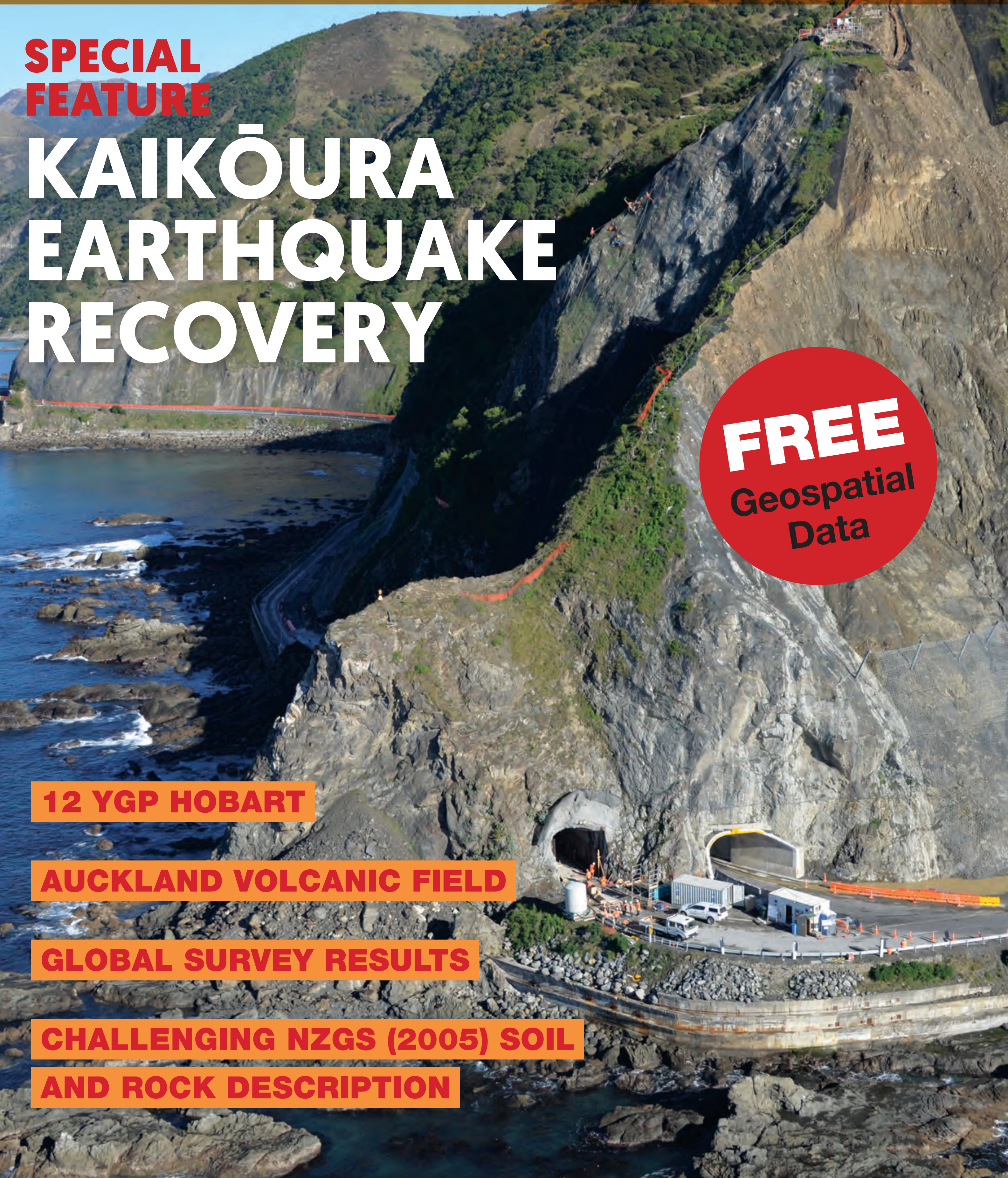
NEW ZEALAND GEOTECHNICAL SOCIETY INC

DECEMBER 2018 issue 96

NZ GEOMECHANICS NEWS

Bulletin of the New Zealand Geotechnical Society Inc.

ISSN 0111-6851



SPECIAL FEATURE

KAIKŌURA EARTHQUAKE RECOVERY

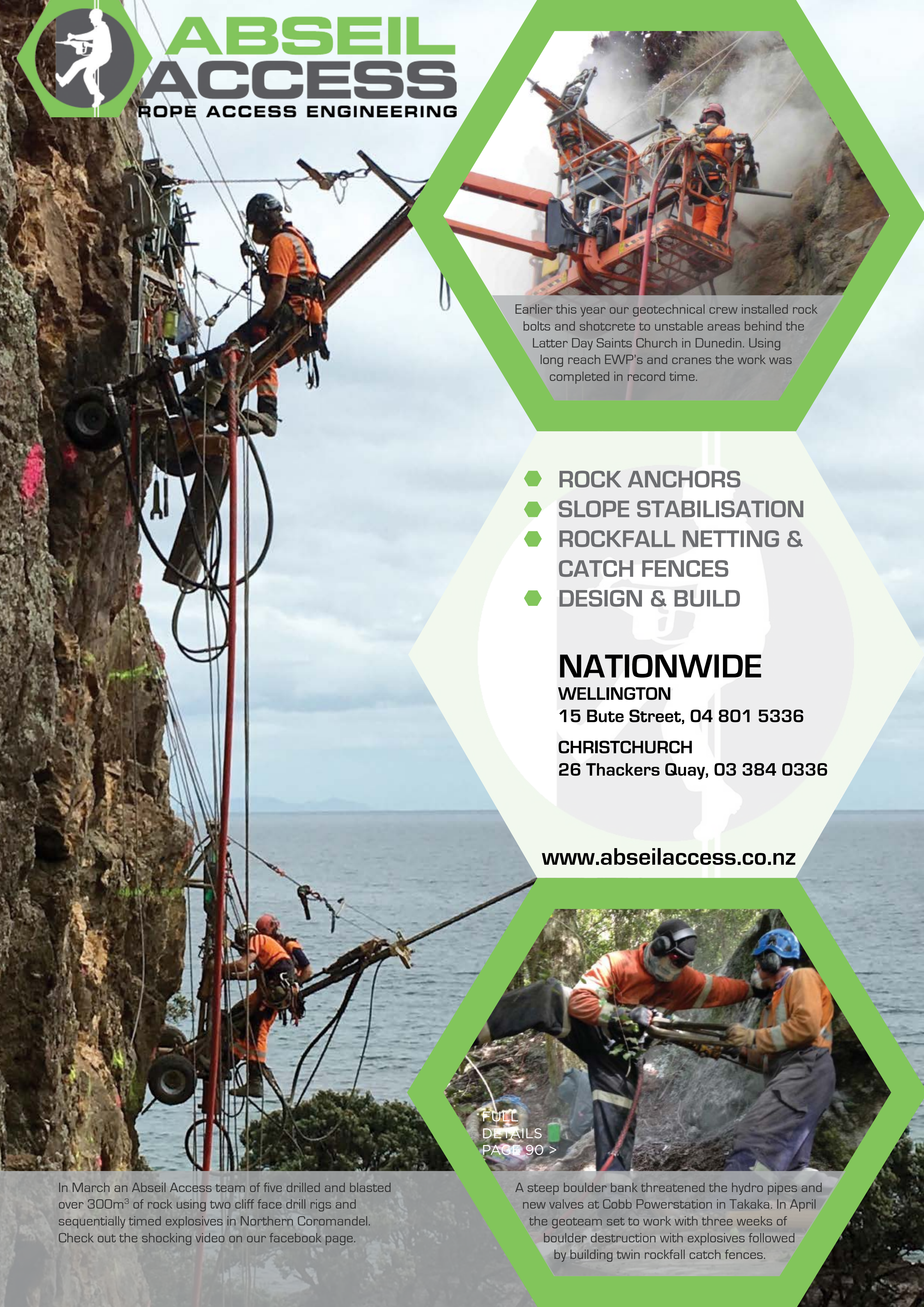
FREE Geospatial Data

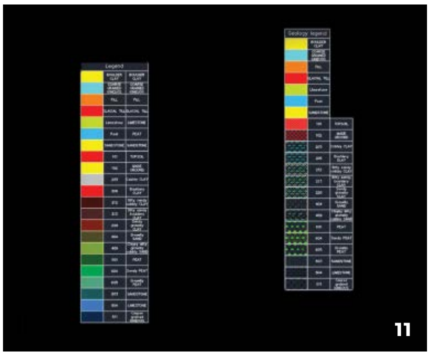
12 YGP HOBART

AUCKLAND VOLCANIC FIELD

GLOBAL SURVEY RESULTS

CHALLENGING NZGS (2005) SOIL AND ROCK DESCRIPTION


ABSEIL ACCESS
ROPE ACCESS ENGINEERING
Earlier this year our geotechnical crew installed rock bolts and shotcrete to unstable areas behind the Latter Day Saints Church in Dunedin. Using long reach EWP's and cranes the work was completed in record time.
ROCK ANCHORS
SLOPE STABILISATION
ROCKFALL NETTING & CATCH FENCES
DESIGN & BUILD
NATIONWIDE
WELLINGTON
15 Bute Street, 04 801 5336
CHRISTCHURCH
26 Thackers Quay, 03 384 0336
www.abseilaccess.co.nz
FULL DETAILS PAGE 90 >
In March an Abseil Access team of five drilled and blasted over 300m³ of rock using two cliff face drill rigs and sequentially timed explosives in Northern Coromandel. Check out the shocking video on our facebook page.
A steep boulder bank threatened the hydro pipes and new valves at Cobb Powerstation in Takaka. In April the geoteam set to work with three weeks of boulder destruction with explosives followed by building twin rockfall catch fences.



COVER IMAGE: Photo Courtesy of North Canterbury Transport Infrastructure Alliance.

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

Tony Fairclough
Chair, Management Committee

THIS SPECIAL EDITION of the New Zealand Geomechanics News has a focus on some of the incredible work which has been completed to date by the North Canterbury Transport Infrastructure Recovery (NCTIR) Alliance.

I hope that all readers will find some time over their Christmas break to read this edition thoroughly as it contains many interesting articles. It is also a worthy record of some of the challenging NCTIR projects to which many of our members have made a significant contribution over the past two years, and which has been recognised on the global stage, winning the Institution of Civil Engineers (ICE) People's Choice Award for 2018.

NZGS WEBSITE

I strongly encourage all members to visit the NZGS website if you have not done so recently. Our Website contains much useful information and reference material, in particular links to videos of select NZGS branch presentations and web-based training that has been developed collaboratively with MBIE and Engineering New Zealand. Many members will find these resources to be an invaluable tool which will help them to enhance their Continuous Professional Development (CPD) programs. The NZGS website also lists all upcoming branch presentations, training courses and symposia which have reached a key milestone with respect to their organisation and certainty, and, noteworthy international conferences and symposia.

Due to the expected demands of his upcoming role as NZGS Chair, Ross has agreed to step down from the NZGS Webmaster role during the coming year. The Management Committee is currently working with Ross to identify and confirm a suitable replacement for this key role. This has proved to be a more difficult than anticipated task. As such I ask any member who feels they may have the appropriate skills and commitment to fulfil this key role contact Ross Roberts (treasurer@nzgs.org) Teresa Roetman (secretary@nzgs.org), or myself at their earliest convenience.

RECENT SOCIETY EVENTS

Between July and December 2018 the NZGS Management Committee, in particular Eleni Gkeli, Teresa Roetman and the branch co-ordinators, have organised numerous highly-successful training courses, society events and branch presentations.

I firmly believe that one of the key activities of the NZGS Management Committee is to co-ordinate and deliver high quality branch presentations, training courses, conferences and symposium. Such events should be of high value to our members, in particular with respect to their CPD. Your management committee has been working hard in recent years to achieve this goal, and, welcomes your continued feedback on past events, and, suggestions for future events. With respect to this issue all members should note, in particular those of you who are located in regional centres, that work is ongoing to video branch presentations and enable you to view these via the NZGS website. Unfortunately, due to various permission, copyright and logistical reasons, it will not be possible to record all presentations, and of those that are, only a few (if any) will be a "live feed". Please be assured that your Management Committee will work to obtain the best-possible recording arrangement for our branch presentations on a case-by-case basis.

UPCOMING SOCIETY EVENTS

Since the inaugural ANZ YGP Conference, which was held in Sydney during 1994, the demand and level of competition to attend this event has grown significantly. In order to maximise the value and quality of experience for the ANZ YGP Conference attendees, the number of delegates is limited to a maximum of 50 (25 from Australia and 25 from New Zealand). This cap on the number of attendees, in combination with significant attendance and travel costs for New Zealand based YGP if the event is held in Australia, has resulted in many worthy YGP delegates being excluded.

In response to this, the NZGS Management Committee has worked with YGP representatives to develop a series of four one-day New Zealand YGP conference events. The aim of these is to complement the ANZ YGP Conference, and, provide an alternative YGP event which affords similar fundamental presentation, networking and professional development experiences.

The inaugural Auckland event was held during October 2018. Planning for events in the Waikato, Central and Southern regions had just commenced at the time of writing this report, and, it is currently expected that they will be held in Hamilton, Wellington and Christchurch respectively during late Q1 2019 or Q2 2019. The NZGS website will post further updates and detail as it become available. It should also be noted that YGP are welcome to attend any of the regional events, no matter what part of New Zealand they hail from.

Planning for several NZGS events, which are to be held during Q1 and Q2 of 2019, is well underway. All members are encouraged to review the "Calendar" page on the NZGS website with regular frequency to stay informed of any upcoming presentations which are relevant to them.

Events which are currently being organised by your management committee include:

Branch Presentations: "2018 NZGS Geomechanics Lecture: Misko Cubrinovski (University of Canterbury)". This lecture will be presented in Auckland, Hamilton, Wellington and Christchurch during Q2 or Q3 of 2019. The feasibility of presenting this lecture in two additional South-Island towns is also being assessed.

Symposium: "21st New Zealand Geotechnical Society Symposium". To be held in Dunedin during October 2020.

Further detail and updates on upcoming events will be provided, as appropriate, via the NZGS fortnightly email to members and the website (http://www.nzgs.org/).

Finally, with respect to potential CPD activities, I wish to remind all members of the upcoming ANZ Conference in Perth (01 to 03 April 2019) and the 20th International Conference on Soil Mechanics and Geotechnical Engineering in Sydney (12 to 16 September 2021). Further detail of these events is provided within this issue of the NZ Geomechanics News, on the NZGS website (http://www.nzgs.org/), and on the Australian Geomechanics Society website (https://australiangeomechanics.org/).

GUIDELINES AND STANDARDS PUBLICATIONS

The Earthquake Geotechnical Engineering Guidelines Series

NZGS has been a key participant in all seven of the Geotechnical Engineering Guideline modules published to date (Modules 1 to 6 and 5A).

