the range of piles relevant to pole walls. Only one result for an $L/B$ ratio of 6 was given and this is plotted on Figure 20. The Stewart results are for a smooth pile and the Georgiadis et al, 2013 results shown in Figure 20 are also for this assumption.

Also plotted in Figure 20 is a reduction factor, $R_f$ given by:

$$R_f = 1/(1 + \tan\theta) \tag{23}$$

The Equation 23 slope reduction factor is given in the Technical Manual for the LPile software (Isenhower and Wang, 2013). This manual references Reese, 1958 who developed an equation for the ultimate lateral resistance of a pile in a clay soil assuming a simple triangular shaped wedge failure near the ground surface. Reese, 1958 did not consider a sloped soil surface but apparently Equation 23 comes from a geometric modification to the wedge assumed by Reese.

Published test results for lateral loading of a rigid pile on slopes are limited. Bushan et al, 1979 carried out a single lateral test for a 5.18 m long drilled pile with a 1.22 m diameter ($L/B$ = 4.2) and load eccentricity of 0.23 m ($e/L$ = 0.044) on the crest of a firm clay soil 20° slope. The soil shear strength and measured ultimate capacity as reported by Georgiadis et al. 2013 were 220 kPa and 2,180 kN respectively. The predicted lateral load capacity using the Georgiadis et al, 2013 equations and assuming $\alpha$ = 0.24 (computed from $s_u$ using Georgiadis et al, 2013) was 2,705 kN; a 24% overprediction of the measured capacity. For the test pile the Georgiadis et al, 2013 method gave a slope reduction factor of 0.86 which is in agreement with the value estimated from Figure 17.

For design of pole walls near the crest of a slope the Stewart $L/B$ = 4 curve gives satisfactory reduction

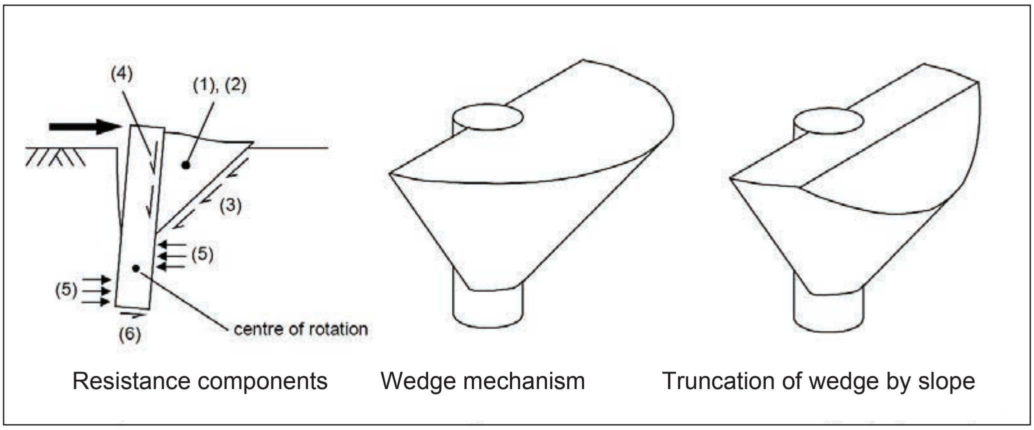


**FIGURE 19. Collapse mechanism for upper bound plasticity method. From Stewart 1999.**


PAGANI

factors. Georgiadis et al, 2013 may underpredict the reduction and Equation 23 will give a conservative lower bound prediction. For a 14° slope and a *L/B* of 8 Stewart gave reduction factors of approximately 0.94 and 0.97 for *b/B* values of 1 and 2 respectively indicating that the slope reduction factor diminishes quite rapidly as *b* is increased from zero. This is in agreement with the Georgiadis et al, 2013 reductions shown in Figure 18 for *b/B* = 2.

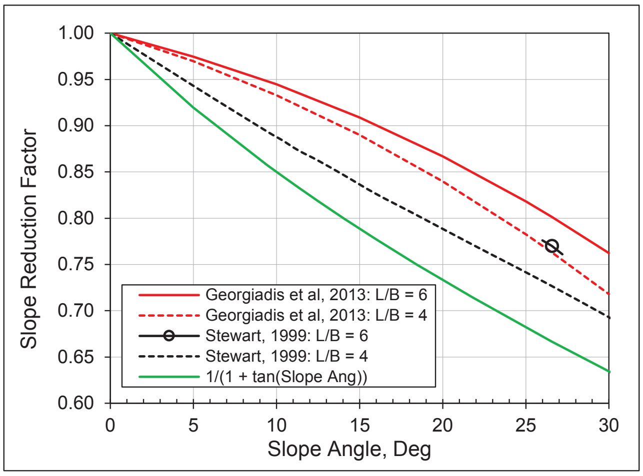


**FIGURE 20. Comparison of slope reduction factors from Stewart, 1999 and Georgiadis et al, 2013. (*e/L* = 0).**

## 8. SLOPE EFFECTS ON DEFLECTIONS AND BENDING MOMENTS

There is little published information on the effect of slopes on the deflections of laterally loaded piles. Georgiadis and Georgiadis, 2010 plot load versus displacement curves in an analytical parametric study of 1.0 m diameter piles with embedded lengths of 5 m, 10 m, and 20 m and laterally loaded at ground level. The piles were located in level ground and on the crest of 20° and 40° slopes in cohesive soil and were assumed to have a roughness factor, $\alpha$ = 0.5. At yield load levels the ground level displacement of the 5 m long pile (*L/B* = 5) at the crest of a 20° slope was approximately 50% greater than the pile in level ground. At the ultimate load the displacement was approximately 200% greater than on level ground. (These approximate increases were estimated from plotted curves.)

Experimental and analytical studies by Chae et al show that the maximum bending moment in a laterally loaded pile at the crest of a 30° slope in cohesionless soil is approximately 15% greater than in a pile located at a distance of 4 diameters from the crest. The parametric study results of Georgiadis and Georgiadis, 2010 for the 1.0 m diameter and 5 m long pile in cohesive soil referred to above show a small increase in bending moment from the level ground case to the 20° slope. The increase cannot be easily read from the plotted curves but it is estimated to be less than 5%.

## 9. CONCLUSION

Down sloping ground in front of a cantilever pole wall can significantly reduce the lateral load capacity of the pole foundations. For a cohesionless soil with a 30° down slope immediately in front of the wall, the reduction in capacity from a level ground case is a factor of approximately 0.4. The reduction for the same slope in an undrained cohesive soil is approximately 0.7. There is very limited test information to support these theoretical reductions. For design, judgement needs to be exercised in applying the slope reduction factors particularly for the cohesive soil case where a larger reduction (based on a lower bound curve) might be applied for walls of high importance. There is no published information on slope effects in a *c*-$\phi$ soil in the wall foundation. For these soils a reduction based on the pure cohesionless and cohesive cases could be used by considering the relative contribution of the friction and cohesion components in the total shear strength.

In cohesive soils lateral load analyses making allowance for a variation in the bearing capacity factor with depth involves the evaluation by iteration of rather complex equations derived from simultaneous solution of the equilibrium equations. Charts have been developed in the present study for both the toe soil yield capacity and pole ground level displacement based on three different assumptions regarding the depth variation of the bearing capacity factor. These provide sufficiently detailed information for design applications.

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## APPENDIX:
## EVALUATION OF GEORGIADIS ET AL, 2013 EQUATIONS FOR LATERAL LOAD CAPACITY OF PILE NEAR A SLOPE

Solution of the two simultaneous equilibrium equations for lateral earth pressure on piles near sloping ground results in Equation 18 in the text which is repeated here:

$$\alpha_1 z_d^2 + \alpha_2 z_d + \alpha_3 e^{-\lambda z_d} + \alpha_4 z_d e^{-\lambda z_d} + \alpha_5 e^{-\lambda \alpha_\theta z_d} + \alpha_6 z_d e^{-\lambda \alpha_\theta z_d} + \alpha_7 = 0 \quad (18)$$

The coefficients in this equation are defined as:

$$\alpha_1 = N_{pu},\ \ \alpha_2 = 2N_{pu}\, e/B\,,\ \ \alpha_3 = (N_{pu} - N_{po})(1/\lambda + e/B)/\lambda,\ \ \alpha_4 = (N_{pu} - N_{po})/\lambda$$

$$\alpha_5 = \frac{N_{pu} - N_{pc}}{\lambda \alpha_\theta}\left(\frac{1}{\lambda \alpha_\theta} + \frac{e}{B}\right) e^{\lambda \alpha_\theta (z_c/B)},\ \ \alpha_6 = \frac{N_{pu} - N_{pc}}{\lambda \alpha_\theta} e^{\lambda \alpha_\theta (z_c/B)}$$

$$\alpha_7 = -N_{pu}\left(\frac{L^2}{2B^2} + \frac{Le}{B^2}\right) - \frac{N_{pu} - N_{po}}{\lambda}\left(\frac{1}{\lambda} + \frac{e}{B}\right) - \frac{N_{pu} - N_{po}}{\lambda}\left(\frac{L}{B} + \frac{1}{\lambda} + \frac{e}{B}\right) e^{-\lambda (L/B)}$$

$$- \frac{N_{pu} - N_{pc}}{\lambda \alpha_\theta}\left(\frac{z_c}{B} + \frac{1}{\lambda \alpha_\theta} + \frac{e}{B}\right) + \frac{N_{pu} - N_{po}}{\lambda}\left(\frac{z_c}{B} + \frac{1}{\lambda} + \frac{e}{B}\right) e^{-\lambda (z_c/B)}$$

Coefficients $\alpha_1$ to $\alpha_7$ are functions of the known parameters; $N_{pu}$, $N_{po}$, $N_{pc}$, $B$, $L$, $e$, $\lambda$, $\alpha_\theta$, and $z_c$
The only unknown factor in Equation 18 is $z_d = z_o/L$. Thus, the equation can be solved by iteration to determine this factor and the rotation depth $z_o$.