A process to finalise the modules which are now in public circulation is scheduled to be completed during the 2018/2019 FY. I strongly encourage all NZGS members to submit any feedback or suggestions they have for improvement to NZGS and MBIE.

All ongoing feedback on the Geotechnical Engineering Guideline modules should be addressed to the email address modulefeedback@NZGS.org. Further detail on the module feedback and finalisation process is provided on the NZGS website.

The NZGS Management Committee has appointed Kevin Anderson and Ross Roberts/Sally Hargraves to the role of Project Manager and Editor for two new Geotechnical Engineering Guideline modules, Ground Anchors and Slope Stability, respectively. An expression of interest to participate in the development of these new guideline modules is expected to be issued to our membership via email, and posted onto the NZGS website, during Q4 of 2018 or Q1 of 2019. I encourage all members with a high level of expertise and experience in these areas to submit an expression of interest and collaborate in the development and issue of these important documents.

Seismic Assessment Guidelines (commonly referred to as "The Red Book")

The Seismic Assessment of Existing Buildings guideline has been published. However, a need to amend some of the structural engineering sections, in particular Section C5, has been identified. Gavin Alexander will continue to act as the NZGS liaison for the Seismic Assessment Guidelines project through the 2018/2019 financial year, and, will provide the NZGS Management Committee and membership with updates as appropriate.

NZ 1170.5 Standard Review

During the 2017/2018 financial year Rick Wentz (Wentz Pacific Ltd) continued to represent the NZGS, and Stuart Palmer (Tonkin & Taylor Ltd) acted as an observer, on the committee which is evaluating a proposed update or Amendment to 1170.5.

NEW LIFE MEMBER

With great pleasure I confirm that Ann Williams (BECA, Auckland) has been promoted to the position of Life Member within the New Zealand Geotechnical Society. Please join me in congratulating Ann for receiving this well-deserved honour.

REPORT CLOSURE

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

Tony Fairclough
NZGS Chair, 2017 - 2019

EDITORIAL

ON 14 NOVEMBER 2016, the M7.8 Kaikōura Earthquake severely damaged the North Canterbury region, closing both State Highway 1 (SH1) and the Main North Rail Line between Picton and Christchurch. Many of our members were actively involved in assessing the infrastructure damage caused by the earthquake, and in the design and supervision of repairs and improvements that were undertaken by the North Canterbury Infrastructure Recovery alliance (NCTIR). In this issue, some of the team share their experiences and some of the lessons learnt.

We are grateful to NCTIR for approval to publish the seven papers presented. But we are also mindful that our members undertake a wide range of work and generate innovative solutions on many other projects. Some also seek to raise issues and challenge us to address their concerns. This time, we have a challenge to parts of the Society's *Guideline for Field Description of Soil and Rock*, in particular to use of the Unified Soil Classification System (USCS). The challenge cast down is that *NZGS should develop its own standalone soil and rock logging guide, and reference to the USCS should be consigned to history.*

Such challenges are healthy, and rational debate is a great way of clarifying issues and making improvements. Please provide your thoughts (for or against) this challenge for a follow up article in 2019. Who knows where it may lead, or how your ideas may influence the outcome?

And looking even further ahead, start planning for your attendance at the 2020 NZGS Symposium to be held in Dunedin. It's going to be one to remember.

There is more information and other interesting stuff hidden in the magazine for you to find. It's like a treasure hunt. Enjoy.

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

NZ Geomechanics News co-editor

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

NZ Geomechanics News co-editor

Due to the success of the recent one day YGP symposium in Auckland, similar one day symposia are currently being organised for 2019 in the following regions.

AUCKLAND | CHRISTCHURCH | HAMILTON/TAURANGA | WELLINGTON

These events are shortened, local versions of the highly successful ANZ YGP conference for the Young Geotechnical Professional (under 35 & less than 10 years' experience) members of NZGS.

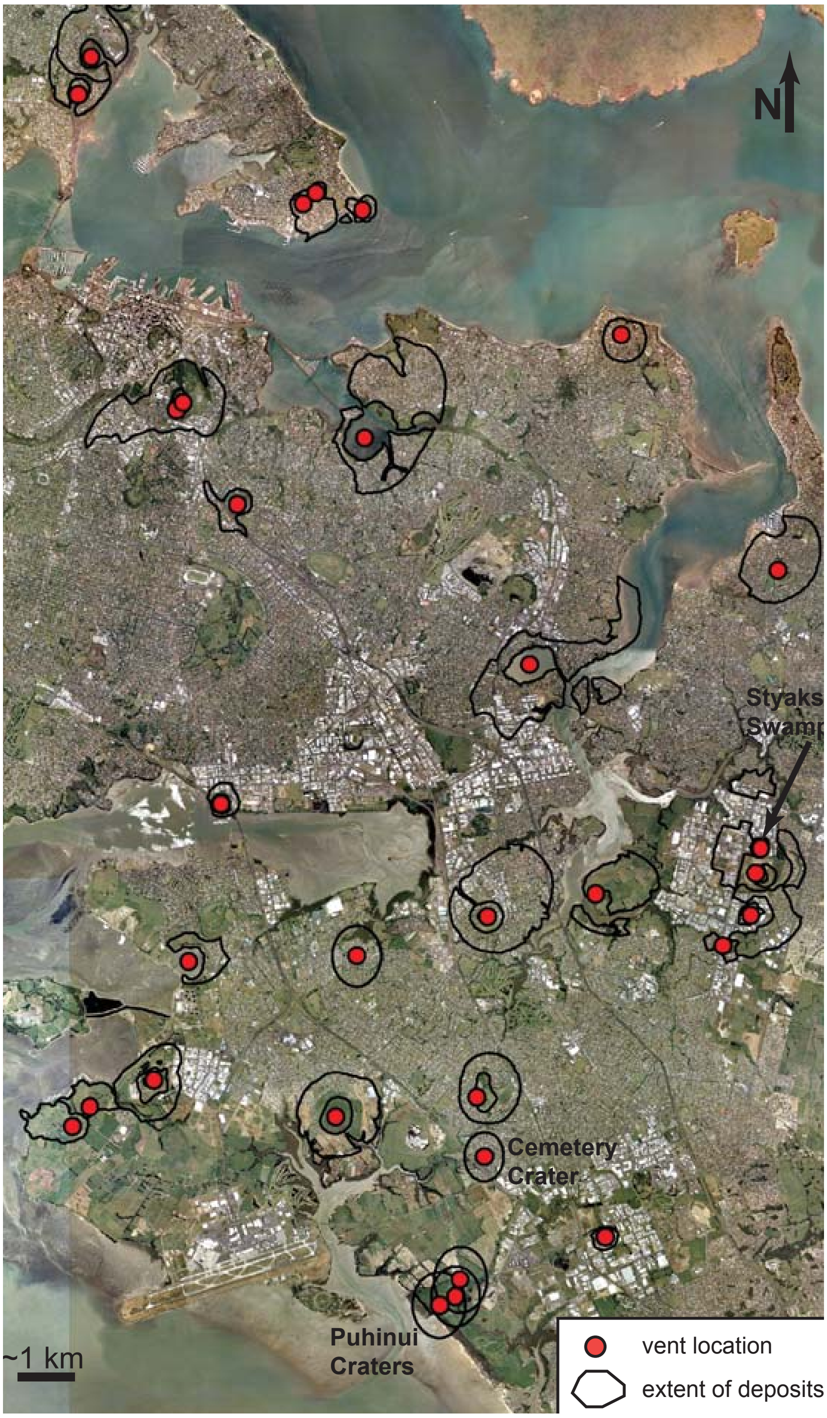
TO REGISTER YOUR INTEREST CONTACT **regional.ygp@gmail.com.**

Please provide a 200 word summary of a geotechnical topic on which you will be able to make a 10-12 minute presentation at the event.

Please look out for further information about these events in the upcoming NZGS Weekly Newsletters.

News – In Brief

REWARD: PRIDE AT HELPING HAZARD SCIENCE, & OUR GRATITUDE!



WANTED: AUCKLAND VOLCANIC FIELD BASALT AND ORGANIC MATERIAL

Volcanologists at the University of Auckland and GNS Science are searching for lava and organic matter samples from various volcanoes in the Auckland Volcanic Field (AVF).

Fig. 1: Tank Farm – *Onepoto*; Mt Cambria – *North Head*; Mt Victoria – *The Domain;* Grafton – *Mt Hobson*; Orakei – *St Heliers*; Panmure – *Te Hopua*; Otara Hill – *Styaks Swamp**; Green Mount – *Hampton Park*; Mangere Lagoon–*Mt Robertson*; Pukewairiki–*Waitomokia*; Otuataua–*Pukaki*; Cemetery Crater*–*Puhinui Craters**; Ash Hill–Kohuora
* high priority volcanoes

Our aim is to date every volcano in the field to get a better understanding of how the field works, what we can expect from future eruptions, and to prepare and mitigate the volcanic risk to the population and infrastructure.

As part of the Determining Volcanic Risk in Auckland (DEVORA) programme, scientists have now dated ~3/4 of the AVF volcanoes, but there are still many gaps in our knowledge!

How can you help?
To reach our ambitious goal, we still need lava and/or organic samples from the following volcanoes, whose locations and deposit extents are shown in the map to the left

WHAT DO WE NEED?
Sample of >400 g lava (see Fig. 2) or >1 g organic material (peat, charcoal, wood)

- Location of sample (map coordinates)
- Drilling log (to ascertain depths, thicknesses, and stratigraphic order)

We are looking for dense, crystalline lava with few vesicles and a 'sugary' texture (see 2b, above). This kind of lava is found in the center of lava flows and usually breaks off into large (5-15 cm thick) plates. For organic material, we are looking for wood, charcoal, or peat that is either contained within a volcanic tuff layer or lies directly above/below a lava flow or tuff layer.

Other Important Information
Any and all dates obtained via the project, and their implications, will be made freely available to the public upon publication of the data.

Contact Us!
If you find lava or organic material around the listed volcanoes, please contact: Dr. Tracy Howe (t.howe@auckland.ac.nz) or Elaine Smid (e.smid@auckland.ac.nz). For more information about DEVORA, please check out our webpages:
www.devora.org.nz
www.facebook.com/DEVORAProject

GEOTECHNICAL INVESTIGATIONS FOR DAMS

Draft *Practice Notes on the Geotechnical Investigations of Dams, their Foundations, and Appurtenant Structures* were presented to a workshop at the recent ANCOLD 2018 Conference in Melbourne.
The purpose of the Practice Note is to benefit dam owners through more effective and targeted investigations to answer key questions about dams and their foundations. Among other things, it describes common issues and objectives of geotechnical investigations for dams, outlines ways to determine the appropriate scope and types of investigations, and provides general advice on appropriate methods. It does not provide details of the investigations methods but directs readers to suitable sources of information.
ANCOLD are working towards completion of the document which espouses principles that apply to all large geotechnical investigations projects.

21st NZGS National Symposium – Dunedin 2020
"Good Grounds for the Future"

The 21st Symposium of the New Zealand Geotechnical Society will take place between 14 and 18 October 2020 in Dunedin. In this Symposium we will strive to explore the challenges and opportunities of our future by learning from the failures and achievements of our past.

Symposium theme

The theme of the Symposium is inspired by the profound changes currently experienced internationally and in New Zealand:

- Natural disasters associated with climate change effects, are becoming increasingly frequent and severe.
- The learnings from the recent Canterbury and Kaikōura seismic events are shifting our thinking in New Zealand with respect to the way we perceive and address the seismic risk.
- Increasing urbanism and growth of population around the cities create more needs for capacity and resiliency of lifelines.
- The fragile international economy is limiting available resources, and potential changes in insurance practices and cover policies require a different approach in achieving resilience for our communities.

These are pressing future challenges for the geotechnical profession, but also great opportunities. The profession is playing a key role in achieving sustainability of our built environment and communities.

At the same time, new technologies, such as intelligent real time remote monitoring systems, Unmanned Aerial Vehicles and computer power create new capabilities. New wireless communication technologies enable dissemination of knowledge and collaboration, without geographical or other barriers. New regulation and guidelines framework provide an invaluable tool for benchmarking geotechnical engineering practice.

Our past failures and achievements can provide invaluable learnings and assist us prepare for the future challenges and make the most of the future opportunities.

Symposium purpose and format

The Symposium aims to attract the most recent developments in geotechnical engineering, and associated disciplines in New Zealand and internationally. We look for stimulating technical discussions and out-of-the-box ideas in the fields of engineering geology, geotechnical engineering, seismology, geophysics, new technologies.

The Symposium will comprise a pre-symposium half-day workshop, one-day field study, and two days of technical talks and discussions. We will use a range of formats: keynote speakers, plenary special technical sessions and discussions, speaker presentations, and poster exhibitions.

Collaboration and communication with structural and infrastructural engineering will be key component of our technical sessions.

Location

The selected location for the 21st NZGS Symposium is Dunedin, New Zealand's best kept secret. Dunedin is a city with unique character architecture and

heritage buildings steeped in history, with abundance of wild life and spectacular country on its door step.

During the Symposium we will seek opportunities for exploring and enjoying the beauties that the city and surroundings can offer.

Pre - Symposium workshop and field study

We are preparing a special gateway to Dunedin for the Symposium Delegates. We are planning a special workshop in Queenstown, prior to the Symposium, on Wednesday 14 October 2020. The workshop will be focused on landslide recognition, interpretation, risk and mitigation and state-of-the-art monitoring systems. It will be presented by local and international experts.

The workshop will be followed by a field study / trip on the next day, Thursday 15 October 2020.

The trip will start from Queenstown through Cromwell Gorge and beautiful Maniototo, where local experts will show us how past excellence in geotechnical practice can educate and inform our future development.

The field study will be completed with a train trip from Pukerangi to Dunedin, through the spectacular Taieri Gorge. Local GNS experts will guide us through the trip and talk about landslides and other interesting geological and geotechnical features of the gorge.

The trip will finish at the iconic Railway Station in Dunedin, right on time for the Symposium Welcome Reception held at the Settlers Museum, opposite the Station.

Communications

The Symposium website is expected to be launched in March 2019. Through the website we will provide all the communications about technical topics, key dates, costs, venue and accommodation.

We will be also issuing regular newsletters to keep you up to date with all developments.

We strive to make the next NZGS Symposium to be “Your Symposium”. Please get in touch with us, we are keen to listen to your suggestions and ideas and we encourage you to contribute in any way you feel you can.

Please contact the Organising Committee through Eleni Gkeli, Symposium Convener and Committee Chair, by email at **Eleni.Gkeli@wsp-opus.co.nz** and by phone on 021 192 0296.

Autodesk Geotechnical Module 2019 released

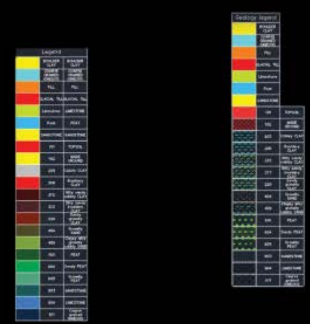
The latest version of Autodesk’s Geotechnical Module enables users to update models faster and present complex geological models more easily than ever before.

“A major improvement to the module means, for the first time, users can update existing borehole information. Boreholes can simply be removed and the updated data imported, which is automatically included in models and profiles,” said Keynetix Technical Director Gary Morin.

Licensed by Keynetix to Autodesk, the Geotechnical Module is a free add-on to AutoCAD Civil 3D. It is a cut down version of HoleBASE SI Extension and can connect to HoleBASE SI to give CAD and BIM teams instant access to centrally-held geotechnical data. It can also be used as a standalone product.

“There are also a number of new features to allow users to customise drawings and profiles,” Morin added. “Geological strips can be moved to avoid them overlapping when boreholes are close together; users can create their own geological legend keys and they can re-order strata, plus we have made a number of improvements to the profile generation tools.”

The new version of the Geotechnical Module can be downloaded from users’ Autodesk Management Console at https://manage.autodesk.com/cep/#products-services/updates and to subscribe to Keynetix’s free Geotechnical Module training, visit http://geotechnicalmoduletraining.keynetix.com/.



XIII IAEG CONGRESS - SAN FRANCISCO 2018

Greg Stock discusses rockfall risk mitigation at Yosemite National Park

THE XIII IAEG CONGRESS was held in San Francisco, California between 17 and 21 September 2018. It was the first time the IAEG Congress has been held in the USA. Organised and run in partnership with AEG the Congress included a diverse range of presentations across the spectrum of engineering geology practice with over nine hundred attendees.

The programme included specific sessions on dams, natural hazards, rock mechanics and tunnelling, aggregates, marine engineering geology, slope stability and GIS remote sensing. Sessions were also held on education and training for engineering geology as well regulations, registration/chartered member status.

There was a little bit for everyone in the programme and a day of midweek field trips provided both local colour and the chance to experience some geology first hand as well savouring the sights, sound and flavours of California. The programme with abstracts can be viewed online at https://issuu.com/aeg275/docs/aeg_2018_pwa. The full proceedings have been published in six volumes by Springer and are available online for about USD $170 each volume.

The congress had a good kiwi turnout, with around 20 people from universities, research institutes and industry. With representatives in the Executive Committee, Technical Commissions and Young Engineering Geologists Committee, New Zealand continues to be a well-represented and very active National Group in the IAEG.

Pedro Martin

Pedro is a senior engineering geologist with Beca in Tauranga. He is the Australasian representative on the Young Engineering Geologists Committee of the IAEG.

Above: YEG members at the Young at Heart event

CONGRESS HIGHLIGHTS

The AEG/IAEG partnership provided a great local flavour to the Congress and captured the crème de la crème of the international engineering geology community. Keynote lectures and fieldtrips were highlights of the Congress.

KEYNOTE LECTURES

Two keynote lectures were presented each morning covering the latest developments in engineering geology research and practice.

My personal picks were:

- *Long and Short-term Response of Rock Slopes to Deglaciation.* Dr. Simon Löw (ETH Zurich, Switzerland) – note Dr Löw is coming to New Zealand so look out for his lectures in late November and early December!
- *The Chain of Geohazards Induced by the 2008 Wenchuan Earthquake – Ten Years of Lessons, Advances, and Challenges.* Dr. Runqiu Huang (Chengdu University of Technology, China)
- *Engineering Geology Considerations for Stability Assessment of Rock Slopes Adjacent to Infrastructure.* Dr. Jean Hutchinson (Queen's University, Canada)

FIELD TRIPS

With 20 field trips both pre, mid and post-congress, there were great options for all tastes. Two of the most popular trips were to Yosemite National Park and West Napa Earthquake and Wine Region (you can guess why). The walking tour of Signal Hill and surrounding projects was also a gem and surprise for those that did not want to travel far. Somehow all trips found a watering hole (or two).

The Yosemite field trip was led by Pete Holland, Engineering Geologist from the California Geological Survey and the group met Greg Stock, Yosemite Park Geologist, on site. Greg explained the challenges of managing 4-5 million visitors per year in an area subject to flood and rockfall hazards. He discussed their recent work on rockfall risk mitigation and presented some pretty serious rockfall monitoring systems (which even use data gathered by climbers as they make their way up the famous Yosemite climbing routes!). For more details visit https://www.nps.gov/yose/learn/nature/rockfall.htm

YOUNG ENGINEERING GEOLOGISTS COMMITTEE UPDATE

Earlier this year the IAEG officially created the Young Engineering Geologists (YEG) committee, whose objective is "*to ensure the future of the IAEG through the promotion of the interests of young engineering geologists (YEGs) and their increasing involvement in the activities of the Association*".

The YEG committee was involved in planning the activities for young members at the Congress. With support from the locals (Morley Beckman in particular) the Congress had a long list of official (an unofficial – even better!) YEG events. Obviously my favourite pick was the Young at Heart Happy Hour at Pedro's Cantina!

As part of the young members' activities, the Richard Wolters Prize is always a highlight. Once again NZ was well represented, and Sarah Bastin from the University of Canterbury was runner up. First place went to Wei-An Chao from Chinese Taipei.

Institutionally we made good progress and for the next term the chair of the YEG will be part of the Executive Committee as an ex-officio member. This means that views and needs of our young members will be directly represented at the highest decision-making level of the IAEG.

For now the main challenge for the YEG is to strengthen ties with young members from National Groups worldwide and to consolidate itself as an international network of young professionals.

For further information on YEG activities visit http://iaeg.info/young-engineering-geologists/ or drop me a line at pedro.martins@beca.com.

This was an event worth attending so start saving for the next Congress in 4 years' time in Chengdu China – promises to be just as good.

12th Australian & New Zealand Young Geotechnical Professionals Conference

Above: The delegates of 12YGPC at the Upper Deck Mures Restaurant for the conference dinner in Hobart waterfront
Middle left: Members of 12YGPC senior industry mentoring panel: Prof. Stephen Fityus, Ross Roberts and Darren Paul (front row from right to left).
Left: From left to right: Sam Glue (NZGS' Young Geotechnical Professionals Fellowship Award), Nicola Manche (AGS' Don Douglas Youth Fellowship Award) and Alexander Rogan (12YGPC 2018 People's Choice Award)

THE TASMANIAN CHAPTER of the AGS welcomed 45 delegates, 3 mentors, 5 sponsors, the National Secretary of AGS (Australian Geomechanics Society) and 4 organising committee members to the 12th Australia and New Zealand Young Geotechnical Professionals Conference (12YGPC), which was held at the Old Woolstore Hotel, Hobart, Tasmania, from 6th to 9th November 2018.

The 45 delegates aged 35 years or under were selected from a record number of 104 abstract submissions. Ten delegates with papers relevant to natural hazards in NZ were selected for jointly funded NZGS-EQC scholarships. The generous support from these organisations highlights the significance of understanding our natural hazards in NZ.

Held every two years, the YGP Conference is rather unique in that only 50 delegates attend and present. The small size allows young professionals a chance to hone their presentation skills in a supportive, constructive environment. Furthermore, social events allowed our delegates a chance to get to know each other and also unwind after the stress of presenting. Social events included welcome drinks, lunch and morning tea breaks during the day, two dinners, and a field trip.

The presentations were engaging and diverse, covering both research and practice. Each delegate presented in a 15 minute slot, with a strict 10 minute presentation time, followed by 5 minutes of questions and discussion from the audience. The timeslot was seldom exceeded, and when it was, presentations generally ended abruptly due to constant ringing of a bicycle bell provided by Colin. 5 minutes proved to be sufficient time for much stimulating discussion, which was very satisfying and entertaining to partake in for all.

Our senior industry mentors (Professor Stephen Fityus (AGS National Chair & University of Newcastle), Ross Roberts (NZGS Vice Chair and Treasurer & Auckland Council), and Darren Paul (Former AGS National Chair & Golder Associates)) very generously donated their time to judge each presentation and choose the winners of the AGS (Don Douglas) and NZGS (Young Geotechnical Professional Fellowship) awards. The mentors provided feedback to the group as a whole and individually. Concentration on individualised mentor feedback was a new feature of the conference, and it was very well received by delegates.

Above: Dolerite columns with irregular weathering forms and precarious rocks on the plateau

Above: Boulder of dolerite with Pinnacle track in foreground

After several hours of hard deliberation by our mentors, the awards were presented at the conference dinner at Mures Upper Deck Restaurant, Hobart. The New Zealand Geotechnical Society's Young Geotechnical Professionals Fellowship was awarded to Sam Glue of Tonkin and Taylor in Christchurch, for his work in designing a geogrid reinforced gravity seawall in New Zealand's high seismicity environment. The Don Douglas Youth Fellowship Award was award to Nicola Manche of Golder Associates Pty Ltd in Brisbane for her work to optimise detailed design using early-works embankments in soft soils areas. The 12YGPC 2018 People's Choice Award was given to Alexander Rogan of Pells Sullivan Meynink in Brisbane, for his case study on impact of jet grouted column variability on a base block in sand. In addition, Darron Lee of EDG Consulting Pty Ltd in Brisbane and Alexander Rogan received the Honourable Mentions Awards of AGS while Francesca Spinardi of University of Waikato and Lauren Foote of ENGEO in Christchurch named the Honourable Mentions Awards of NZGS.

Sam and Nikki will have the opportunity to present their papers and to represent New Zealand and Australia, respectively, at an upcoming international conference.

After a night enjoying the hospitality scene of Hobart which included renditions of Dave Dobbyn's Welcome Home and Waltzing Matilda, fresh air, fantastic views and interesting geology up Kunanyi / Mount Wellington was highly enjoyed by attendees on Friday morning. The field trip was led by Colin Mazengarb, and his colleagues from Mineral Resources Tasmania (Tasmanian Government). Mount Kunanyi boasted some impressive rock formations, mass wasting features, and landslides and views of Hobart and surrounds.

To sum up, 12YGPC was an outstanding conference, and one that I will personally always remember fondly. It was a learning experience not only for delegates, but also organisers, with several grey areas and challenges being encountered along the way.

The 12YGPC saw the rising of future leaders and a bright future of trans-Tasman geotechnical professions. We look forward to 13YGPC to be held in Australia, 2020.

Reported by:
Philippa Mills (NZGS YGP representative and engineering geologist at Coffey NZ Ltd.) and Dr Hong Liu (AGS Tasmania representative and lecturer at University of Tasmania).

Proudly sponsored by
PLATINUM SPONSOR
coffey
A TETRA TECH COMPANY
GOLD SPONSOR
KELLER
SILVER SPONSOR
SEEQUENT
BRONZE SPONSOR
Tasmanian Government
And
THE OLD WOOLSTORE APARTMENT HOTEL

1st Auckland & Northland Young Geotechnical Professionals' Symposium

Below: Winners 1 - Best presentations: Tiana Epati & Áine McCarthy

Above: Winners 2 - Spot prize (Scala lifter from Underground Investigations) - Tobias Francis

THE INAUGURAL AUCKLAND & Northland Young Geotechnical Professionals' Symposium was held on 27 September 2018 at the Grand Mercure Hotel. Throughout the day each of the 15 attendees delivered a 10-minute presentation, suffering the fate of Grant Murray with a Nerf gun if they ran overtime, before answering a few questions. A good day was had by all, with the chance to learn from and get to know a few more of their peers. The day was finished off with dinner and awards at Mexico.

Grant's pick for best presentation went to Tiana Epati from Hawthorn Geddes for her presentation "How to create an environment to achieve successful outcomes when working with contractors". The people's choice award went to Áine McCarthy from Jacobs for her presentation "Characterising the settlement potential of Auckland volcanic ash". Tobias Francis of GWE Consulting Ltd picked up the spot prize of a Scala Jack donated by Underground Investigation.

We were joined by YGP representatives from Hamilton, Wellington and Christchurch so similar events are likely to run elsewhere in the first half of next year.

Thanks to Geotechnics and Underground Investigation who sponsored the day and to Grant Murray of JGM Associates and Ross Roberts of Auckland Council for providing mentorship.

Reported by:
David Buxton & Áine McCarthy

13th Australia New Zealand Conference on Geomechanics

Perth Convention & Exhibition Centre

Perth, Western Australia • 1-3 April, 2019

AUSTRALIAN GEOMECHANICS SOCIETY

NEW ZEALAND GEOTECHNICAL SOCIETY INC

SIMSG ISSMGE

IAEG AIGI ENGEOL

ISRM

Our keynote speakers

2018 NZGS Geomechanics Award Lecture:
Professor Misko Cubrinovski
University of Canterbury, Christchurch, New Zealand
Assessment of soil liquefaction: critical factors that make the difference

Professor Fumio Tatsuoka
Professor Emeritus, University of Tokyo and Tokyo University of Science
Geosynthetic-reinforced soil structures for transportation – from walls to bridges

Dr Oskar Sigl
Managing Director
Geoconsult Asia Singapore
Dealing with the challenges of underground construction

Mike Jefferies
Civil Engineer and Senior Consultant ('Emeritus') of Golder Associates
TBC

Marc Woodward
Senior Principal Geotechnical Engineer
CMW Geosciences
Effective communication – A critical component of geotechnical engineering

Rob Day
Associate Principal, Geotechnical, Arup
2016 AGS Practitioners Award Lecture
Trying to make a difference – and why sometimes we can't

Dr Doug Stewart
Principal Geotechnical Engineer, Golder
Unexpected ground movements and their impact

155 Full Papers over 3 Days

Topics include:

- Rock mechanics / mining
- Soil mechanics / soft soils
- Offshore geotechnics
- Construction / practical / case studies
- Geotech design / numerical modelling
- Pavements / rail
- Walls / slopes
- Railway geotechnics
- Unsaturated soil mechanics

@ anzgeomechanics2019@arinex.com.au

www.geomechanics2019.com.au

Sponsors include

Gold Sponsor

IGS Insitu Geotech Services

Bronze Sponsor

Beca

Coffee Cart Sponsorship

wsp

Pocket Program Sponsor

KELLER

Silver Sponsors

GEOBRUGG BRUGG Safety is our nature

GEOFABRICS Smarter Infrastructure

GOLDER

KELLER

PROBEDRILL GEOTECHNICAL SURVEY

USB Sponsor

KELLER

Promote your brand!

Bring your organisation to the forefront of the industry and stand out in the crowd of your competitors. Sponsorship and exhibition opportunites are still available to extend the reach of your brand and achieve premium exposure. To discuss opportunities available, please contact the Arinex Sponsorship and Exhibition team on sponsorship@arinex.com.au or by calling **+61 2 9265 0700**

Accomodation

From $159 per night:
Special accomodation rates available through your registration.

Experience Perth & Western Australia in Autumn

Attend ANZ Geomechanics and extend your stay to experience Western Australia with direct flights daily.