The dimensionless lateral force capacity of the pile is given by Equation (19) in the text which is repeated here:

$$H_u/s_u\, B^2 = c_1 z_d^2 + (c_2 + c_3 z_d) e^{-\lambda z_d} + (c_4 + c_5 z_d) e^{-\lambda \alpha_\theta z_d} + c_6 \quad (19)$$

The coefficients in this equation are defined as:

$$c_1 = -N_{pu}(B/e),\ c_2 = -(N_{pu} - N_{po})(B/e)/\lambda^2,\ c_3 = -(N_{pu} - N_{po})(B/e)/\lambda$$

$$c_4 = \frac{N_{pu} - N_{pc}}{\lambda^2 \alpha_\theta^2}\left(\frac{B}{e}\right) e^{\lambda \alpha_\theta (z_c/B)}\,,\ \ c_5 = -\frac{N_{pu} - N_{pc}}{\lambda \alpha_\theta}(B/e) e^{\lambda \alpha_\theta (z_c/B)}$$

$$c_6 = \frac{B}{e}\left[\frac{N_{pu}}{2}\left(\frac{L}{B}\right)^2 + \frac{N_{pu} - N_{po}}{\lambda^2} + \frac{N_{pu} - N_{po}}{\lambda^2}\left(\lambda \frac{L}{B} + 1\right) e^{-\lambda (L/B)} - \frac{N_{pu} - N_{po}}{\lambda^2}\left(\lambda \frac{z_c}{B} + 1\right) e^{-\lambda (z_c/B)} \right.$$

$$\left. + \frac{N_{pu} - N_{pc}}{\lambda \alpha_\theta}\left(\frac{z_c}{B}\right) + \frac{N_{pu} - N_{pc}}{\lambda^2 \alpha_\theta^2}\right]$$

Coefficients $c_1$ to $c_6$ are functions of the known parameters; $N_{pu}$, $N_{po}$, $N_{pc}$, $B$, $L$, $e$, $\lambda$, $\alpha_\theta$, and $z_c$. Therefore, after $z_d$ is evaluated by Equation 18 all parameters in Equation 19 are defined.

As part of the present study Equation 18 was solved for typical pole wall parameters by a spread sheet analysis using the Solver add-in for the iteration. Evaluation of Equation 19 gave agreement with the results for the ultimate lateral load capacities given in Georgiadis et al 2013.

# What's On at Waka Kotahi

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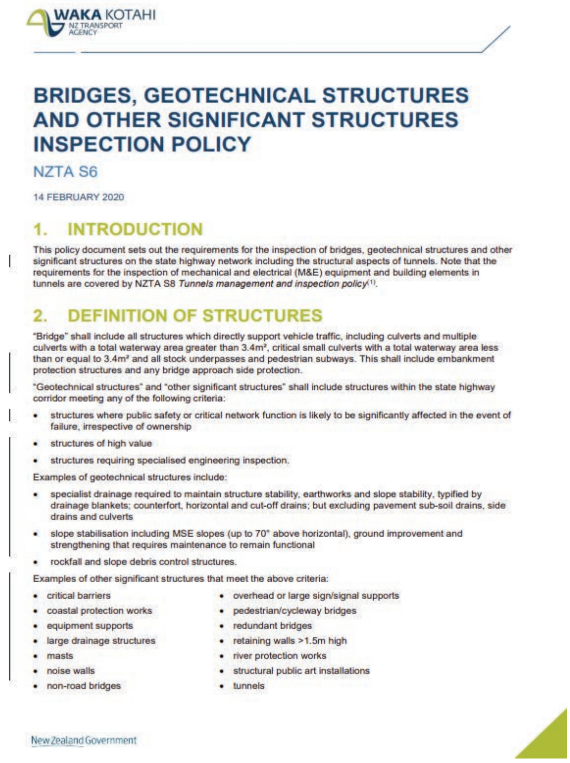
*Comments and opinions expressed in this article are not necessarily those of Waka Kotahi*

**Stuart Finlan**
*Waka Kotahi*



WAKA KOTAHI
NZ TRANSPORT AGENCY

**BRIDGES, GEOTECHNICAL STRUCTURES AND OTHER SIGNIFICANT STRUCTURES INSPECTION POLICY**

NZTA S6

14 FEBRUARY 2020

**1. INTRODUCTION**

This policy document sets out the requirements for the inspection of bridges, geotechnical structures and other significant structures on the state highway network including the structural aspects of tunnels. Note that the requirements for the inspection of mechanical and electrical (M&E) equipment and building elements in tunnels are covered by NZTA S8 *Tunnels management and inspection policy*[1].

**2. DEFINITION OF STRUCTURES**

"Bridge" shall include all structures which directly support vehicle traffic, including culverts and multiple culverts with a total waterway area greater than $3.4m^2$, critical small culverts with a total waterway area less than or equal to $3.4m^2$ and all stock underpasses and pedestrian subways. This shall include embankment protection structures and any bridge approach side protection.

"Geotechnical structures" and "other significant structures" shall include structures within the state highway corridor meeting any of the following criteria:

- structures where public safety or critical network function is likely to be significantly affected in the event of failure, irrespective of ownership
- structures of high value
- structures requiring specialised engineering inspection.

Examples of geotechnical structures include:

- specialist drainage required to maintain structure stability, earthworks and slope stability, typified by drainage blankets; counterfort, horizontal and cut-off drains; but excluding pavement sub-soil drains, side drains and culverts
- slope stabilisation including MSE slopes (up to 70° above horizontal), ground improvement and strengthening that requires maintenance to remain functional
- rockfall and slope debris control structures.

Examples of other significant structures that meet the above criteria:

- critical barriers
- coastal protection works
- equipment supports
- large drainage structures
- masts
- noise walls
- non-road bridges
- overhead or large sign/signal supports
- pedestrian/cycleway bridges
- redundant bridges
- retaining walls >1.5m high
- river protection works
- structural public art installations
- tunnels

New Zealand Government

## WAKA KOTAHI WELCOMES ADDITIONAL SMES

Programme and Standards-Transport Services-System Resilience Team, led by Richard Topham, has welcomed new subject matter experts in the geotechnical and drainage fields. This brings the geotechnical SMEs up to three persons (Dante Legaspi, Sigfrid Dupre and Sivaram Thirumoorthy) with Rajika Jayarantne joining as senior stormwater engineer. A resource increase already snapped up!

## SLOPE ASSESSED RISK LEVEL COURSE(S)

The Waka Kotahi/NZTA ARL slope assessment courses are in the early stages of planning for two courses centred in the North Island in October/November 2022. With border restrictions ending we may finally get the show on the road! Those who were accepted for the first course will retain first right of refusal for that course with the additional course to be provided to enable further participation.

Those seeking more information can contact ARL_Course@nzta.govt.nz

## S6 BRIDGES, GEOTECHNICAL STRUCTURES AND OTHER SIGNIFICANT STRUCTURES INSPECTION POLICY

The latest version of S6 policy was published in February 2020 and introduces geotechnical structures for the first time, to get a copy, google 'NZTA S6'.

Work has started on the geotechnical structures policy (S7) and its associated inspection manual and course to replace the existing geotechnical aspects in S6 (Bridges, geotechnical structures and other significant structures inspection policy).

We are looking for any interesting images of failed geotechnical structures for inclusion in the inspection manual, so if you have any we'd be keen to hear from you: geotechnics@nzta.govt.nz

## GEOTECHNICAL MANUAL: GOOD IDEA?

We had an overwhelming response to a multi-audience survey supporting a 'Geotechnical manual' to sit alongside the Bridge manual. There were some interesting differences in opinion as to the content which we will follow up on. The geotechnical manual is planned to be developed over a number of years (added to) to become the source of geotechnical requirements and guidance. Initially the document would pull together all the current geotechnical aspects covered by many NZTA/WK documents including the State Highway Procurement Manuals, which would be an immediate benefit for practitioners!

If you have any views, again let us know: geotechnics@nzta.govt.nz

## SHARING RESEARCH

Thanks to those who provided/shared historical research done on WK projects. Suspect there's still a lot out there, so if you know of any research that has been done for Transit/NZTA/Waka Kotahi; have a copy under your computer monitor or holding the back door open, or even somewhere on that flash drive, it would be great to receive a copy: geotechnics@nzta.govt.nz

## UNSTABLE SLOPES AND ROCKS!

As part of our initiative to utilise learnings from projects, we are currently developing two guidance documents: Rockfall Protection Systems Maintenance Manual (an area where there is little to no guidance for works completed prior to 2020) and Rockfall Protection Structures Design Guidance (to unify design approaches and methods which currently vary widely between designers with associated varying risk profiles for Waka Kotahi). Both of these documents relate to Waka Kotahi highway works though it is recognised they may be adopted by 'others'. If you represent a local authority or other potential user, we'd be keen to hear from you to ensure we provided a 'rounded' document. Contact either myself or geotechnics@nzta.govt.nz

## WRITING PAPERS ON EXPERIENCE GAINED/ PROBLEMS SOLVED ON WAKA KOTAHI PROJECTS

Please be aware that any paper written for submission to a conference, or any other forum has to be reviewed by Waka Kotahi prior to submission. We simply need to ensure a number of aspects of papers. We are not undertaking critical analysis of the content; simply ensuring the content reflects Waka Kotahi and/or is appropriately caveated. Generally, these reviews are undertaken by the Waka Kotahi technical specialists through the applicable project manager. For geotechnically related papers feel free to contact myself directly if unsure. Be assured Waka Kotahi is keen to promote learning across our projects.