- Two world renowned wine regions
- A magical aquatic playground along the West Australian coastline
- Bushwalks through the Hills
- Explore Perth City or take a cruise along the Swan River to Fremantle

Features / Topics

- ISSMGE CAPG Workshop on 'Collaboration in Geotechnical Engineering'
- Poster sessions
- 200+ papers in proceedings
- Gala dinner at Fraser's, State Reception Centre, Kings Park

Sponsorship Opportunities

Are you interested in sponsoring and securing your exhibition space?
Contact us on +61 2 9265 0700

Conference Managers: **Arinex Pty Ltd**
3/110 Mounts Bay Road, Perth, Western Australia 6000 Tel: 08 9486 2000
Sponsors are correct as off 6 Nov 2018.

Registration NOW OPEN

anzgeomechanics2019@arinex.com.au

www.geomechanics2019.com.au

Figure 1: Image of Lyall Bay in Wellington that has been modelled by blending aerial imagery, a digital surface model and building footprints.

A wealth of government geospatial data is online and free to use. Land Information New Zealand (LINZ) wants engineers to take full advantage.

NATIONAL MAPPING AGENCY LINZ provides authoritative charts and topo maps, but is also now focused on publishing geospatial data that models New Zealand's built and natural environments in detail. The agency shares its datasets—ranging from road and river data to aerial imagery and LiDAR—on data.linz.govt.nz.

Roger Carman, LINZ's Group Manager for Topography, sees the engineering community as key customer group. "Our strategy is to maximise the value that New Zealand gets from our data, products and services by improving them and making them open for re-use. Engineers are already a major customer group for the online data, and we want this use to grow. LINZ publishes terabytes of data that will provide valuable insight and context for your projects."

Right now, the LINZ Data Service provides:

- Aerial imagery covering 95% of NZ
- A growing number of LiDAR datasets
- Topo map data layers and hydro charts, addresses and property data
- A new dataset of building outlines.

LINZ and councils are also half way through digitising NZ's archive of 600,000

Figure 2: Aerial image of Lyall Bay. You can find imagery datasets at data.linz.govt.nz. Start at the NZ Imagery surveys dataset, which brings together information to help you determine what imagery data is available for a particular location.

Figure 3: A digital surface model of Lyall Bay, Wellington. LINZ publishes both surface and ground models and the raw underlying LiDAR points.

Figure 4: Building outlines data for Lyall Bay. Building outlines is a new dataset available at data.linz.govt.nz

Figure 5: Historical aerial imagery of Lyall Bay that is available at retrolens.nz

historical aerial photos, which provide a record of land use change that dates as far back as far as 1936. You can find the images online at retrolens.nz

Alongside the LiDAR available on the LINZ Data Service, LINZ also publishes raw point cloud LiDAR data on opentopography.org. A user recently calculated that there is now data covering 18,000 sq km of New Zealand available on this site.

Coming to the LINZ Data Service in the future:

Aside from the comprehensive data now available, Roger would like engineers to keep an eye on these platforms because LINZ is planning to publish:

- Region-wide LiDAR data for Northland and the East Coast that is currently being collected
- New updates of aerial imagery as it's collected by councils
- A new dataset for a complete, connected river network.

List of links mentioned in this article:

- data.linz.govt.nz for all LINZ data
- retrolens.nz for historical imagery
- opentopography.org for raw LiDAR point clouds.

Introduction

THE 14 NOVEMBER 2016, M7.8 Kaikōura Earthquake severely damaged the North Canterbury region. The earthquake resulted in significant ground shaking triggering widespread landslides, extensive coastal uplift on multiple faults, and a locally sourced tsunami. The earthquake caused significant damage over a very large area, closing both State Highway 1 (SH1) and the Main North Rail Line between Picton and Christchurch.

The North Canterbury Transport Infrastructure Recovery (NCTIR) was set up by the government under the Hurunui/Kaikōura Earthquakes Recovery Act 2016. Its immediate purpose was to repair and re-open the earthquake-damaged road and rail networks on the Kaikōura Coast and the inland road (SH70) by the end of 2017. The full scope of work managed and delivered by NCTIR is summarised below:

1. Establish access to Kaikōura via SH1 south and Inland Road (Route 70) to reconnect the Kaikōura community, and enable Kaikōura to reopen as a tourist destination.
2. Strengthen and manage the roading infrastructure on the SH1 alternate route (via Lewis Pass) to cope with the extra traffic and to improve safety and journey time for customers, including:
   a. Provision of a coordination role to the management and incident response needs
   b. Improvement works to maintain an acceptable level of service for the increased traffic volumes
   c. Provision of a coordinated management and incident response on the alternate route Picton to Christchurch Lewis Pass
   d. Coordination of maintenance and incident response on the Inland Road (Route 70)
3. Re-connect the road and rail links from Kaikōura to Picton, which provide critical transport routes for freight and tourism industries to Kaikōura as well as vital access for many local communities positioned between Christchurch and Picton.
4. Restoration of the Transport System along the coastal route between Cheviot and Clarence including:
   a. Design to meet the requirements of KiwiRail and the NZ Transport Agency
   b. Remove slips and other damaged assets
   c. Construct the works – rail and road
   d. Maintenance of the safety of the public and workforce throughout
5. Design and construct safety, resilience and tourist amenity improvements on SH1 between Oaro and Clarence.
6. Reinstate a safe, functioning Kaikōura harbour.
7. Provide environmental, planning, consenting, and stakeholder management for the above works

Many members of the NZ Geotechnical Society were actively involved in the documentation of the infrastructure damage caused by the earthquake, and subsequently in the design and supervision of the repairs and improvements for resilience and reliability that were undertaken by NCTIR. In this issue, some of the team share their experiences and the lessons learnt. We are grateful to NCTIR for approval to publish the seven papers presented.

Kaikōura Earthquake Slope Hazards - Risk Mitigation and Network Resilience

Richard Justice

Richard is the geotechnical principal in NCTIR for 'Kaikōura South', being responsible for slope hazard mitigation works for the transport corridor between Oaro and Peketa. At his 'home organisation' of ENGEO, Richard is a Principal Engineering Geologist in the Christchurch office. Areas of special interest include landslide and slope stability assessment, geotechnical risk assessment, geomorphological and geological interpretation for corridor projects.

Greg Saul

Greg is a Principal Geotechnical Engineer with WSP-Opus in Christchurch. He has over 30 years of engineering experience on variety of infrastructure projects around New Zealand, specialising in stability assessment, stabilisation and foundation design. He was involved in the initial post Kaikōura Earthquake early response for highways and recently the North Canterbury Infrastructure Recovery works.

Doug Mason

Doug is a senior engineering geologist within Opus' Wellington Civil Infrastructure Group, and leads a team specialising in natural hazards, infrastructure resilience and GIS. Doug has over 12 years' experience in hazard assessment, resilience and risk assessments, and geotechnical investigations for lifeline infrastructure and land use planning projects.

Figure 1: Site locations with coastal corridor sections indicated

1. INTRODUCTION

More than 20 faults are inferred to have moved in the M7.8 November 2016 Kaikōura Earthquake, which resulted in extensive damage to the coastal slopes and caused uplift at several locations. Between Clarence and Oaro, the transport corridor is located on a narrow coastal strip of land between steep mountainous slopes and the Pacific Ocean. Over 80 landslides occurred along this 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 Kaikōura and 14 km on the northern section of the coast (Figure 1) were most heavily affected.

An extensive geotechnical program was commenced by NCTIR in January 2017 to re-establish this nationally vital portion of New Zealand's transport infrastructure as quickly as practicable. The intent of this paper is to provide a background of the assessment of slope hazard and resulting risk mitigation works carried out as part of the NCTIR recovery program.

2. GEOLOGICAL SETTING

Basement 'Greywacke' of the Pahau Terrane forms the hills along the coast south of Kaikōura and much of the North Kaikōura Coast. The Greywacke typically comprises slightly weathered sandstone and mudstone (argillite), often with a mantle of moderately weathered rock close to the ground surface. The mudstone is typically weak and the sandstone is moderately strong to strong. At the northern end of the project area, the basement rock is Tertiary-age slightly weathered, calcareous siltstone with minor silty limestone interbeds and is typically very weak to weak.

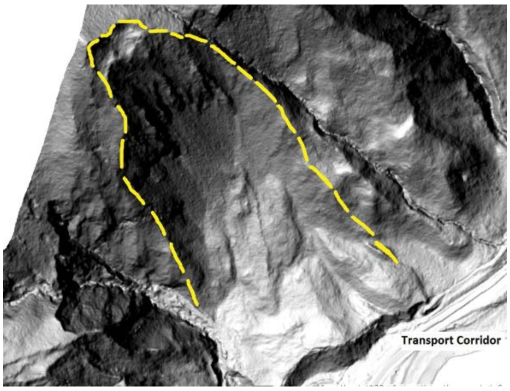


Figure 2: A large scale prehistoric landslide (scarp outlined in yellow) identified in LIDAR DEM

Colluvium overlying the Greywacke is typically a mixture of rock fragments, silt and sand. It is widely distributed over the basement rock throughout the project area, where slope angles are less than 45°. The colluvial mantle is typically 0.5 m to 1 m thick near the ridge tops, and increases in thickness downslope, with a maximum observed thickness of approximately 15 m.

The presence of large, pre-existing landslides on the slopes facing the coast is evident in both published topographic contour maps and in LiDAR data obtained after the earthquake (Figure 2). Rapid coastal erosion was likely occurring as sea level was rising until about 6,000 years before present. This toe erosion is inferred to have had a major destabilising effect on the coastal slopes, initiating the landslides and keeping them active. Tectonic uplift of the coastline in the last 6,000 years (likely associated with earthquakes similar to the Kaikōura earthquake) has stranded the landslides above the eroding effect of the sea and reduced the destabilising effect. Consequently, the large landslides are now much less active than they have been in the past, and their behaviour is driven by climatic and tectonic events rather than coastal erosion.

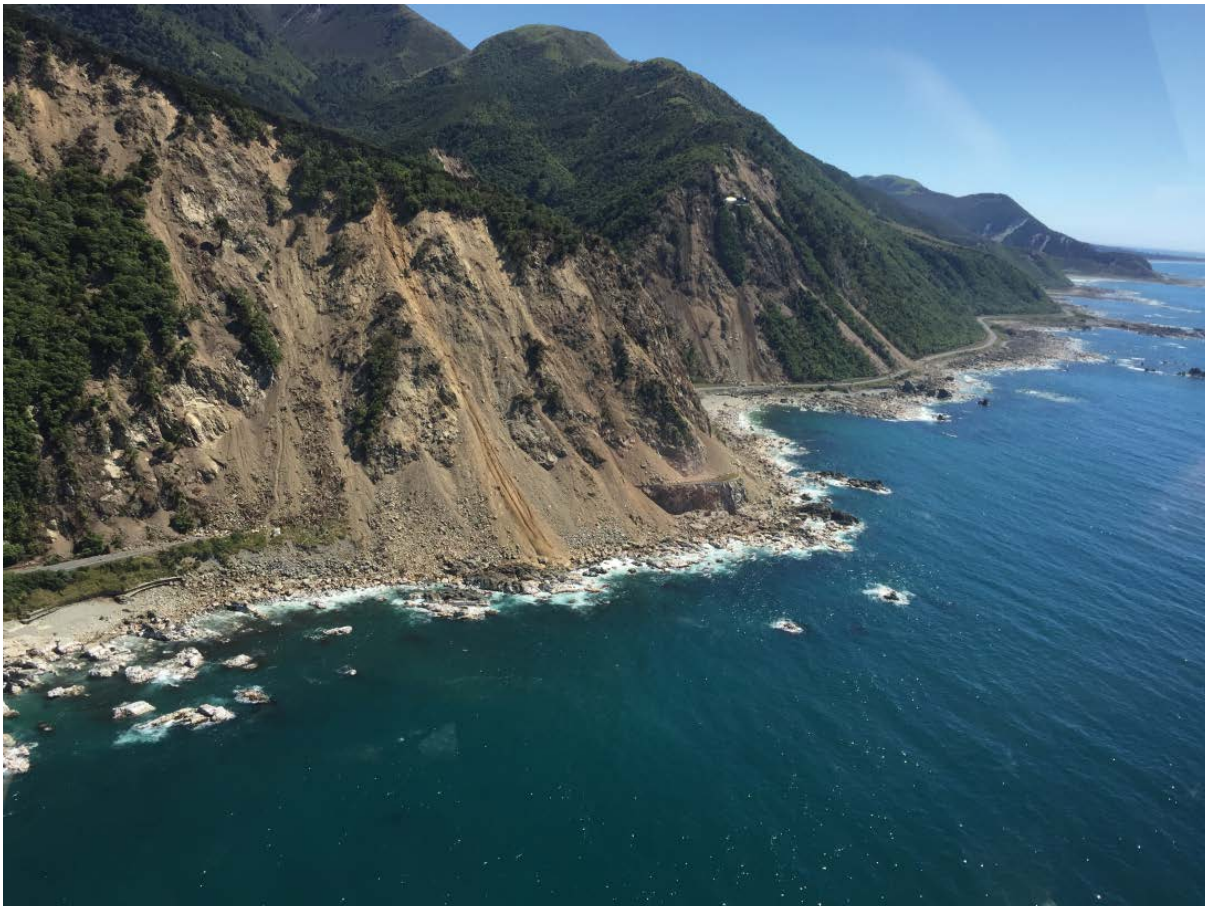


Figure 3: Slip P6, Ohau Point Kaikōura North, buried SH1

3. EARTHQUAKE EFFECTS AND POST-EARTHQUAKE OBSERVATIONS

The combination of rapid tectonic uplift, coastal erosion and oversteepening of the slopes to form the road and rail corridor had rendered the slopes vulnerable to instability. This was demonstrated in the Kaikōura earthquake and historically in rainstorm events, such as occurred in Cyclone Alison in 1975 (Bell 1976).

Fault movements during the Kaikōura Earthquake caused disruption and displacement of the road and rail formations and structures at several locations, but the main impacts on road and rail were due to earthquake shaking and associated slope failures.

The majority of the slope failures caused by the Kaikōura Earthquake were evacuative rock and debris avalanches (Figures 3 to 6) that, in some cases, involved the release of large volumes of material (in excess of 50,000 m^3 at Ohau Point for example, Figure 3). Between these large failures, smaller volumes of slope materials or rock were released downslope. Some of this debris reached the road/rail corridor, but in many cases this material has halted on well vegetated hillslopes which are around 35° to 40°. In addition, widespread hillside and ridge cracking occurred during the earthquake, without downslope release of material.

Figure 4: Slip P4, Kaikōura North, buried SH1 and displaced the rail track

Slope hazards affecting the transport corridor were typically classified by the NCTIR geotechnical team in accordance with the Varnes Landslide Classification, updated by Hungr et al (2013). Four principal slope failure styles were typically observed, as follows.

Figure 5: Rock and debris avalanche at the south portal of Rail Tunnel 13, Kaikōura South

Figure 6: Slip P7A, Kaikōura North, buried road and rail. Emergency works underway

3.1 Large Scale Landslides

Pre-existing degraded landslide headscarps and evacuated slopes are common along the coastline, although they are generally well vegetated and difficult to recognize on the ground. While in some cases co-seismic deformation of several metres appears to have occurred during the Kaikōura Earthquake, typically these prehistoric landslides have shown no evidence of post-tectonic movement. Reactivation is therefore anticipated to only occur co-seismically under very large earthquake events, with displacements expected to be up to several metres.

3.2 Rockfall and Rock Avalanche

Rockfall triggered by strong shaking was widespread throughout the project area (Figure 7). Source areas included rock outcrops and deposits of boulder colluvium. The scale of rocks observed ranged from fist-sized cobbles to large boulders up to several metres across. The associated ground damage effects of earthquake shaking on the hillslopes included denuding of soil slopes, tension cracking and dilation of rock outcrops, all of which have increased the potential for future rockfall onto the transportation corridor.

Figure 7: Rockfall debris at site P2 near Irongate Stream, Kaikōura North

3.3 Shallow Colluvial Landslides

Shallow soil failures typically involved failure of the colluvial mantle and weathered highly fractured upper parts of the rock mass, as indicated in Figure 8. After the earthquake, these failures were typically reactivated by or immediately following periods of extended rainfall.

Figure 8: Shallow soil failure with displaced soil block; Site SR26, Kaikōura South

Figure 9: Debris Flow at Jacob's Ladder, Kaikōura North, following Cyclone Gita (February 2018).

3.4 Debris Flows

The transportation corridor has a history of debris flows triggered by prolonged or intense rainfall in steep, erosion-prone catchments that have inundated the road and railway (e.g. Bell, 1976).

The November 2016 earthquake triggered many landslides that did not reach the road or rail, or failed into streams flowing towards the coast. In several locations the exposed slip debris and colluvium on the slopes have since been mobilised by high intensity rainfall, forming debris flows of water-laden rock and soil material that have inundated the transportation corridor (Figure 9). The debris fans that have accumulated consist of silty and sandy gravel, with some cobbles and boulders, some deposited in layers several metres in thickness.

4. IMPLICATIONS FOR THE KAIKŌURA COAST

Overseas experience has shown that large earthquakes not only trigger extensive landsliding but also increase the number and intensity of subsequent rainfall-induced landslides (Lin et al., 2006; Zhang et al., 2014). Experience from the 1999 Chi-Chi earthquake in Taiwan and the 2008 Wenchuan earthquake in China shows that the critical rainfall thresholds for triggering landslides and debris flows decrease significantly following large earthquakes, commonly reducing to between 25% and 75% of the pre-earthquake threshold.

The heightened landslide initiation probability persists for over a period of several years, with a gradual return to the pre-earthquake conditions (e.g. Chen et al., 2013, Hovius et al., 2011). In mountainous terrain, it can take significantly longer to return to pre-earthquake levels of landsliding (Li et al., 2016; Tang et al., 2016). Analysis of the distribution and characteristics of coseismic landslides triggered by the 1929 Murchison and 1968 Inangahua earthquakes identified hillslopes that did not fail in the 1929 earthquake but subsequently failed in the 1968 event as a result of the damage previously caused (Parker et al., 2015).

An example of this overall decrease in stability is 'Slip 29A' in the Oaro-Peketa section south of Kaikōura (Figure 10) which failed as a result of heavy rainfall during Cyclones Debbie and Cook in April 2017 from an area with no historical record of instability, and that had not failed in the Kaikōura earthquake.

In addition to the increased probability of apparent 'first time' slope failures like Slip 29A, many of the debris flow sites that have developed over the time interval since the Kaikōura earthquake have originated from ground damaged or debris accumulated from evacuative failures high on the slopes.

The inference drawn from all available evidence was that, consistent with documentation from overseas and from the Murchison and Inangahua earthquakes, the slopes along the Kaikōura Coast are now much more sensitive to rainfall and earthquake effects, and can be expected to remain that way for several decades.

Figure 10: Slip 29A, following failure in April 2017.

5. TRANSPORT CORRIDOR RECOVERY IN AN EARTHQUAKE DAMAGED ENVIRONMENT

5.1 Project Requirements

The Asset Owner outcome requirements for the NCTIR program were to achieve an acceptable level of safety and service for day to day operations. For KiwiRail this meant operational controls remaining in place after 30 November 2018 were compatible with business-as-usual operations, and for the New Zealand Transport Agency (NZTA) this meant improving reliability of the post-earthquake damaged network.

Two Key Outcomes were required from the NCTIR Program:

First Outcome – Risk to Life

Target risk levels were determined separately by both Asset Owners, with ALARP principles applying for risks lower than these levels.

The risk estimation for the State Highway was completed using the New South Wales Roads and Maritime Services (NSW RMS) Guide to Slope Risk Analysis (Version 4, 2014). This tool permits the rapid and robust estimate of the level of risk posed to road users from landsliding (including rockfall and mass instability) affecting the transportation corridor.

Slope risk adjacent to the rail corridor was assessed using the KiwiRail developed points based slope hazard rating (SHR) system (Justice, 2012) as a proxy. Sites with a higher points rating typically pose a higher risk to the network than slopes with low ratings.

Second Outcome - Level of Service

KiwiRail and NZTA specified acceptable duration outages compared to return period, expressed as an Annual Recurrence Interval (ARI), for outages ranging from a few hours for frequent events, to over 120 days outage for events in excess of 100-year annual recurrence interval.

5.2 Outcome 1 - Risk to Life

5.2.1 Management Framework

The Risk Management approached adopted by the NCTIR geotechnical team was based around the underlying principles of Landslide Risk Management approach developed by Australian Geomechanics Society (AGS, 2007). The approach considered the following key areas:

1. Hazard Analysis (hazard and consequence analysis);
2. Risk Estimation;
3. Risk Evaluation and
4. Risk Mitigation.

5.2.2 Hazard Analysis

Engineering Geological assessment, in conjunction with a GNS study of slope vulnerability and run-out distances (Massey et al, 2017), was used to assess the potential for each of the identified failure mechanisms to affect the entire transport corridor. This highlighted the possible consequences of future rainstorm and seismic triggered instability.

Geomorphological information including field observations, LiDAR hillshade assessment and surface difference modelling was used to evaluate the relative likelihood of a significant failure at each site. Evidence of slope movement included observation of tension cracks and scarps in the upper part of the slope, in some cases measured (by monitoring) in response to rainfall or earthquake aftershocks.

Fault traces and ridge rents that experienced movement as a result of the earthquake were observed throughout the project corridor. Movement of ridge rents as a result of earthquake shaking is not necessarily associated with landslide movement. However, where vertical offset has been observed on a pre-existing scarp and the sense of movement is downslope it has been interpreted as landslide deformation. By way of example, the Half Moon Bay landslide complex, Kaikōura North, exhibits a range of faults, ridge rents, tension cracks and landslide scarps (Figures 11 and 12).

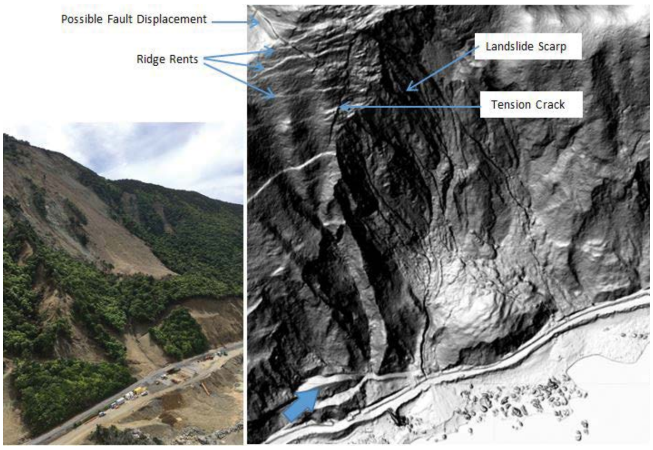


Figure 11: Oblique Aerial View and LiDAR grey scale Image of Half Moon Bay Landslide Complex,
Kaikōura North (photograph location and direction indicated by blue arrow)

Figure 12: Example of GIS- based field mapping data (December 2016 aerial imagery) of area shown in Figure 11.

Outputs from landslide characterisation provided information on the possible size and extent of future failures and informed the spatial distribution of landsliding across the transportation corridor. This data was incorporated into the risk and resilience analysis.

5.2.3 Risk Estimation and Evaluation

Analysis of Frequency of Slope Failure

Rainfall triggering thresholds for landslides have been developed by Glade et al. (2000) for the Wellington region. Given the similar geological setting, these thresholds were considered generally applicable to the greywacke terrain in the Kaikōura area. The Glade et al. (2000) study used antecedent rainfall (based on the rainfall over the preceding 10 days) as an index of soil moisture compared to daily rainfall to identify threshold conditions for landslide triggering. The probability that triggering rainfall will occur can then be determined from the frequency/magnitude distribution of the local rainfall record compared to slope instability records.

Records of rainfall and slope movement have been kept since completion of the immediate post-disaster clearance of slips on the north and south coasts. Data has been systematically recorded since March 2017. Comparison of the daily rainfall, the antecedent daily rainfall index and the estimated size of failures is shown in the Figure 13 and typically indicates:

- Active landslides are prone to further debris movement in very small rain events, proportional to the antecedent rainfall condition and the amount of rainfall on the day of slope failure.
- Antecedent rainfall has a strong influence on the amount of rain required to trigger further slope movement.
- Few failures are initiated under heavy rainfall with low antecedent rainfall.
- Relatively large scale failures are commonly initiated following the cessation of rain under high antecedent rainfall conditions.
- Landslide debris fans are being mobilised into channelised debris flows in modest rain events and cause on-going problems to the transport corridor.
- Slopes that did not show obvious signs of failure or deformation in the earthquake have the potential to develop into large slope failures (e.g. Slip 29A).

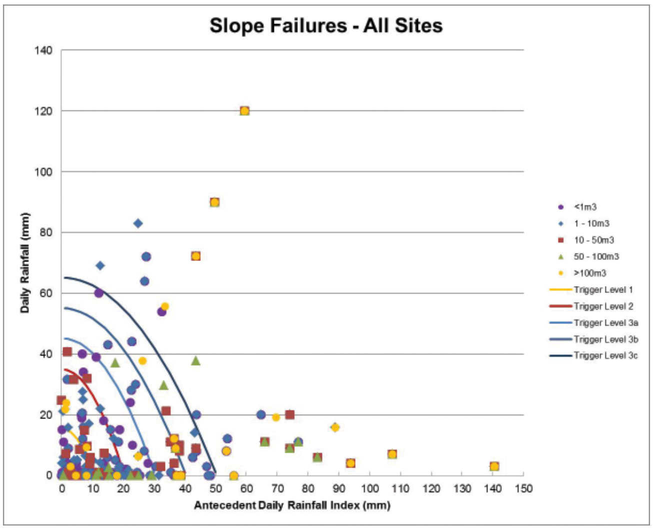


Figure 13: Observed Rainfall Induced Slope Failures, March 2017 onwards

Consequence Analysis

Where slope failure interacts with an 'element at risk' (i.e. road or rail user) the consequence of that interaction was identified in accordance with the requirements of the NSW RMS system or KiwiRail's slope rating system. The assessment of the consequence of failure at any one site typically involved assessment of several variables, including:

- Boulder size (derived from landslide characterization reporting)
- Run-out distribution (estimated from site observations, rockfall modelling calibrated from site observations or derived from Fahrboeschung angle assessment (Massey et al, 2017),
- temporal probability (typical average daily traffic) and
- vulnerability (for road users).

The consequence of the interaction of the hazard and the 'element at risk' typically depends upon the magnitude of the hazard (e.g. runout reaches rail and/or road) and the relative level of protection afforded to the 'user' by the immediate environment (for example, what type of vehicle is the user travelling in).

5.2.4 Risk Mitigation

At any one site, risk mitigation involved implementation of one or several solutions. In general, mitigation options involved engineered stabilisation and protection works. However, non-engineered works, such as operational controls in response to earthquakes or rainfall, or remote monitoring were considered as part of the suite of mitigations.

Figure 14: Rockfall Mesh and Anchor installation, Ohau Point

Figure 15: Rockfall Attenuator, Parititahi Tunnel North Portals, Kaikōura South

Figure 16: Hybrid gabion basket wall and rockfall fence, Kaikōura South

Figure 17: Irongate Bridge and SH1 realignment at Slip P2, Kaikōura North

Stabilisation and Protection Works

Engineered risk mitigation solutions can be considered as either reducing the likelihood (active mitigations) or consequence (passive) of failure. Active solutions typically involved, but were not limited to:

- Scaling, boulder removal and sluicing;
- Bulk earthworks;
- Anchored rockfall netting (Figure 14).

Passive structures included, but were not limited to:

- Catch-ditch earthworks
- Rockfall and Debris Catch Fences/Attenuators (Figure 15)
- Shallow Landslide Barriers
- Earth Bunds and Hybrid bunds/Fences (Figure 16)
- Transport Corridor realignment (Figure 17)
- Rock and Debris Avalanche Shelters
- Remote monitoring

Operational Controls

While the majority of slope risk is likely to be associated with the areas where failure occurred during the earthquake, the experience from SR29A suggested that there was some likelihood of first-time failure occurring in previously unidentified areas was also possible as well as reactivation of pre-historic landslides. With this in mind, rainfall Trigger Action Response Plans (TARPs) were developed for road and rail, with probabilistic thresholds established based on the rainfall-slope failure relationship that was determined for the Kaikōura area. The probabilistic threshold curves developed reflect various probabilities of exceedance in relation to slope failure in accordance with Glade et al (2000). Based on rainfall, the TARP enables assessment of slope risks for running freight trains, SH1 traffic and for construction activities, hence predicting when closures may be needed (Paterson, 2018).

To enable the Asset Owners to sufficiently plan alternatives and notify customers if a route closure is likely, the TARP model was developed into a forecasting tool. Based on rainfall records and rainfall forecast over the future four days, it has been possible to assess the likelihood of slope failure in advance. The levels shown on Figure 13 reflect trigger points for differing actions as shown by Figure 18.

In addition to the TARP, a series of remote monitoring installations were placed in a number of key locations to provide an early warning of slope instability affecting (primarily) the rail line. These fences typically comprised a series of string potentiometers connected to tripwires supplemented with tiltmeters installed on fence posts. On impact on any fence, an alarm was automatically generated to enable train services to be halted until an on-ground assessment could be completed and any failed material removed, if necessary.