# NZS 4431:2022

## Engineered fill construction for lightweight structures strengthens the foundation for modern buildings

NZS 4431:2022 – Engineered fill construction for lightweight structures is an update of the 1989 standard, with significant changes reflecting over thirty of years of improvements in geotechnical investigation, design and construction practice, test methods and environmental, consenting and health and safety obligations.

The standard plays a fundamental role in ensuring engineered fill construction is delivered safely and provides a stable foundation for lightweight residential and commercial buildings, and their associated infrastructure. Current needs have been factored in including sustainability in design and climate change considerations, planting and revegetation, cultural heritage and archaeology and latest material types.

### FOR GEOTECHNICAL DESIGNERS, CONTRACTORS AND CERTIFIERS

NZS 4431 is for those working across the construction project lifecycle – from the geotechnical designers responsible for the design and specification of earthworks projects, to the contractors responsible for implementing those specified works and finally the certifier responsible for confirming that construction is in accordance with the design, consent conditions, and earthworks specification.

### CONSENSUS-BUILT FOR BROAD PERSPECTIVES

Developed through Standards New Zealand's trusted and internationally-used consensus approach, committee members provided expertise from engineering geologists and geotechnical engineers, design consultancies, councils, civil engineering contractors, heritage, health and safety, regulation and testing, and academia and research.

## THE FOUNDATION FOR COMPLIANCE

Forming the core of Building Code compliant foundations and generously sponsored by the building regulator Building Systems Performance (BSP), NZS 4431 is not a specification, but rather a framework reflecting the iterative processes used by the geotechnical designer, contractor and certifier.

To comply with this standard, certified engineered fill construction needs to meet the requirements of a project- and site-specific earthworks specification, supported by appropriate design documentation, engineering drawings, quality assurance and testing requirements, and building and consent conditions.

## DRIVING CONSISTENCY AND MINIMISING THE LEARNING CURVE

P4431 Committee Chair Ross Roberts, Head of Engineering Resilience for Auckland Council, says, *'NZS4431 is a critical element in the system that ensures buildings in New Zealand are safe. The updated standard brings a balance of appropriate minimum requirements and the flexibility needed to accommodate earthworks across the varied geology of New Zealand.*

*It achieves this by setting out the process to define a robust earthworks specification, with the core elements pre-defined for commonly encountered conditions and the flexibility to amend these where appropriate to suit other site conditions. This standard will be supported by a template earthworks specification, due to be published for consultation by the New Zealand Geotechnical Society in the coming months.*

*The committee's core intent was to drive consistency in earthworks specifications. This should make it easier to comply with those specifications by reducing variability between projects thereby minimising the learning curve – and the number of mistakes and amount of rework – on each project, reducing costs and project delays. Providing guidance on how to develop these specifications, and a template to follow, should encourage good practice, resulting in more robust and sustainable earthworks with fewer failures.'*

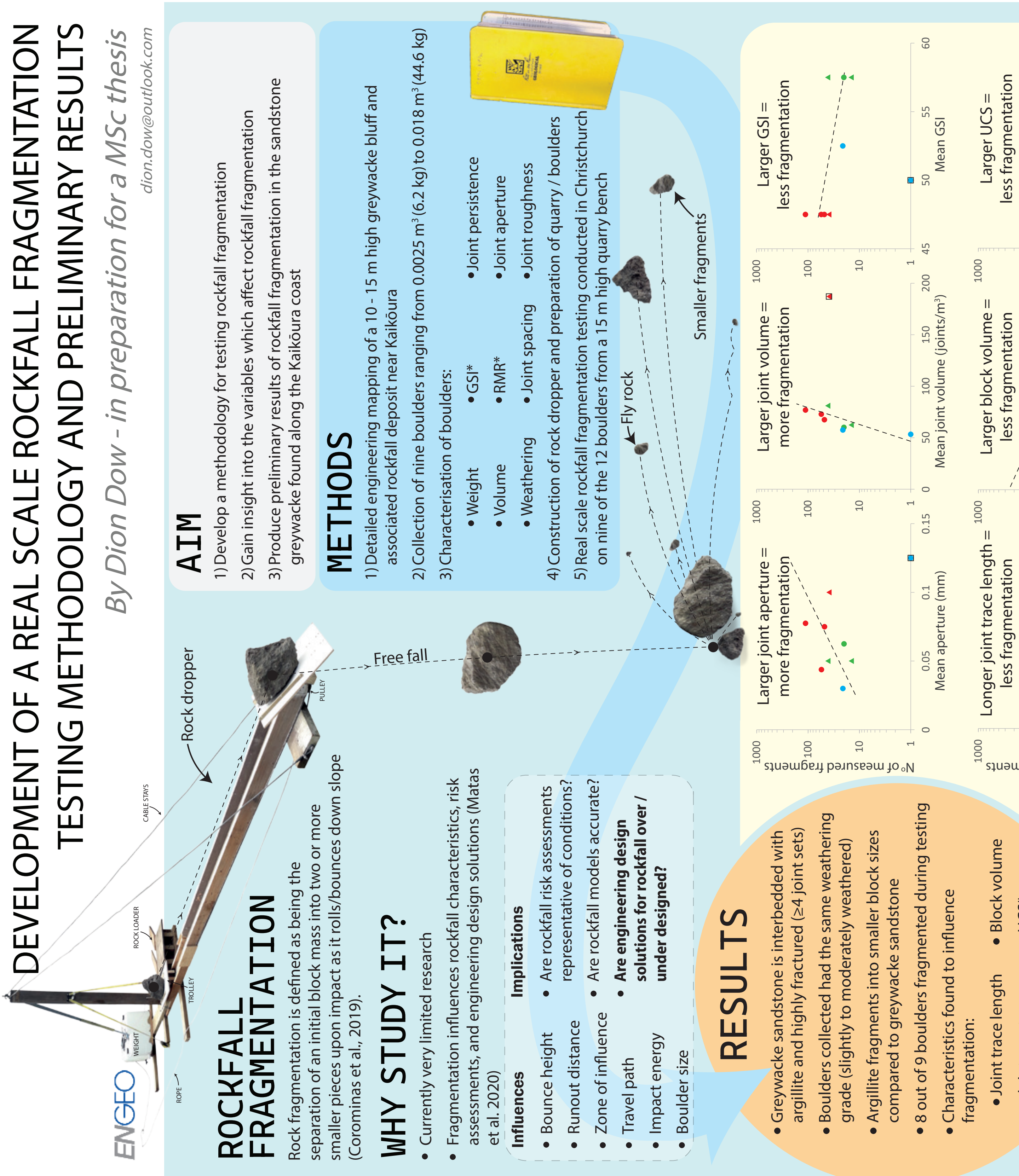
## SPONSORED TO HELP YOU MEET THE BUILDING CODE

NZS 4431 forms part of the package of more than 120 standards sponsored by Building System Performance, to help those in the construction sector to do the right thing. By bringing together New Zealand's various practical and theory-based experts from across the industry, all in consensus on good practice, NZS 4431 should help keep people safe and buildings standing for generations to come.

NZS 4431:2022 – *Engineered fill construction for lightweight structures is available to download from Standards New Zealand from the end of May.*

https://www.standards.govt.nz/get-standards/sponsored-standards/building-related-standards/

# STUDENT POSTER COMPETITION WINNERS




**1ST PLACE**

**AS USUAL WE** had many entries to this year's poster competition. The standard was high and we would like to thank everyone who entered.

We are pleased to announce the winning posters for 2022 are

### 1ST PLACE

**Dion Dow**

**Development of a Real Scale Rockfall Fragmentation Testing Methodology and Preliminary Results**

### 2ND PLACE

**Winnie Pan**

**Cobb Dam Dams Resilience Research Program (DRRP)**

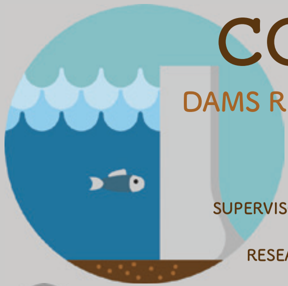
### 3RD PLACE

**Jack Mackay-Neal**

**Toe Buckling and Geomorphology of the Mt Crichton DSGSD**

**Congratulations to you all.**

2ND PLACE



# COBB DAM

## DAMS RESILIENCE RESEARCH PROGRAMME (DRRP)

WINNIE PAN
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### Background

Cobb Dam is an earth embankment dam situated in Kahurangi National Park on the Cobb River; construction for it finished in 1956. The dam was the first hydro-electricity dam built using modern soil control methods. It is also the first to include internal water pressure, seismic, magnetic survey measurement devices and radiographic testing of penstock welds. Water is stored at 794m in altitude, and it has the highest hydraulic head of almost 600m. The power station has a 32MW capacity, and its average annual output is 192GWh, is enough to meet Nelson's full electricity requirements. The power station was commissioned in 1944; Trustpower took over the ownership/operation of Cobb in 2003 and they are responsible for the monitoring and surveillance of the dam.[1]

Like most hydropower dams in New Zealand, Cobb Dam was constructed before modern dam safety practices and guidelines - such as NZSOLD Dam Safety Guidelines (2015) - were established. Therefore, it is crucial to determine if the dam adhered to the standards. Dam safety is crucial so that harmful effects on people, property and the environment can be mitigated through an appropriate Dam Safety Assurance Programme.