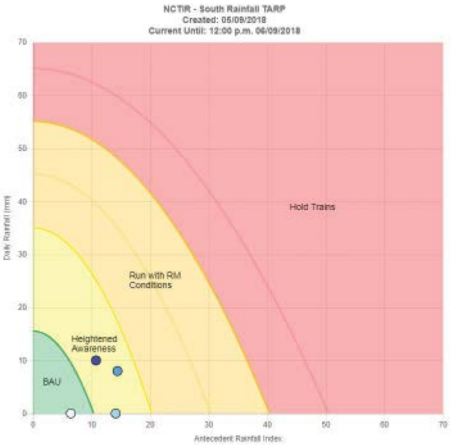


Figure 18: Example of Rainfall TARP for rail. The dark blue dot represents the current day. The lighter blue dots represent forecast conditions over the next three days (in order of increasing lightness).

5.3 Outcome 2 - Level of Service

For the NCTIR project, "Resilience" was defined as follows (Mason et al, 2018):

Resilience = Robustness + Redundancy + Response

The future slope performance assessment was based on the NCTIR geotechnical team's assessment of the landslide areas, supplemented by geomorphological assessment of the slopes which have not failed. The assessment considered the slope failure history and the types of anticipated slope failures, observations of the time taken to clear debris, and the requirements or logistics for repair and reinstatement. The type and extent of any planned or completed engineered works was also taken into account.

Initiatives that strengthen each of these aspects will therefore contribute to overall improvement of the resilience.

Robustness

Options to improve the robustness of the corridor included realigning the road and/or rail, engineered works to reduce the potential for slope failure, and engineered works to reduce the potential for inundation of the corridor. Specific measures adopted included:

- Seaward realignment of the rail and road corridors away from the hazardous hillslopes;
- Construction of new bridges to allow debris flows to pass beneath the transport corridor;
- Slope stabilisation with rock bolts and mesh;
- Up-slope earthworks and slope re-profiling;
- Installation of engineered rock fall fences and landslide barriers;
- Installation of drainage measures to relieve groundwater pressures or control surface water runoff.

Redundancy

There are few options to improve redundancy for the rail as the MNL is the sole rail route between Canterbury and Marlborough. For the state highway, redundancy is improved by:

- Upgrading the capability of the alternative routes (principally the Lewis Pass route through state highways 63, 65 and 7);
- Identifying areas where alternative routes can be quickly established to avoid potentially damaged assets (for example, alternative bypass routes for vulnerable bridges).

Response

The NCTIR Alliance has provided KiwiRail and the NZ Transport Agency with the ability to react quickly in the event of a hazard event. After the completion of the NCTIR project period, the following measures will enhance organisational response for management and maintenance of the network:

Installation of tripwire fences with remote monitoring sensors to notify when rock falls or slips occur;

Monitoring of unstable slopes with GPS sensors and ground surveys to provide a forewarning of slope failure;

Implementation of streamlined response plans for hazard events - there are numerous slope assets that have been built under the NCTIR works, many of which protect both road and rail. Recognising this, KiwiRail and NZTA are embarking on creating a joint maintenance process to determine how combined issues can be best responded to, such that personnel and plant can be mobilised quickly in the event of failure.

The combination of engineering works to improve the robustness of the transport corridor and an improved response via improved maintenance and management practices means that NZTA's and KiwiRail's level of service are expected to be met for relatively frequent future events, but could be exceeded in larger events.

6. CONCLUSIONS

The November 2016 Kaikōura earthquake caused widespread damage and severe disruption to the road and rail corridor along the Kaikōura Coast. Residual damage in hill slopes that did not fail co- or post-seismically but have been sufficiently weakened to fail in the next large triggering event means that the damage legacy of the earthquake will persist in the landscape for decades.

Recovery of this nationally vital portion of New Zealand's transport infrastructure was consequently focused on (a) reducing life safety risk and (b) improving the resilience of the network.

For slope risk reduction, recovery works have included a suite of engineered risk mitigation solutions that can be considered as either reducing the likelihood (active mitigations) or consequences (passive mitigations) of failure. These physical works are supplemented by non-engineered measures, including a response plan to elevated rainfall, which allows both Asset Owners to proactively manage risks to the transport corridor.

7. REFERENCES

Australian Geomechanics Society (25007) Landslide Risk Management Guidelines. Australian Geomechanics, Volume 42 No 1 March 2007

Bell, D.H. (1976). High intensity rainstorms and geological hazards: Cyclone Alison, March 1975, Kaikōura, New Zealand. Bulletin of the International Association of Engineering Geology, 14: 189-200.

Chen, J.C., Jan, C.D., Huang, W.S. (2013). Characteristics of rainfall triggering of debris flows in the Chenyulan watershed, Taiwan. Natural Hazards and Earth System Sciences 13: 1015-1023.

Hungr, O., Leroueil, S., and Picarelli, L. (2013). The Varnes classification of landslide types, an update. Landslides; April 2014 Vol 11, Issue 2 pp 167-194.

Glade, T., Crozier, M., Smith, P. (2000). Applying probability determination to refine landslide-triggering rainfall thresholds using an empirical "Antecedent Daily Rainfall Model". *Pure and Applied Geophysics* 157: 1059-1079.

Hovius, N., Meunier, P., Lin, C.-W., Chen, H., Chen, Y.-G., Dadson, S., Horng, M.-J., Lines, M. (2011). Prolonged seismically induced erosion and the mass balance of a large earthquake. Earth and Planetary Science Letters 304: 347-355

Justice, R (2012). Development of a Slope Risk Rating System for New Zealand Rail. 11th Geomech Conf, Melbourne, July 2012.

Li, L., Yao, X., Zhang, Y., Iqbal, J., Chen, J., Zhou, N. (2016). Surface recovery of landslides triggered by 2008 Ms8.0 Wenchuan earthquake (China): a case study in a typical mountainous watershed. Landslides 13: 787-794.

Lin, C.-W., Liu, S.-H., Lee, S.-Y., Liu, C.-C. (2006). Impacts of the Chi-Chi earthquake on subsequent rainfall-induced landslides in central Taiwan. Engineering Geology 86: 87–101

Lin, M.L. et al (2008). The effects of earthquake on landslides – a case study of the Chi-Chi earthquake, 1999. In *Landslides and Engineered Slopes* (Chen et al – eds). Taylor & Francis, London. p193-201

Mason, D., Justice, R., McMorran, T. (2018). Assessment of the resilience of the North Canterbury transport network following the 2016 Kaikōura earthquake. Proc. 2018 IPWEA NZ Conference, Rotorua, 20-22 June 2018. Paper No. 507a, 8 p

Massey C.I, Taig T.T, Lukovic B, Rosser B, Ries W. (2017) The North Canterbury Infrastructure Recovery Alliance: Pilot study for assessing landslide hazards along the road and rail corridors GNS Science Consultancy Report 2017/185 DRAFT 5 October 2017

Parker, R.N., Hancox, G.T., Petley, D.N., Massey, C.I., Densmore, A.L., Rosser, N.J. (2015). Spatial distributions of earthquake-induced landslides and hillslope preconditioning in the northwest South Island, New Zealand. Earth Surface Dynamics 3: 501-525.

Paterson Z. (2018). Development of an Early Warning System for Rainfall Induced Slope Failure Affecting Railway in a Post-Earthquake Setting. 12th YGPC, Hobart (in prep)

Tang, C., Van Westen, C.J., Tanyas, H., Jetten, V.G. (2016). Analysing post-earthquake landslide activity using multi-temporal landslide inventories near the epicentral area of the 2008 Wenchuan earthquake. Natural Hazards and Earth System Sciences 16: 2641-2655

Zhang, S., Zhang, L.M., Glade, T. (2014). Characteristics of earthquake- and rain-induced landslides near the epicenter of Wenchuan earthquake. Engineering Geology 175: 58–73.

Ohau Point Ring Net and TECCO Mesh Drape

This paper was previously submitted to NZSEE 2018 and published as a poster

ABSTRACT

Following the M7.8 Kaikōura Earthquake in New Zealand, Hiway Geostabilization (HGS) was awarded the North Face Ohau Point Scaling and High Tensile Mesh Installation contract. The original design to mitigate rockfall hazards included a pinned high-tensile steel wire mesh. During the project's design phase, the ground conditions encountered were some of the worst in the region as the slope was highly affected by ground shaking.

Subsequently the original programme was delayed significantly by the condition of the slope. In order to accelerate access for road construction crews below Ohau Point, a value-engineered ring net drape was proposed by HGS to cover the bluffs' highly unstable upper section. The design, using WASHDOT and FHWA guidelines, was completed and approved in less than a week and required coverage of approximately 5,500 m2 on 40° to 80° slopes. Key components of the design were the use of a 6-on-1 ring net to optimise coverage and minimise choking during helicopter installation, and the consideration of the frictional resistance (interface friction) provided by the ring net on the shallower upper slopes. The project exhausted the global supply of 6-on-1 ring nets after materials were airfreighted from suppliers in Western Europe and North America.

Once installation was completed in early September 2017, manual scaling of the 12,000 m² lower face was initiated using roped access rockfall technicians. The seawall construction team was subsequently provided staged access to the beach platform below Ohau Point; the first people to access this area in more than ten months.

Mat Avery

Mat Avery is a Senior Engineering Geologist and Project Manager for Geovert NZ. As well as business development work he is often heavily involved in helping clients solve geotechnical problems especially in the fields of rock fall and slope stabilisation. With fourteen years' experience, six of those as a consulting Eng Geo, Mat has extensive experience in the planning and implementation of geotechnical construction projects in a variety of geological conditions.

Graeme Quickfall

Graeme has 30 years of experience working in the United Kingdom, Canada, Australia, throughout the Pacific Islands, and New Zealand, where he is General Manager Hiway Geotechnical. Graeme has wide ranging hands-on experience across operations, project management, business development, and technical engineering fields. He pioneered the use of deep soil mixing and related ground improvement technologies in New Zealand.

1. INTRODUCTION

Shortly after midnight on Monday 14th November, a large magnitude 7.8 quake struck the small settlement of Waiau in North Canterbury, north east of the South Island. The quake was the largest in New Zealand since the magnitude 7.8 Dusky Sound earthquake in 2009 (GNS 2016). While shaking was widespread with over 15,000 recorded 'felt reports', the worst shaking occurred about 50 seconds after the quake rupturing started. The energy of the quake progressed north over several minutes with surface rupture recorded on a total of 21 faults. The length of all the fault ruptures combined is close to 100km (GNS 2016).

The degree of ground shaking was high, recorded as Modified Mercalli Intensity Scale (MMI) Level 8 – Severe. This level of shaking causes considerable damage in ordinary buildings with partial collapse including fall of chimneys, factory stacks, columns, monuments, walls, etc.

Figure 1: Ohau Point, December 2016 (photo courtesy Opus)

At a human level people experience difficulty standing and furniture and appliances shift. Geologically the damage depends on the geological setting. In Kaikōura and the surrounding area, especially the coastal highway State Highway 1, north and south of the town, many of the slopes range over steepened weathered, fractured greywacke. During the quake the shaking caused significant and widespread damage with a total of 26 major slips (and many smaller slips); closing both State Highway 1 and the Main North Rail Line between Picton and Christchurch.

The earthquake most severely impacted the coastal road and rail which hugs the coastline along a stretch of some 20 km north and south of the coastal tourist town on Kaikōura. These links serve as the South Island's main rail and state highway route. The transport corridor follows the coastal alignment and is constructed on a very narrow bench at sea level. The coastal mountain range rises from sea level up to some 500 metres of elevation. The earthquake triggered a significant series of rockfall landslides with some 40 major slides reportedly dislodging some 750,000 m^3 of rockslide; which buried many sections of the transport corridor. The town of Kaikōura was completely isolated until access south was re-established via an alternative inland route. During November and December 2016, the initial recovery effort involved helicopter support to service Kaikōura and this effort was hugely supported by the coincidental presence of an international Navy operation which provided a airlift and marine support from USA, Canada, Australia and New Zealand.

The highway and rail north were closed for 12 months and a reconstruction team, North Canterbury Transport Infrastructure Recovery (NCTIR), was quickly established as a government led alliance, tasked with the rebuild. A deadline of 12 months was established to have the northern road and rail corridor opened by Christmas 2017. The monumental task of the design-build recovery at an initial value of $ 1.5 billion and a workforce of up to 1500 personnel began in earnest in January 2017.

2. OHAU POINT

Ohau Point is the largest and most significant and challenging rockslide triggered by the Kaikōura event. Strategically this rockslide feature created a pinch point for the construction of a temporary haul route. This site quickly became the highest priority repair site and critical path to enabling the initial haul route to be established. The site is some 300 metres wide at the base and follows a triangular shape with the apex some 300 metre elevation above the roadway. As stated in the project specification (provided by NCTIR), the north face of Ohau Point "comprises closely jointed sandstone and mudstone of the Pahau Terrane deposits, consisting of bedded sandstone and mudstone. Initial helicopter sluicing and scaling of the face revealed surficial soils and boulders below the head scarp with a highly fractured and open jointed rock mass beneath".

Due to the significant damage caused by the November 2016 Kaikōura Earthquake, the slope had been extensively sluiced by helicopter; with additional scaling by roped access crews carried out to approximately the mid-point of the upper section.

3. REMEDIAL SOLUTIONS

3.1 Immediate Response

Following the earthquake, access beneath Ohau Point was not possible for safety reasons but also due to the sheer volume of collapsed material covering the road and beach platform. The average annual daily traffic (AADT) volume pre earthquake was ± 7000 vehicles.

Having been significantly affected by landslide instability during the November 2016 Kaikōura Earthquake, Ohau Point had been extensively sluiced by helicopter in the weeks following the quake. Scaling by roped access crews along the crest and on the upper slopes of the face was also undertaken as safe access allowed. Even following this treatment, the slope continued to present a significant source of rockfall onto the lower slopes and highway bench below.

3.2 Permanent Remediation

The permanent rockfall source treatment on Ohau Point included high tensile steel wire TECCO mesh from Geobrugg; secured with rock anchors in an offset diamond pattern. This allowed for safe access to the lower reaches of the slope for further remedial work, and debris clearance on the highway bench below. Prior to the mesh and anchor installation, further manual scaling was required to prepare the slopes and make them safe for access. The total area of treatment on the upper slope was in the order of 5,600 m². The lower face, requiring manual scaling and localised rock bolting, had an area of around 12,500 m².

The original contract was awarded to HGS in March 2017. The design required pattern bolting and TECCO mesh installation over the (assumed, approximate) 3,400m² upper slope. As part of the contract negotiation, HGS proposed as an alternative anchor bar our proprietary low carbon & low phosphorous steel hollow bar product. This alternative was requested due to high-carbon steel content possessing lower durability in comparison to a lower carbon steel bar. A reduced carbon content also improves the ductility of the steel, which in a high level seismic environment is critical.

The original programme allowed for approximately three months of construction time including manual scaling, pattern bolting, and TECCO mesh installation. Understandably, the client was requiring access to the road and beach platform below in the shortest timeframe possible to enable construction of a new seawall and highway.

3.3 Fast Track Stabilisation

With weather delays, unfavourable winter conditions, and difficult & dangerous ground conditions causing delays in completing the original design work, the client requested HGS provide an alternative methodology/solution to fast track access below the site. In June 2017 HGS, after considering a number of stabilization options, selected a mesh drape system concept. This option provided the fastest means of stabilising the slope to make the slope safe for the installation team and the road reconstruction team working below. Additionally, after evaluating ways to improve productivity, the site was deemed too unstable to add further drilling equipment as the site would become too crowded. HGS then provided a number of options, involving alternative solutions, to accelerate the programme. These included,

1. Drape Mesh (Geobrugg S4-130 SPIDER)
2. Drape Mesh (Geobrugg G65/3 TECCO)
3. Drape Ring Net (6-on-1 with 10" rings)
4. Sacrificial Mesh with Gunite (Geofabrics DT mesh)
5. Gunite (polypropelene fibres)

After careful consideration, including material supply and overall programme, the ring net drape option was accepted by the client as an addition to the existing TECCO mesh drape. The ring net was required to contain potential large volume instabilities and a 6-on-1 ring net chosen as it maintains its width when lifted – critical when installation tis via helicopter as time needed to be kept to a minimum. The TECCO mesh was installed to stop small diameter material releasing and creating a hazard to workers on the slope.

This design came with considerable challenges. Including allowance for overlap and wastage, the global supply of 6-on-

1 ring net was exhausted, with suppliers in North America and Western Europe providing their entire stock (approx. 5600 m^2). The design (by HGS) and review (by NCTIR) process was completed within one week and perimeter anchor installation began immediately following. Due to time constraints, materials were air freighted from Europe and North America. The ring nets' installation began three weeks after the start of the perimeter anchor installation.

4. DESIGN OF RING NET AND MESH DRAPE

It was necessary to make a number of assumptions during the design of the drape. Some of these included

1. Slope will be partially scaled prior to drape installation:
   - Loose unstable material scaled by hand
   - Some of the large semi-detached blocks will be blast removed prior to drape installation. Some may be blast removed post drape installation.
   - Full slope will be considered rough terrain for purpose
2. Drape system is intended to:
   - Control potential block failures and have these failures retained behind the ring net drape for maintenance and clean-up
   - Control spall of rock down to a size of 30-50mm diameter
   - Provide maximum volume block failure (single failure/per panel) estimated at 2m^3
3. Drape system is not intended to:
   a. Achieve 100% rock fragment containment
   b. Contain soil ravelling
   c. Support the slope during or following significant ground shaking
4. Drape system is of the secured type as system is anchored across the base by wire rope with brake elements

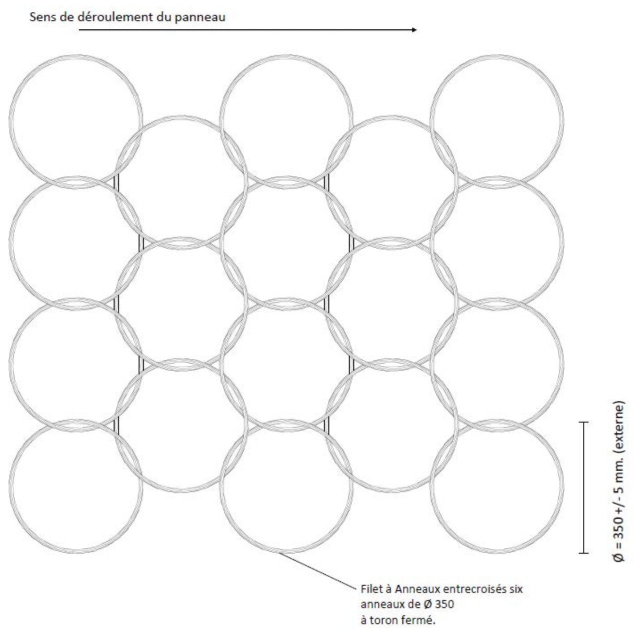


Figure 2: Typical 6 on 1 ring arrangement

Based on these assumptions, and international availability of ring net, the IGOR 6-on-1 ring net was chosen combined with Geobrugg G65/3 TECCO mesh. The ring net was placed on the slope first as this has the higher strength and is more capable of retaining larger blocks should they fail during construction. Additionally, the ring net conforms to the slope better than the high tensile chain link mesh. The TECCO was placed over the ring net to form a permanent solution. The IGOR D300 ring net is a 300/6 (300mm diameter, 6 strands per ring) with 6-on-1 ring assembly (any one ring has six connected rings). Various panel sizes were used based on what was available from the suppliers (30, 40 and 50 m^2 panels). Maximum load bearing capacity of the IGOR net is 340 kN or 30 kN / ring.

The TECCO mesh is made of high-tensile steel wire with an ultimate tensile strength of 1770 N/mm^2. The diamond shaped, three-dimensional mesh exhibits homogeneous characteristics with a load-bearing capacity of 150 kN/m in the primary bearing direction (lengthwise). This product also uses Geobrugg's Ultracoating corrosion protection system providing significantly higher corrosion resistance than standard galvanizing. Mesh panel sizes are 105 m^2.

Consideration was given to installing the TECCO mesh under the ring net. However for two reasons it was decided that the TECCO installed on the outside is most suited:

1. The ring net will conform to the highly irregular slope better than the TECCO
2. The TECCO does not have sufficient strength to retain any potential large volume failures that may occur during the installation process (prior to installing ring net).

4.1 Design

Many factors were considered during the design of the drape. Critical were the components of the system; ring net, mesh, anchor bar etc, but also the slope geology and geometry. The upper slope of Ohau Point is characterised by two different slope angles - the top 35 m of the slope is at a substantially lower angle than the bottom of the slope with a slope angle of ± 35° for the top 40 m (surficial soils and boulders) and ± 65 ° for the lower 40 m (predominantly competent rock). The frictional resistance (interface friction) of this top section would have a significant impact on the anchor design loads. However, time constraints meant the calculation of this inexact science was not possible, but based on calculations from

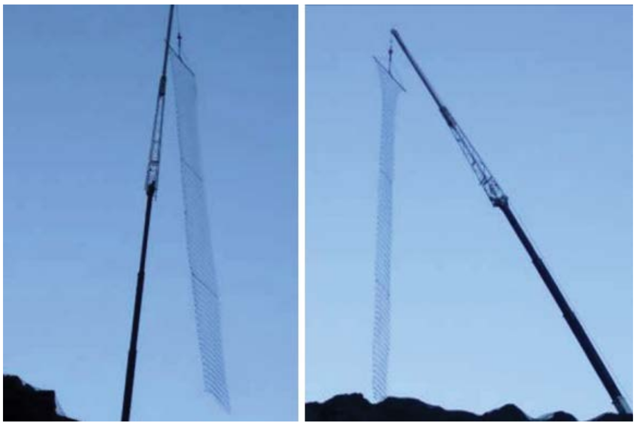


Figure 3: 60n1 net (left) and other net (right). Note contraction (necking of other net during lifting. (photo courtesy FHWA, 2009)

previous projects with similar slope profiles and ring net types, the frictional resistance on the upper slope could realistically mean there would be no load transfer onto the anchors from almost half of the drape system.

As the consequences of a system failure on the slope were very high, it was decided to include the full height of the drape system in the perimeter anchor calculations. The weight of the combined ring net and mesh was calculated, as was the weight of likely debris loads (from failures) and impact loads (minimal). The total load on the mesh was calculated as,

$$P_{TOTAL} = W(_{DRAPE}) + W(_{DEBRIS}) + W(_{IMPACT})$$
$$= 274\text{ kN} + 3{,}100\text{ kN} + 25\text{ kN}$$
$$= \underline{3{,}399\text{ kN}}$$

With an applied factor of safety (FoS) = 1.3 = 4,418 kN

The capacity of the perimeter anchors was calculated at 430 kN based on destructive pull out tests undertaken on site. With a factor of safety of FoS = 2 applied, the

Figure 4: Acceptance testing of the perimeter anchors.

Figure 5: Helicopter installation of the ring net

load requirements on the 36 anchors was less than half the capacity of any individual anchor. Increasing the redundancy of the system was a second row of anchors (taking the total number to approximately 72 each) installed to lift the edge of the drape above ground level and creating an 'open throat' system to catch any rolling boulders from above the system.

5. INSTALLATION METHODOLOGY

The basic construction sequence allowed for the installation of the perimeter anchors and wire rope prior to the installation of the ring net and mesh. The total programme for the installation was delayed due to unfavourable drilling conditions on the site. Two light-weight wagon rigs were in operation working outwards to either side of the crest. To accelerate the overall programme, the second row of anchors was eliminated from the lower portions of the perimeter, these were redundant from a design perspective so this had no detrimental effect on the system's performance. Also to accelerate the programme, the ring net installation began before the completion of the perimeter anchor installation. This increased the resource requirements on the project as two drill rigs were in operation as well as a full grouting crew plus the drape installation crew.

A critical part of the drape's installation was the ground crew. Logical and organised preparation of the ring net and mesh panels was essential to a seamless installation

REDI-ROCK®

Redi-Rock® is a complete engineered retaining wall solution that uses giant concrete blocks that stack up like Lego to create strong, great-looking wall solutions for Gravity Walls and MSE Reinforced Walls

APPLICATIONS

- Gabion basket alternative
- Slip repairs
- Roads and bridges
- Retaining walls for subdivisons, sea walls, waterways and much, much more

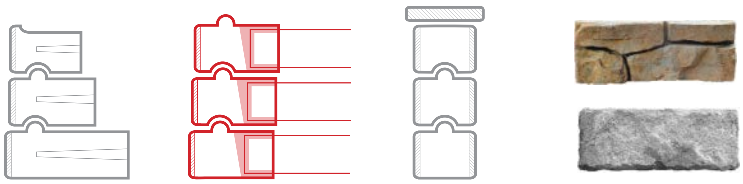
NZTA ROAD SLIPS

Opus and Fulton Hogan selected Redi-Rock Walls as the optimum solution for a series of road slips on State Highway 15, Northland.

The project was completed quickly and safely. Everyone involved is happy with the outcome.

Your Complete Wall Solution

Each block in the Redi-Rock sustem is available in any texture and can coordinate in the same wall, giving you the flexibility you need.



Gravity Walls MSE Walls Freestanding Walls

Design Software

Redi-Rock® retaining wall system is designed for engineers, by engineers.

Designing critical wall structures is important work - and you need the best resources available to help you do your job.

All the design tools and software can be downloaded at **redi-rock.com**

***Redi-Rock is NZTA Approved.** Talk to us today and explore Redi-Rock, the clever retaining system - and it looks great too.*

DURACRETE
PRECAST TO LAST

To learn more visit **www.redi-rock.com**
Email: sales@duracrete.co.nz
Phone: David 021 951 057

process. Different size panels were required at different times based on the slope's profile of the slope. With a turn-around time of approximately 8 minutes between hoists, clear communication between the helicopter pilot, installation crew on the slope, and the ground crew was critical. All panels were pre-shackled on the ground to accelerate their installation on the slope. As a measure of the scale of the operation, as well as the global supply of ring net being utilized, the shackles consumed on the project exhausted the Australasian supply three times over. The project required weekly supplies of air freighted product delivered from Australia.

5.1 Perimeter rock bolt and wire rope installation

The perimeter anchors were installed up to 6.0m deep with a socket length of 2.0m required into competent rock. Grout take on the holes was highly variable, with between theoretical volume up to in excess of 3000 litres per hole. Light-weight wagon mounted drill masts were used, running on a combination of pneumatic and hydraulic drives. All power plants for the rigs were located at the top of the bluff in a secure working area with two 375 cfm compressors providing air flush. With access to the site restricted, all materials, consumables, and fuels were delivered daily or as required by helicopter.

A single 20 mm diameter steel wire perimeter rope was connected to each anchor head around the perimeter of the system. This was used to connect the ring net and mesh to the top support anchors. Once all anchors were installed and the grout sufficiently cured, the wire rope was tensioned. The maximum length between end terminations was 30 m in order to allow sufficient tension to be applied to the rope without introducing excessive friction into the system. A secondary 16 mm diameter wire rope to provide load share was installed in short lengths tying the front row of anchors to the back row.

5.2 Ring net and mesh installation

The various ring net panels were 10m long by 3m, 4m and 5m wide. Panels were pre-stitched on the ground with two panels connected together length ways (to create one 20m long super panel). Originally three panels were combined into one super panel. However with the variable wind conditions on site, it was deemed more efficient (and safer) to reduce each load to two panels. The 1.5 ton connecting shackles were installed along one side and the bottom edge of the super panels to accelerate the installation process on the slope. A specially developed spreader bar was fabricated which allowed the pilot to release the panel remotely once placed on the slope to allow for safer panel installation for the rope access crew working on the slope. The panels and spreader bar were attached to the helicopter via a 30 m long line. Using highly qualified and experienced pilots, the panels were flown onto the slope at every available weather window. A total of 23 days were required to fully install the drape system.

Once the drape was installed, three bottom ropes were connected across the base of the drape with rated braking elements installed at the northern end. The bottom ropes were then tensioned to ensure no material could escape the base of the drape should another failure occur.

6. PROJECT MILESTONE

At the completion of the drape's installation, the road crew below Ohau Point pushed a haul road across the raised beach platform. Then for the first time since the earthquake, road crews working to re-open State Highway 1 had access all along the coastal route north of Kaikōura.

"By August 2017, a construction access platform was cut around the hillside, enabling machinery to make quicker progress to clear the slip material. For the first time since the earthquake, work crews from both sides of the landmark could work together."

REFERENCES

GNS Science, 2016. M7.8 Kaikōura quake the biggest since the Dusky Sound jolt in 2009 - 15/11/2016, *GNS media release*, 14/11/2016 7:49am

FHWA, 2009. Use of Ring Nets for Slope Protection for Rockfall: End-of-Construction Report. *Experimental Feature Study, Geotechnical Report C-7540, SR 28, Rock Island – Rock Slope Nettings.*

NCTIR, 2017. SH1- Ohau Point, the last slip in the red zone. September-November 2017 - milestones achieved. *NCTIR Newsletter, online release.*

ACKNOWLEDGEMENT

This paper was previously submitted to NZSEE 2018 and published as a poster.

Figure 6: Completed drape showing perimeter anchor connection detail

Seismic Design of Geotechnical Structures for NCTIR

ABSTRACT: This paper outlines key elements and the main findings for seismic design of geotechnical structures, i.e. earth retaining systems as well as natural and man-made slopes, and provides an unabridged roadmap for a streamlined design following the principles of value engineering and performance based design. Although emphasis is placed on the work that was carried out by North Canterbury Transport Infrastructure Recovery (NCTIR) on State Highway 1 following the 14 November 2016 Mw 7.8 Kaikōura Earthquake, the derived conclusions are valid for any other NZ Transport Agency (NZTA) related geo-mechanical Performance Based Design.