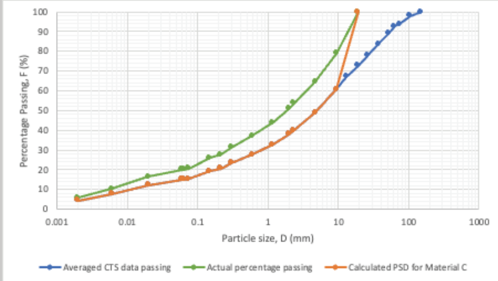
The main experiment undertaken as part of this summer research programme was Sand Castle Tests (SCT) to look at the soil's self-healing capabilities and its ability to withstand a crack. Soils that do not self-heal and withstand a crack can provide pathways for erosion to occur and this can have detrimental consequences to the dam.

### Materials

SCTs can evaluate the collapsibility, cracking and self-healing potential in soils. The test is based on the phenomenon when water hits a sandcastle at the beach, causing collapse. Basic soil properties such as moisture content, relative density, fines content influence soils' ability to hold a crack. If the structure collapses, the soil behaves in a non-cohesive way. If collapse does not occur, the material can sustain an open flooded crack or defect and fail to protect the dam from concentrated seepage and/or erosion.[2] While SCTs were developed to assess the collapsibility of filter materials, we conducted this test on a gravelly core material to assess its likelihood to sustain a crack or defect.

To perform the SCT, we had to produce a grading curve representing the Cobb Core accurately but also be able to fit into the proctor mould. To do this, we averaged the three gradation curves undertaken on core samples taken from the field to produce the entire particle size distribution (PSD). Using core sample fractions in the lab (re-used following other testing), we calculated the particle size distribution of the re-used sample. Given the size of the proctor mould, we limited the maximum particle size to 19 mm and calculated the Material C grading using mass replacement of coarser gravels and cobbles. We then used our gradation curve to calculate required masses of different particle sizes, mixed everything, and the "Material C" soil was ready to test.

### Particle Size Distribution Curve (PSD)




The graph shows the following:

- PSD for the average of Material A, B and C from CTS data
- PSD of the Cobb Core from CTS data
- Mass-replaced PSD of Cobb Core Material C

The mass-replaced PSD was used to perform the SCTs.

### Methodology

There is no definitive method for SCTs; however, we established a suitable procedure from literature reviews. The maximum dry density (MDD) of the Cobb Core material from commercial laboratory data of 2.13 t/m³ was used to determine the mass of material required for the proctor mould for 100% relative compaction. The material was cured overnight in containers with distilled water to a moisture content of 8%. After curing, a tamping rod was used to compact the material into 4 layers to fill up the proctor mould and cured over a second night. The test was performed in a fish tank with a stopwatch because the key measurement was the time to collapse, a ruler for sizing references, a camera for record collapse patterns and the specimen taken out of the mould sitting on a sieve with water just below to allow seepage of water from the bottom. Once the specimen was on the sieve, the timing began, and the tank was filled to 30mm above the initial water level to allow seepage into the sides. After the test was done, the material was taken out and oven dried to determine the mass and density of the material and hence the relative compaction of that test.

Figure 1: SCT Set Up

Figure 2: No Collapse

Figure 3: Collapse

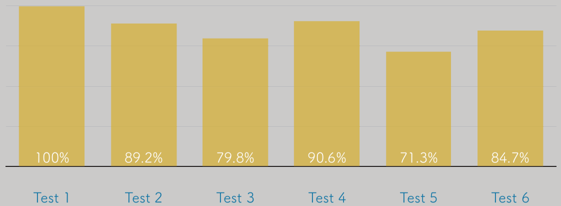
Figure 4: Partial Collapse

### Results

#### Relative Compaction

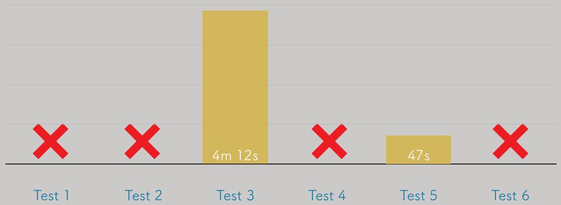
Relative compaction is the specimen mass to the MDD mass. Different relative compactions resulted in either collapse, partial or no collapse of the specimen. The target moisture content (MC) for each test was 8%, as this was the optimum moisture content (OMC) from commercial laboratory data.

Comparison between Relative Compaction of each Test




#### Collapse Time

Comparison between Collapse Time of each Test




Test 3 (relative compaction 80%) and Test 5 (relative compaction 70%) collapsed. Indicating the soil is self-healing at these levels of compaction. Above 80%, it is not self-healing.

### Discussion

The collapse time with respect to relative compaction shows that above 80% relative compaction, the Cobb Core material - like most fine-grained dam core materials - is not self-healing. The soil is no longer self-healing between 80-85% as the 85% relative compaction did not collapse. We conclude that compaction is significant in regards to the self-healing properties of the dam core. Field sampling in 2017 showed that the core was well-compacted, with relative compaction likely to be in the range of 90-100% MDD. If a crack, void or defect were to form and the soil was unable to collapse to infill the crack, the core soil would likely be reliant on a downstream filter zone to arrest possible concentrated seepage/and or erosion. Further research is required to conclude the extent to which Cobb Dam follows modern geotechnical guidelines.

References

1. Trustpower. (2020). Cobb Power Station. Retrieved from https://www.trustpower.co.nz/our-assets-and-capability/power-generation/cobb
2. FEMA. (2011). Filters for Embankment Dams: Best Practices for Design and Construction. Federal Emergency Management Agency. United States

# Toe Buckling and Geomorphology of the Mt Crichton DSGSD

Jack Mackay-Neal—jlm184@uclive.ac.nz

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

3RD PLACE

## Objective

Test if the criteria for Toe Buckling in the Cromwell Gorge (Ridl, 2021) is applicable elsewhere within Otago, and map the geomorphology of the Mt Crichton Deep Seated Gravitational Slope Deformation (DSGSD).

## Toe Buckling Criteria

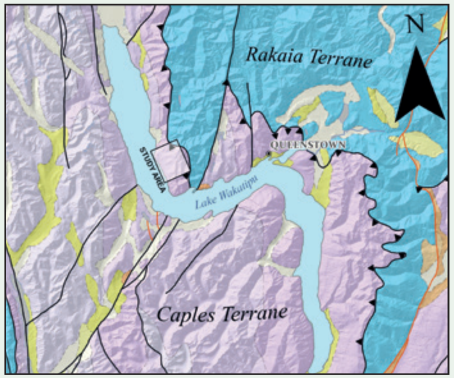
· Slopes of at least between 500 and 800m in height

· Affected by the stresses of the Kaikoura Orogeny

· Foliation following or steeper than the hillslope angle

· A sequence of psammitic and pelitic schist, allowing for flexural slip within the schist

## Site Overview

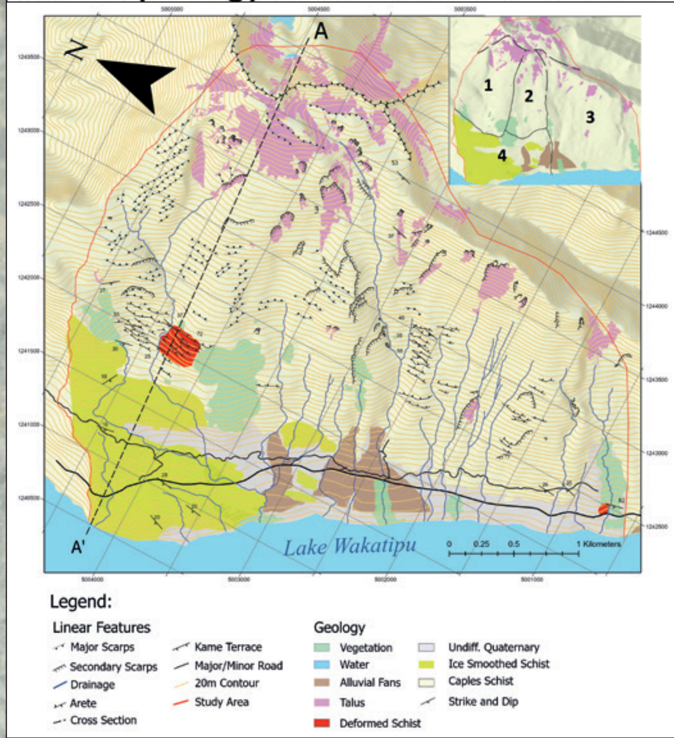



Mt Crichton sits 1500m above Lake Wakatipu with a DSGSD on its Western slopes. It lies proximal to the Queenstown -Tairei Synform, tracing the northern limb of the lake. The lake is a product of extensive Pleistocene glacial episodes.

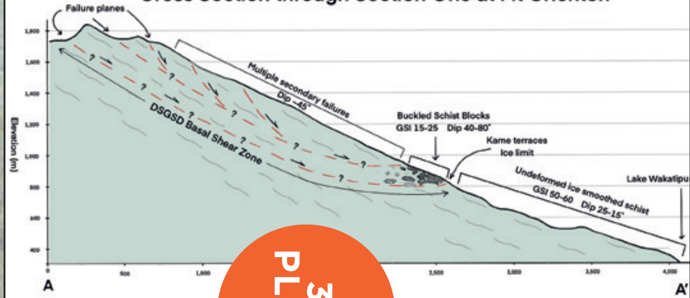
## Methodology

Mapping was performed in ArcGIS, by drawing features on satellite imagery. A site visit focussed on verifying the GIS mapping, characterising, and measuring the schist.

## Geomorphology of the Mt Crichton DSGSD




## Cross Section through Section One at Mt Crichton




## Findings

- The slide can be dissected into three discrete slide portions, based on their toe location
- Buckled schist was observed in Zones 1 and 3
- Buckling in Zone 1 occurs 500m above the lake level, and 100m above lake level in Zone 3
- Kame terraces demarcate the toe of Zone 1, and the limit of ice smoothed schist
- No movement was observed within or below the ice-smoothed schist
- Foliation follows the hillslope, flattening to 15° towards the Lake and Synform

## Discussion

Ice buttressing appears to have limited the extent of Zones 1 and 2. Non-deformed ice-smoothed schist and kame terraces at the toe of Zones 1 and 2 indicate prolonged ice levels at this level. Lack of failure of the schist below Zones 1 and 2 after ice retreat is likely a function of flatter foliation and a potential strength increase by lithologic variance. The lower slope prominence of Zone 3 may have contributed to it failing following ice retreat. Ridl's criteria (2021) was accurate in predicting toe buckling outside of the Cromwell Gorge.

Further buckling on the slope cannot be ruled out given limited time for site walkovers and the large site size. LiDAR would aid in future mapping the site and determining the true location of the slope toes. Detailed models of the subsurface cannot be drawn without intrusive data—geophysics or boreholes.

**Acknowledgements:**

Romy Ridl for her invaluable assistance throughout this study, David Bell in aid of elucidating the mechanisms of slope failure, Bruce Douglas for allowing me access to Mt Crichton and Conor Cussins for his field assistance. A thanks to the Mason Trust for their financial assistance for this research.

**References:**

Ridl, R. N. (2021). Evaluation of buckling deformation in the schist of the Cromwell Gorge, New Zealand.

Turnbull, I.M., (2000). Geology of the Wakatipu Area. Institute of Geological and Nuclear Sciences 1:250 000 Geological Map 18.

**Background:** Photo of greenschist from Mt Crichton

# Professor Michael J Pender

## 19 June 1943 – 15 December 2021

**MICHAEL JOHN PENDER,** or Mick to his friends, colleagues, and students, obtained his BE(Hons) and PhD in Civil Engineering from the University of Canterbury in 1966 and 1971, respectively. In 1972, he received a post-doctoral fellowship at the Engineering Department of the University of Cambridge. Upon his return to NZ in 1974, he worked at the Ministry of Works and Development Central laboratories in Lower Hutt where he was in charge of the Geotechnical Laboratory. In 1977, he joined the University of Auckland as a Senior Lecturer and was named Professor of Geotechnical Engineering in 1985 – a position he held until his retirement in March 2021.

At the University, he served as the Head of Department, and was on the University Research Committee, Scholarships Committee, Discipline Committee, Honours Committee, and various appointment committees. Moreover, he also served as expert consultant in many engineering projects through UniServices. He was a Visiting Professor to the European School for Advanced Studies in the Reduction of Seismic Risk (ROSE School), University of Pavia.

He was a passionate teacher and excellent mentor. He received the 2005 Faculty of Engineering Distinguished Teacher Award and the 2005 University of Auckland Sustained Excellence in Teaching Award. From 2002 – 2007, he was on the “Top Teachers List” in the School of Engineering. He had supervised numerous PhD, ME, and Part 4 Project students, many of whom are now leaders in the New Zealand’s geotechnical industry.

He was an outstanding researcher and problem solver. For nearly a half-century, Mick dedicated his research to understanding the properties of NZ geomaterials, most of which are unique to NZ. Consequently, much of his effort was directed toward explaining to the local geotechnical profession how these properties were different and that they could not simply import understanding from overseas. Highlights include the soils in Wellington derived from the in-situ weathering of greywacke, the elucidation of the properties of residual soils in Auckland, the stability of slopes in closely jointed Wellington greywacke, the properties of coal and the measurement of in-situ stresses in the Huntly West mine, and measurement of the compressibility of the materials in the Wairakei-Tauhara geothermal field. Prior to his retirement, he also worked with colleagues on the properties of volcanic soils from the Central North Island. The NZ Geotechnical Society bestowed him the 1996 Geomechanics Lecture Award (NZGS' premier award), and he delivered a lecture on the topic "Aspects of geotechnical behaviour of some New Zealand materials".

A parallel workstream that occupied his time was the design of foundations for buildings, bridges, and other infrastructure. For many years, he promoted the integrated design of structure-foundation systems and recognised the need for better collaboration between the structural and geotechnical communities. He led the introduction of LRFD (Load and Resistance Factored Design) into geotechnical practice in NZ. Some of his recent work had involved field testing of shallow and deep foundations subject to cyclic loading to better understand the nonlinear stiffness and damping of these foundations under earthquake actions. Before his death, he was in the final stages of preparing a monograph on the *Design of Earthquake-resistant Foundations* (which we, his colleagues, hope to complete before the end of 2022).

His service to technical societies and the engineering profession was unparalleled. He was a Distinguished Fellow of Engineering NZ; a Life Member of the NZ Geotechnical Society; and a Fellow, Life Member, and former President of the NZ Society for Earthquake Engineering. From 1991 – 1995, he was the Australasian Vice-President of the International Society for Rock Mechanics. In 2012, he was elected as an International Honorary Member of the Japanese Geotechnical Society, a testament to his well-established international collaborations.

He was adept in both numerical and experimental aspects of geotechnical engineering. The constitutive model he proposed to explain the deformation mechanisms of overconsolidated clays appeared in Géotechnique in 1978; reputable researchers in the field are still citing the paper! He loved to tinker with various experimental equipment. For example, in early 2000s, he developed a free-standing and portable laboratory pressure system (pressure regulator) that is still being used in our Geomechanics Laboratory.

Colleagues and students benefited significantly from his wisdom and mentorship. He had an open-door policy, and everybody was welcome in his office (often with classical music playing in the background). Discussions with him were not only confined to geotechnical problems and concepts but also to arts, religion (he had a firm Catholic belief), and philosophical issues. He loved woodwork and furniture-making; he had an impressive set of tools for this purpose in his basement workshop. Following the Christchurch earthquake, he made his own "tiltmeter" to measure how much the downtown buildings had leaned as a result of the shaking.

His office library had books that covered all these wide-ranging topics – even giving away some of his early-edition books when he found the office library of his younger colleagues a bit wanting. Even in the digital age, he was a frequent visitor to the university's Central Library, always curious about newly-arrived books on topics of his interest.

To honour his retirement, Mick presented an "Exaugural Lecture" at the University of Auckland in May 2021, wherein he presented his thoughts on research, teaching, and professional work. The lecture, held in front of an overflowing audience, was well-received and showed his endless enthusiasm for engineering, great intellect, and wonderful memories. His lecture is available on YouTube: https://www.youtube.com/watch?v=uh8YzhAyGy8.

Following his death, the messages we received from many people (both from NZ and around the world) who had been influenced by Mick's passion for teaching and research had a common theme – Mick was a great mentor, a true scholar and a gentleman, and a great role model and inspiration. He will be missed by all who knew him.

*The Geomechanics Group, University of Auckland*

# Prof. David Hiley Stapledon

## 20 November 1930 to 15 December 2021

**DAVID WAS ONE** of Australia's foremost engineering geologists with a career spanning more than 50 years. An extraordinary human being as well as an extraordinary geologist, David was a legend in both Australia and New Zealand.

His career started with the Snowy Mountains scheme in 1951 (his MSc thesis in 1961 was 'Geological studies for the planning and development of Tumut 2 underground power station') and went on to span consulting, working and living in the Mekong Delta (1960-1963), teaching at the University of South Australia (1977-1993) and more consulting (until 2005).

David was well known to (and hugely respected by) many of NZ's engineering geologists and geotechnical and dams engineers as a user-friendly, approachable reviewer (along with Laurie Richards) on the Clyde landslides project in the late 1980's. In addition to his contributions to engineering geology in Australia, including as a mentor to some of their current leaders in the field (Alan Moon and Fred Baynes for example), David was an invited speaker at several conferences here in NZ including the Dams and Canal Symposium in Alexandra in 1983 and the 1992 International Symposium on Landslides in Christchurch. In 1996 he was awarded the John Jaeger Memorial Medal for "contributions of the highest order over a lifetime commitment to the geotechnical profession in Australia".

Among his many contributions, David was a co-author of 'Geotechnical Engineering for Dams', a comprehensive (and much thumbed) textbook on the geotechnical and geological aspects of the investigations for, and the design and construction of, new dams and the review and assessment of existing dams.

For a number of NZ's current and past senior engineering geologists, the retention of David Stapledon and Laurie Richards in 1988 to review work done on the landslides along the (future) Clyde dam reservoir was their first exposure to independent external review. Who needed that??? But it turned out to be one of the best things that happened to us – it influenced our whole careers in a hugely positive way.

I knew of David from his report on Kangaroo Creek dam. We (NZ Geological Survey at the time) had used it as a guide to our own Engineering Geological Construction Reports for Moawhango Dam and the Upper Waitaki scheme components, including Pukaki Dam and the Ohau canals.

To me, David was a breath of fresh air. He thought like I did, but better – in 1988 he had the advantage of nearly 40 years' experience to my 10 - and he had a huge desire to share his learnings! David's experience and supportive approach meant that he quickly earned our respect. From him, the engineering geologists on the Clyde landslides stabilisation team learned, and retained forever, what seem like the obvious but for many were truly beneficial lessons. These included:

- Clearly separate fact from interpretation in all of your work
- Record all factual information on maps, plans and logs at a useful (understandable) scale
- Present the interpretation (ground model) clearly on cross sections
- Support your interpretation with detailed information and references – it must be verifiable by others
- Know the limitations of your model and keep working to improve it

His key message was: In doing all of this, talk to the engineers using your information so that they understand the significance of the geology to their project.

I have spent the last 30 years trying to pass these (and many other) lessons on to the engineering geologists and engineers with whom I have since worked. In doing this I have tried to be like David – supportive, not critical, kind and friendly. If I end my career with half the *respect that David had, I will be more than happy.*

*Don Macfarlane*

# International Society for Rock Mechanics and Rock Engineering (ISRM)

## Report for New Zealand – June 2022

**Paul Horrey**

*Paul Horrey is a principal and engineering geology specialist with Beca and manages the company's Southern Geotechnical Team based in Christchurch. He has worked extensively in New Zealand and overseas in infrastructure, mining and hydropower and has a particular interest in natural hazard mitigation and risk management.*

**Romy Ridl**

*Romy is an engineering geologist with KiwiRail covering the geotechnical aspects of the South Island's rail network. She is a former Engineering Geology and Rock Mechanics lecturer at the University of Canterbury and has geotechnical experience from Australasia and Africa.*

### ISRM CONGRESS 2027 HOSTING BID UPDATE:

You will recall from our last update that NZGS is bidding to host the 16th ISRM International Congress in 2027. This is ISRM's flagship event and occurs every four years. A little like the Olympic Games, potential host countries must bid many years in advance to be able to host the Congress. An organising committee has been formed which comprises Paul Horrey, Stuart Read, Eleni Gkeli, Romy Ridl and Christoph Krauss. The Committee, with the valued support of Tourism New Zealand has selected Christchurch as our host city and developed a promotional video and supporting information pack which were presented to the ISRM Council on 17 November 2021. Several of our bid organising committee will attend the next Council meeting at the LARMS 2022 ISRM Symposium in Asuncion, Paraguay to make a final presentation of our bid. The Council will then vote on which country will host and the outcome should be known during the conference. Other bidders are South Korea and China.

### BOARD AND COUNCIL MEETINGS:

The 2022 ISRM Board, Commissions and Council meetings will be held in Asuncion, Paraguay during LARMS 2022 on 15-16 October.

### ISRM AWARDS:

The Rocha Awards for 2022 are.

WINNER:

- Radhika De Silva, Sri Lanka, for the thesis "Rock fracture stimulation using a slow energy releasing fracturing compound for permeability enhanced in situ leaching", presented at Monash University, Australia.

RUNNERS-UP:

- Masoud Rahjoo, Iran, for the thesis "Directional and 3 D confinement dependent fracturing, strength and dilation mobilization in brittle rocks", presented at University of British Columbia, Canada;
- Li Xiaofeng, China, for the thesis "Research on Rock Fracturing and Fragmentation Subject to Intensive Impact", presented at University of Chinese Academy of Sciences, China.

### KEY INTERNATIONAL SYMPOSIA 2022-3

Eurock 2022 - Rock and Fracture Mechanics in Rock Engineering and Mining in Espoo, Helsinki-region, Finland, 12 to 15 September 2022. The full programme has been published and early bird registrations are open until June 15.

The 2022 International Symposium will be held in conjunction with the IX Latin American Congress on Rock Mechanics, Rock Testing and Site Characterization in Asuncion, Paraguay on October 12-19, 2022.

2023 ISRM 15th International Congress, Salzburg, Austria 9-14 October 2023. Abstracts close 30 June 2022.

Other regional events and specialised events are listed on the ISRM website.

### NEWSLETTER

The latest ISRM newsletter (No 56 dated December 2021) is available on the ISRM website

### NEW ISRM SHORT COURSE

A new ISRM short course "Ground control consideration in high-production longwall mining in Australia" was recorded and made available in 2021 by Prof. Ismet Canbulat, from the University of New South Wales, Sydney, and has now been added to the ISRM website.

## TRAINING OPPORTUNITIES

Your attention is drawn to the excellent technical resources available on the ISRM website. These include Suggested Methods, Reports, Glossary (multi-lingual), digital library, Slide Collection, Online Lectures, Videos, Online Courses and Book Series. Some items are available for purchase, but many are free of cost.

## YOUNG PROFESSIONALS' WEBINARS

The ISRM Young Rock Engineers group have established the ISRM Young Members Monthly Seminars.

The most recent presentations in the series have been:

### 5TH SEMINAR - 27 MAY 2022

- Directional and 3D-Confinement-Dependent Fracturing, Strength and Dilation Mobilization in Brittle Rocks - Masoud Rahjoo (AECOM - Canada)
- Stochastic Discrete Element Modelling for Pillar Strength Determination: a First Step in a Risk-Based Pillar Design Approach - Juan José Monsalve (Virginia Tech University - USA)

### 4TH SEMINAR - 28 APRIL 2022

- Experimental analysis of burst type extreme rock failures and rock fracture under high-stress conditions by Selahattin Akdag (Australia).
- Coseismic rock slope failure mechanisms - insights from landslides triggered by the 2016 Mw 7.8 Kaikōura earthquake by Corine Singsein (New Zealand).

## ISRM ON-LINE LECTURES

A NEW on-line lecture has been given since the last report:

- December 2021 *The 36th Online Lecture "Methodolo gies of underground rock support and applications"* Prof. Charlie Li
  This has been recorded and is available on the ISRM website.

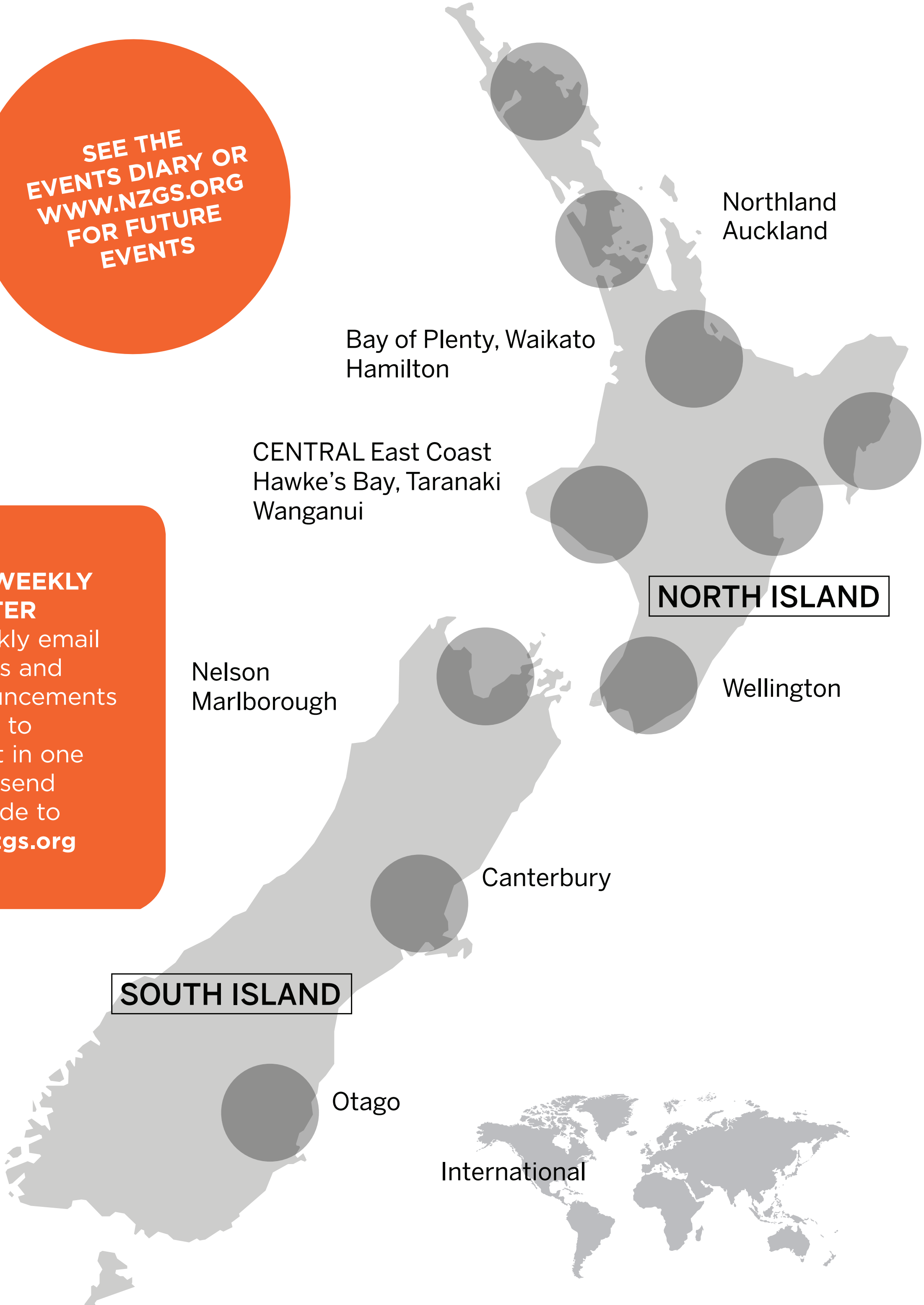
**Paul Horrey**
*NZGS ISRM Liaison*

**Romy Ridl**
*NZGS ISRM YGP Coordinator*
*June 2022*

# Society Branch Reports

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

**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**



## Auckland

The Auckland Branch have been steady over the past 6 months, playing it conservative and continuing with online talks through the recent Covid restrictions, however, with things seemingly easing there is a view to get in-person events back up and running.

In a first, the team took the annual Auckland Branch Christmas Mini Symposium online in late December, which provided a great finish to the year for events. Although the ability for participants to network was hindered to some degree by the online nature of the event, the silver lining was that there was no registration fee, and the event was open for all to tune into. A range of great speakers put their hands up and presented interesting and thought-provoking presentations.

In recent months, the Auckland team have been supporting a number of international speaker talks with Rolly Orense; talks have included: *'Mine Subsidence: Cause, Effect, Mitigation', 'Design analysis of single and multiple debris barriers'*, and *'Experimental Investigation into Multiple Occurrence of Liquefaction'.* Turn out to these online events has been great and judging by the questions posed, there always seems to be an engaged audience.

Looking forward, the Auckland Branch have re-organised the event *'Panel discussion - The role of a peer reviewer in geotechnical design'.* Some may recall that this event was cancelled only 2 hours before it was due to start last year, and in a complete coincidence, it will be run on the same day exactly 12 months later! We also have an upcoming talk from the members of the CRL design and constructions teams, plus many other upcoming evening talks.

In the coordinator space, Sirini De Silva has recently stepped away from the Auckland Branch after a relatively short stint due to a move overseas – we wish her all the best and give thanks for her input. In exciting news, a call for expressions of interest was put out to the NZGS members and we had several great candidates come forward. Ben and Jay were impressed with the quality and keenness of members to contribute and give back to NZGS. They welcome Lynette Sla and Vick Kumaran to the coordinator team and look forward to seeing what they bring to the role.

## Tauranga

While the Tauranga branch has experienced a COVID-19 related hiatus of in person events, we hope our members have been able to attend some of the online offerings that continue to be advertised through the NZGS calendar. Over the coming months we are positive about restarting in person gatherings. We would love to have local presenters speaking about the projects they have been working on in recent times, to bring a local flavour to the usual range of travelling presenters and quality recorded material used at NZGS events. If you have something you would like to share with the local Geotechnical community, please reach out to Kim or James. It doesn't have to be a 'feature length' event, as combining several shorter presentations into one evening has been very popular in the past.

*James Griffiths*

## Gisborne

The Gisborne Branch held its first presentation in December 2021, where Merrick Taylor, Senior Associate at Beca presented a webinar on the geotechnical aspects of the design of upgrading the Gisborne Wastewater Treatment Plant, including ground improvement to achieve Gisborne District Council's resilience objectives. Merrick's presentation was extremely well attended, not just by practitioners based in Gisborne, but by the wider NZGS community. While covid has been disruptive to all of us, it has really helped to ensure that NZGS can reach every geoprofessional across the country, and small branches like Gisborne have really benefitted from joining other branch presentations remotely. The Gisborne branch is planning to highlight other local projects through the year and will aim for an in-person event too.

*Frances Neeson*

## Wellington

Since the December Geomechanics issue, the Wellington Branch has hosted two presentations for NZGS members. While both presentations had to be held online due to Covid-19 restrictions, this allowed people from across New Zealand to dial in and listen to the talks. Both talks were very well attended, with about 230-240 participants in each talk. In case you missed either of the presentations, they were recorded and are available to view on-demand on the NZGS website.

In December Chris Massey from GNS Science presented on the *geomechanical characterisation of greywacke for dynamic slope stability analyses*. In his talk, Chris talk presented some of the findings from his team's SLIDE research project, including regional-scale landslide trends, landslide susceptibility factors, and a discussion of Wellington's greywacke rock slopes. He concluded his talk by discussing what the findings from their research imply for the next large local earthquake in the Wellington Region.

In March this year we had Matt Hill, also from GNS Science, present on the *application of 3D geological modelling in Wellington to assist with engineering geology and understanding natural hazards*. In his talk, Matt discussed various geological models of the Wellington region which have been created at different scales. He also covered some of the challenges of urban geological modelling, the data that underlies the models, and how these models can be used for practical applications and understanding natural hazards.

We want to thank Chris and Matt for their great presentations, and for being able to present online. A big thanks also goes out to Holly and the Engineering NZ team for hosting the online sessions and making everything run smoothly.

We've got some exciting events planned for the remainder of 2022, including a possible fieldtrip and some further (in person) presentations, so keep an eye out for upcoming events in the NZGS newsletter! As always, if you have any ideas for presentations or events, please get in touch.

*Christoph Kraus*

## Otago

The Otago branch has had a quiet few months but we have a couple of events to set up later in the year, including the postponed South Dunedin Groundwater presentation. We sent a survey out to our members to understand the type and style of events they would like to see more of, and we will shortly arrange an event to network with our members and go through the results. Matt has stepped down as a Branch Coordinator and has been replaced by Tim Plunket from GeoSolve, so please get in touch with Tim or Hannah if you have any interesting sites or projects locally you would like to talk about to the branch.

**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.**

# Branch Coordinators

## 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. I look forward to improving the geotechnical features of soils in Northland. I enjoy the coastal lifestyle of Northland

**phil@coco.co.nz**

## AUCKLAND

### JAY DODDABALLAPUR

Jay is a UK and NZ registered Chartered Geotechnical Engineer and is a Senior Associate at Jacobs. He has worked in the UK, Middle East and New Zealand on buildings, infrastructure and marine projects. He has experience in design and management of temporary and permanent works with a particular focus on providing value engineered, sustainable and buildable solutions.

**Jay.Doddaballapur@jacobs.com**

### BEN FRANCIS

Ben is a geotechnical engineer with Tonkin & Taylor in Auckland and has a BE(Hons) and MEngSt(Geotech) from the University of Auckland. He has a broad interest in geotechnical engineering design, with a focus in liquefaction and geotechnical earthquake engineering. He works on technically challenging projects across NZ and internationally.

**BFrancis@tonkintaylor.co.nz**

### LYNETTE SLA

Lynette is a geotechnical engineer with McMillen Jacobs Associates. She graduated in the US with a BSc (Civil) and MSc (Geotechnical) from the University of Washington before joining the company in 2011. Lynette moved to NZ in 2017 and is now based in their Auckland office providing design support and coordination for a variety of underground projects in the US, NZ, and Australia, with a particular focus on tunnel and shaft excavation support.

**sla@mcmjac.com**

## AUCKLAND

### VICK KUMARAN

Vick is a Chartered Geotechnical Engineer and is a Technical Director at Mott MacDonald. He has extensive experience working with geotechnical consultancy firms and specialist contractors, accumulating a variety of exposure to complex ground conditions and engineering risks. He has worked on various infrastructure projects in New Zealand, Australia and Malaysia.

**Vick.Kumaran@mottmac.com**

## WAIKATO

### LUKE STANLEY

Luke is an engineering geologist with CMW Geosciences in Hamilton. He graduated from the University of Plymouth, UK with BSc (Hons), 2016 and MGeol Geology, 2017 before moving into the geotech sector. After working in the south-west of the UK, he decided to chase warmer climates and gain experience working with the very variable soils in the Waikato. He has been working with CMW since 2019 on projects throughout the central north island. **lukes@cmwgeo.com**

### BEN SMITH

Ben is an engineering geologist with HD Geo in Hamilton. He graduated from the University of Waikato with a BSc in GeoSciences in 2013, and has since gained knowledge and experience working on projects through the central north island. Outside of work he enjoys the outdoors, surfing, music and the odd DIY renovation project.

**ben.smith@hdgeo.co.nz**

### SHIMA SHEYBANI AGHDAM

Shima is a geotechnical engineer currently working at HD Geo in Hamilton. She studied for a bachelor's in structural engineering in Azarbaijan before graduating with a Masters in geotechnical engineering in 2015 from Shahid Rajaee University in southern Iran. She moved to New Zealand in 2016 and joined HFC in Canterbury before moving to join HDGeo in April 2019.

**shima@hdgeo.co.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 Senior Lecturer at the University of Waikato and a Senior Geotechnical Engineer with ENGEO and is based in Tauranga. Kim's experience includes earthworks, detailed seismic assessments, building foundation design, 3Waters projects and resilience. Kim's research interests cross a broad range of geotechnical areas including the behaviour of pumiceous soils, ground improvement and soil-foundation-structure-interaction.

**KDeGraaf@engeo.co.nz**

## GISBORNE

### FRANCIS NEESON

Frances is an Engineering Geologist and Eastern region Geotechnical Manager with LDE Ltd in Gisborne. She has over 12 years of experience having previously worked in Auckland, Christchurch, and Kaikoura. She has previously held the role YGP Rep on the NZGS Management Committee from 2014-2017. Frances is looking forward to building momentum with the new Gisborne Branch and actively contributing to the NZGS community again.

**f.neeson@lde.co.nz**

## 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**

## WELLINGTON

### CHRISTOPH KRAUS

Christoph is a chartered Engineering Geologist at Beca in Wellington. His key interests include analysing complex geology, as well as the assessment and mitigation of natural hazards. He's conducted fieldwork in a range of different geological settings in New Zealand, Patagonia, Samoa, and Antarctica. Outside of work, Christoph's interests include travel, exploring the outdoors, football, and photography.

**Christoph.Kraus@beca.com**

### ADAM SMITH

Adam is a professional Engineering Geologist (CMEngNZ) with 12 years' consultancy experience throughout Australia and New Zealand. Adam's key technical experience encompasses slope stability assessment and remediation, geotechnical design of retaining structures, ground improvement, design and construction monitoring of large MSE walls, and management of large scale residential and commercial earthworks operations.

**ASmith@engeo.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**

### 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**

## NELSON

### KYLIE JOHNSON

I'm an Engineering Geologist working for Pattle Delamore Partners based in Nelson. I have been a NZ geologist for the past 9 years and a keen member of NZGS. I work closely with our Engineering NZ branch to bring events and functions to the Nelson region. I have a strong interest in site investigations and mapping around the top of the South Region.

**Kylie.Johnson@pdp.co.nz**

## CANTERBURY

### SARAH BARETT

Sarah is an Engineering Geologist at Beca Ltd in Christchurch. She has experience in natural hazard assessments and completed a PhD and post-doctoral role researching geomorphic influences on observed liquefaction following the 2010-2011 Canterbury earthquakes and 2016 Kaikoura earthquake, and evidence for paleo-liquefaction. In her spare time Sarah enjoys riding her horse and working on her lifestyle property.

**sarah.barrett@beca.com**

### SAM BURGESS

Sam is a geotechnical engineer at Tonkin and Taylor in Christchurch. She has over 4 years' experience in geotechnical engineering, predominantly based in tunnelling projects. Outside of work she enjoys rock climbing, mountain biking and skiing.

**SBurgess@tonkintaylor.co.nz**

### DUNCAN HENDERSON

Duncan is a Geotechnical Engineer at Tonkin & Taylor in Christchurch. He completed his Masters of Engineering at the University of Canterbury in 2013, studying the performance of residential house foundations in the Canterbury earthquakes. Since then, he has worked as a Civil and Geotechnical Engineer, on a range of construction and design projects, including large civil infrastructure projects such as the earthquake recoveries following both the Christchurch earthquakes for SCIRT and the Kaikoura earthquakes for NCTIR. In his spare time Duncan enjoys a range of outdoor pursuits including mountain biking, tramping and disc golf.

**DHenderson@tonkintaylor.co.nz**

## OTAGO

### HANNAH UDELL

Hannah is an Engineering Geologist at Tonkin & Taylor based in Dunedin, having previously worked out of the Christchurch and Hamilton offices. She has over 7 years' experience predominantly based around geotechnical site investigations and a variety of natural hazard assessments.

**HUdell@tonkintaylor.co.nz**

### TIM PLUNKET

Tim is a chartered geotechnical engineer with GeoSolve Ltd in Dunedin currently working on projects over the Otago and Southland region. Previously Tim was based in Christchurch. Tim is has 10 years of experience with his main work areas are focused building foundations, retaining walls and geotechnical structures.

**tplunket@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**

**AFTER A DISRUPTED** two years with Covid our branches are planning some exciting events . Keep an eye on the newsletters for upcoming announcements and information.

*Teresa Roetman*

**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 2021-2022

| POSITION | NAME | EMAIL |
|---|---|---|
| **Chair** | Eleni Gkeli | chair@nzgs.org |
| **Vice Chair & Treasurer** | Jen Smith | treasurer@nzgs.org |
| **Immediate Past Chair** | Ross Roberts | Ross.c.roberts@gmail.com |
| **Elected Member** | Richard Justice | rjustice@engeo.co.nz |
| **Elected Member** | Rolando Orense | r.orense@auckland.ac.nz |
| **Elected Member** | Ayoub Riman | ariman@engeo.co.nz |
| **Elected Member** | Emilia Stocks | estocks@tonkintaylor.co.nz |
| **Co-opted YGP Representative** | Helen Hendrickson | ygp@nzgs.org |
| **Co-opted *Geomechanics News* Editors** | Camilla Gibbons<br>Robert Kamuhangire | editor@nzgs.org<br>robert@kga.co.nz |
| **Co-opted Website Editor** | Olivia Gill | Oliviag@cmwgeo.com |
| **IAEG Australasian Vice President** | Doug Johnson | djohnson@tonkintaylor.com |
| **ISRM NZ Representative** | Paul Horrey | Paul.horrey@beca.com |
| **ISSMGE NZ Representative** | Rollo Orense | r.orense@auckland.ac.nz |
| **Appointed Secretary** | Teresa Roetman | secretary@nzgs.org |

## 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 $135 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/**

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## 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.

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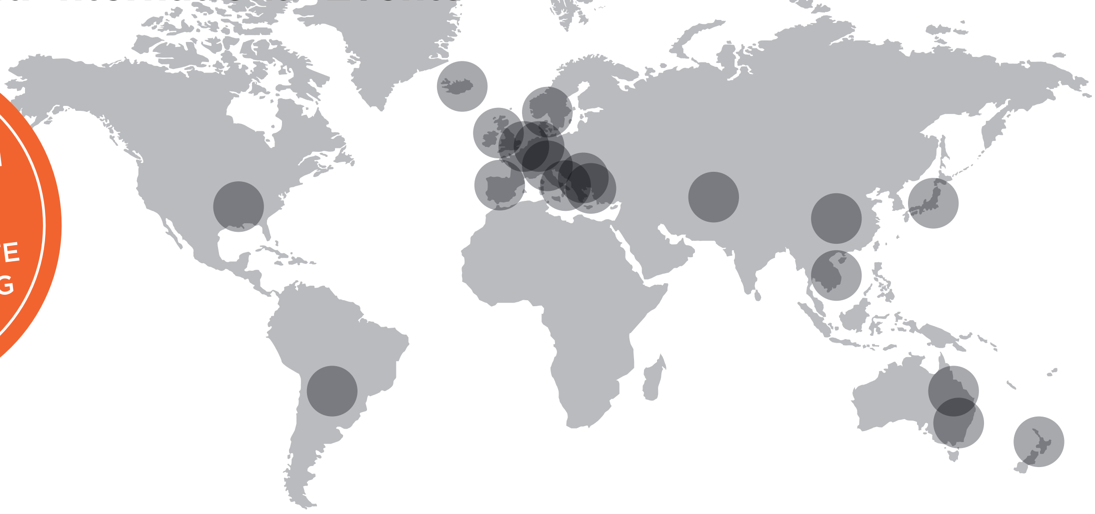

LINKS ARE AVAILABLE FROM THE NZ GEOTECHNICAL SOCIETY WEBSITE WWW.NZGS.ORG

## 2022

**28-31 August**
**Texas, USA**
4th International Symposium on Frontiers in Offshore Geotechnics

**12-15 September**
**Helsinki, Finland**
EUROCK2022

**20-23 September**
**Rotterdam, The Netherlands**
11th International Conference on Stress Wave Theory and Design and Testing Methods for Deep Foundations

**16-19 October**
**Asuncion, Paraquay**
LARMS 2022 – Challenges in Rock Mechanics: Towards A Sustainable Development of Infrastructure

**12-14 October**
**Rome, Italy**
13th Conference on Risk Analysis, Hazard Mitigation and Safety and Security Engineering

**2-5 November**
**Rotorua, New Zealand**
14th NZGS/AGS Young Geotechnical Professionals Conference

**22-26 November**
**Hanoi, Vietnam**
ARMS12 Asian Rock Mechanics Symposium

**29-1 November/December**
**Melbourne, Australia**
AusRock2022 - 6th Australasian Ground Control in Mining Conference

## 2023

**2–5 May**
**Milos Island, Greece**
8th International Conference on Unsaturated Soils

**24-26 May**
**Reykjavik, Iceland**
NROCK2022

**7-10 June**
**Oslo, Norway**
3rd JTC1 Workshop on Impact of Global Changes on Landslide Risk

**26-28 June**
**London, UK**
Numerical Methods In Geotechnical Engineering 2023

**2-5 July**
**Cairns, Australia**
ANZ2023 – 14th Australia and New Zealand Conference on Geomechanics

**14-18 August**
**Kazakhstan**
17th Asian Regional Conference on Soil Mechanics and Geotechnical Engineering (17ARC)

**17-21 September**
**Rome, Italy**
12th International Conference on Geosynthetics

**21-27 September**
**Chengdu, China**
The IAEG XIV Congress

**9-14 October**
**Salzberg, Austria**
15th ISRM International Congress on Rock Mechanics

**20-22 November**
**Fukuoka, Japan**
2nd International Conference on Construction Resources for Environmentally Sustainable Technologies

## 2024

**25-30 August**
**Lisborn, Portugal**
XVIII European Conference on Soil Mechanics and Geotechnical Engineering