NZTA Bridge Manual Third Edition (NZTA BM3), Amendment 2 (2016), AASHTO LRFD Bridge Design Specification (2016), Eurocode 8 Part 5 (2004), FHWA-NHI-11-032 (2011), and MBIE/NZGS Module 6 (2017) are compared and discussed.

Keywords: Pseudo-static, Retaining Structures, Embankment, Seismic Design, Kaikōura earthquake, Performance Based Design

1. INTRODUCTION

During the 14 November 2016 Mw 7.8 Kaikōura Earthquake, slope and embankment instability was observed at several locations along Northern State Highway 1 of the South Island of New Zealand. Extensive geological and geotechnical investigations were conducted on landslide-affected areas. Parametric slope stability analyses were carried out for the damaged embankments (considering the actual geometry at the time of the instability and that of the new construction) with the objective of realignment and resilience assessment.

NZTA BM3 (2004, 2016), with an emphasis on post-earthquake performance, was the basis of all the aspects of realignment design. NZTA BM3 manual allows for any departures from specified limitations whenever they cannot be practical or are uneconomical to satisfy as long as they are suitably established, internationally recognized and widely used.

According to NZTA BM3, soil structures are categorized into two main groups, namely slopes and retaining structures, with their own specific seismic performance criteria that relate to their level of importance, height, association with bridges and protective effect to adjacent properties. The Author believes that due to the many adjustments and departures applied frequently and by engineers with different levels of expertise, structures that belong to the same class are often treated differently. This creates uncertainties on how to properly follow the regulations.

In this paper an attempt is made to fill the gap in the knowledge and minimize uncertainties in NZTA BM3 arising from high seismic demand and/or cost consideration using technically detailed bases that are widely and internationally acceptable for geotechnical earthquake engineering and Performance Based Design.

2. THEORETICAL BACKGROUND

Earthquake-induced ground accelerations can result in significant inertial forces in slopes, embankments, and retaining structures and these forces may lead to instability (i.e. failure) or permanent deformation. Current practice for the analysis of the performance of slopes, embankments, and retaining structures during earthquake loading is to use any or a combination of the related methods:

- Limit Equilibrium using a pseudo-static representation of the seismic forces;
- Displacement-Based Analysis using the Newmark sliding block concept for preliminary design and more rigorous, stress-strain, numerical modelling methods

Ioannis Antonopoulos

Ioannis is a Chartered Geotechnical Engineer with a focus on large infrastructure and development projects. He has extended experience in large projects and working within interdisciplinary teams in both design and construction including geotechnical earthquake engineering design, water reservoirs, road, surface & deep foundations, cut & cover, tunnelling, slope stability, hydrogeology and water resource management.

for advanced and detail design;

The selection between the two approaches would normally be made on the basis of the complexity of the slope geometry and soil conditions within the slope, the level of ground shaking, and the performance based issues for earthworks design.

2.1. Limit Equilibrium Pseudo-static Stability Analysis

In this approach induced seismic loads are modelled as a static horizontal force in conventional limit equilibrium analysis to either verify that the capacity (C) is greater than the demand (D) or to evaluate the capacity/demand (C/D) ratio (or factor of safety, FoS) for comparison to an acceptable value. The seismic load is defined by means of seismic coefficients k_h and k_v (horizontal and vertical component of acceleration, respectively), which are determined on the basis of the peak ground acceleration (α_{max} or PGA), the earthquake magnitude (Mw), the geometry of the soil mass being loaded, and a performance criterion (i.e. the allowable displacement). This type of analysis is generally referred to as a pseudo-static stability analysis.

2.1.1 Selection of seismic coefficient

The results of pseudo-static analyses are critically dependent on the value of the seismic coefficient, kh. Selection of an appropriate pseudo-static coefficient is the most important, and most difficult, aspect of a pseudo-static stability analysis. The seismic coefficient controls the pseudo-static force on the failure mass, so its value should be related to some measure of the amplitude of the inertial force induced in the potentially unstable material.

If the slope was rigid, the inertial force induced on a potential slide would be equal to the product of the actual horizontal acceleration and the mass of the unstable material. This inertial force would reach its maximum value when the horizontal acceleration reached its maximum value.

The seismic coefficient is typically assumed to be some function of the site-specific Horizontal Peak Ground Acceleration (α_{max}). Seed and Martin (1966) and Dakoulas and Gazetas (1986), using shear beam models, showed that the value of kh for earth dams depends on the size of the failure mass.

In recognition of the fact that actual slopes and many retaining structures are not a rigid body and that the peak acceleration exists for only a short time, the pseudo-static coefficients used in practice generally correspond to acceleration values well below α_{max}.

The seismic coefficient can range from significantly less than 50% of the α_{max} to the full α_{max}, depending on the slope/retaining structure height, the magnitude of the earthquake, performance requirements, and the designer's views. The α_{max} can be height-adjusted. This adjustment depends upon the seismic environment, and increases with slope/retaining structure height.

The design seismic coefficient and associated minimum required C/D ratio are selected such that behaviour of the slope or retaining structure, in terms of permanent deformation, is within a range considered acceptable. A C/D ratio of less than 1.0 when using the height-adjusted α_{max} as the seismic coefficient implies some permanent movement of the slope or retaining structure.

2.2.2 Maximum seismic coefficient

Madabhushi et al. (2009) explain that the maximum acceleration, α_{lim}, which can be transmitted through a soil stratum in a seismic event is limited by the shear strength of the soil. This limiting acceleration can be estimated using the below relationships.

- For dry granular soil (cohesionless) the peak limiting acceleration is given by

$$\alpha_{lim} = \tan\phi$$

where φ' is the effective friction angle

- For fine grained soil (cohesive) the equivalent relationship is given by

$$\alpha_{lim} = \frac{S_u}{\gamma \times h}$$

where s_u is the undrained shear strength at depth *h*, and γ is the average bulk unit weight of the soil mass.

Application of these relationships indicates limiting horizontal accelerations of between 0.5g and 0.8g for dry granular soils, and 0.5g for depths approximately greater than 5m for fine grained soils. Consequently, the horizontal coefficient of acceleration (k_h=0.5α_{max} for slopes or k_h=0.5α_{max} or 0.67α_{max} for earth retaining structures) should not be selected to be greater than 0.4g.

When a design α_{max} greater than 0.8g is required then soil mass failures are expected. To complete the design and to assess the site response and performance, time history analyses, using finite element or finite difference based methods, or the advanced method discussed in the NZTA BM3 Section 5.4.2 should be employed.

2.2. Displacement-Based Seismic Stability Analysis

In contrast to the limit equilibrium approach, the displacement-based approach involves the explicit calculation of cumulative seismic deformation. The potential failure mass is treated as either a rigid body or deformable body, depending on whether a simplified Newmark sliding block approach or more advanced numerical modelling is used.

The purpose of the Newmark (1965) method is to estimate the slope deformation for those cases where the pseudo-static FoS is less than 1.0 (i.e. the failure condition). The Newmark (1965) method assumes that the slope will deform only during those portions of the earthquake when the out-of-slope earthquake forces cause the pseudo-static FoS to drop below 1.0. When this occurs, the slope will no longer be stable, and it will be accelerated downslope. The longer that the slope is subjected to a pseudo-static FoS below 1.0, the greater the slope deformation. On the other hand, if the pseudo-static FoS drops below 1.0 for a mere fraction of a second, then the slope deformation will be limited.

A limitation of the Newmark (1965) method is that it may prove unreliable for those slopes that do not tend to deform as a single block. An example is a slope composed of dry and loose granular soil (i.e. sands and gravels). The individual soil grains that compose a dry and loose granular soil will tend to individually deform, rather than the entire slope deforming as one single block. In a sloping environment, the individual soil particles not only will settle, but also will deform laterally in response to the unconfined slope face.

Bray et al. (2018) state that the pseudo-static slope stability procedures and Newmark sliding block analyses form the basis for a preliminary estimate of the expected seismic displacement of the earth system. However, for critical earth systems, or when liquefaction may occur, dynamic nonlinear effective stress analyses using finite-element or finite-difference methods with robust soil constitutive models may be employed.

It is important to note that the following methods may be employed depending on the design stage, the project's complexity, and the level of associated risk.

Preliminary- Concept Design Stage

- Newmark sliding block; and
- Pseudo-static slope stability procedures;

Detailed Design Stage, Critical Earth Systems, and Liquefaction

- Dynamic nonlinear effective stress analyses using finite-element or finite-difference methods with robust soil constitutive models; and
- Advanced quasi-static effective stress analyses (not appropriate for liquefaction) using finite-element or finite-difference methods with robust soil constitutive models.

3. CODES AND GUIDANCES

3.1 NZTA Bridge Manual Third Edition, Amendment 2 (2016)

According to NZTA BM3, earth retaining systems cover the following elements:

- Non-integral bridge abutments and independent retaining walls associated with bridges;
- Retaining walls not associated with bridges;
- Earth retaining structures (including mechanically stabilised walls and slopes); and
- Slopes designed on the basis of undergoing displacement.

These are classified into the following categories:

- Gravity (concrete, gabion, crib) and reinforced concrete cantilever walls;
- Anchored gravity, cantilever, and soldier pile walls; and
- Mechanically stabilised earth (MSE) walls including soil-nailed walls, reinforced soil walls (inextensible and extensible reinforcement).

The designer shall derive the design loads on the structure in accordance with NZTA BM3 Section 6.2, taking into consideration the flexibility and likely deformation of the structure, and the allowable displacement or deformation of the system. Careful consideration shall be given to the interaction between the structure, the ground and foundations, under static, dynamic, earthquake and construction conditions. The deformation and displacement of the structure shall be compatible with the performance requirements for the structure and its interaction with adjacent or supported structures and facilities. However, due to high seismic demand and/or cost considerations various departures are frequently lodged. Because these changes are introduced by designers with various levels of expertise and experience, in many cases similar structures located in similar settings are treated differently. The main features with respect to geotechnical design stipulated in NZTA BM3 are outlined below;

- Section 6.1.2 Table 6.1 stipulates the maximum acceptable limits of settlements (total and differential) and total horizontal displacements. These requirements are provided without a direct link to a referenced methodology that relates them with a range of seismic coefficients used in pseudo-static stability analysis.
- Section 6.2.2 stipulates that the design earthquake loads to be applied to soils, rock and independent soil structures shall be derived as set out in the same section. The peak horizontal ground acceleration to be applied shall be unweighted and derived for the relevant return period.
- Section 6.2.3 stipulates that for the structural design of earth retaining structures the design horizontal ground acceleration to be used in computing

seismic inertia forces of non-integral abutments and independent walls and of the soil acting against them shall be weighted and derived for the relevant return period and hazard factor.

- Section 6.3.2 stipulates that the factor of safety against instability shall be assessed using conventional slope stability analysis with load and strength reduction factors of one, and the seismic coefficient, k_h, associated with the relevant earthquake accelerations as set out in 6.2.2. If the factor of safety is less than 1 and the failure mechanism is not brittle (such as in rocks where the initiation of failure could substantially reduce the strength of the materials), then the critical seismic coefficient associated with the ground acceleration at which the factor of safety is one, k_γ shall be assessed using large strain soil parameters consistent with the likely displacements due to earthquake shaking.
- When $k_\gamma < k_h$ the displacement likely at the design ultimate limit state seismic response, and under the Major Event (ME) associated with bridge collapse avoidance, shall be assessed using moderately conservative soil strengths consistent with the anticipated stress-strain behaviour and relevant strain levels and a Newmark sliding block displacement approach. Displacements may be assessed using the methods described by Ambraseys and Srbulov, Jibson, Bray and Travasarou.
- Section 6.4.1 specifies that for the assessment of the stability of embankments using pseudo-static seismic analysis, the peak ground acceleration to be applied shall be derived in accordance with 6.2 for the annual probability of exceedance associated with the importance of the slope as defined in 2.1.3. In applying the pseudo-static analysis, the α_{max} shall not be factored down by a structural performance factor or any other factor. However, slopes designed on the basis of undergoing displacement are discussed in Section 6.6.1, where it is specified that embankments designed on the basis of permitting displacement under earthquake response, the requirements of 6.6.9 shall also be satisfied.
- Section 6.6.9 stipulates that vertical accelerations shall be taken into consideration in the design of retaining structures, and that earth retaining structures and slopes may be designed to remain elastic under the design earthquake load specified in 6.2.2 or to allow limited controlled permanent outward horizontal displacement under strong earthquake shaking.

3.2 Supplementary Codes and Guidance

NZTA BM3 Section 6.6.2 and 6.6.9 (in the specific case of mechanically stabilized earth walls and slopes) set out a comprehensive list of design standards and codes of practice that provide guidance on the design of retaining structures, including Road Research Unit Bulletin 84 (RRUB84), NZTA Research Report 239 (RR239), Eurocode 8 – Part 5 and AASHTO LRFD Bridge Design Specifications 2014 which can be used as supplementary documents to clarify any ambiguities. AASHTO and Eurocode are recommended to provide guidance on the design of retaining structures. FHWA-NHI-11-032, LRFD Seismic Analysis and Design of Seismic Design of Transportation Geotechnical Features and Structural Foundations, Reference Manual, (2011) along with Eurocode can provide design methods for both slope (and embankment) and retaining structures.

RRUB84 is stipulated in the NZTA BM3 Sections 6.6.2 and 6.2.4 as the basic document to be used to distinguish the wall elements among rigid/elastic, stiff, and flexible depending on the wall movements. RRUB84 defines three types of walls with different displacement characteristics, retained soil behaviour, and associated seismic coefficients. RR239 identifies the difference between MSE Walls and MSE Slopes based on their face inclination from the vertical. Different types and associated design horizontal seismic coefficients are presented in Table 1 below.

Retaining Structure Type	Related Research/Guidance	Associated Seismic Coefficient (k_h)	
		Internal Stability	External Stability
Rigid/Elastic Wall	RRUB84	1.00 α_{max}	1.00 α_{max}
Non Rigid/Elastic - Stiff Wall	RRUB84	0.75 α_{max}	0.75 α_{max}
Non Rigid/Elastic – Flexible Wall	RRUB84	0.50 α_{max}	0.50 α_{max}
MSE Wall	RR239	$(1.3-\alpha_{max})\alpha_{max}$	0.6 α_{max}
MSE Slope*	RR239	$0.6\alpha_{max}$	0.5 α_{max}

Table 1: Road Research Unit Bulletin 84 and Research Report 239 Retaining wall types

* MSE Walls have faces that are less than 30° inclination from the vertical, while MSE Slopes have faces that are more than 30° from the vertical.

3.3. AASHTO LRFD Bridge Design Specifications (2016)

AASHTO 2016, Section 11.6.5 stipulates that the seismic horizontal acceleration coefficient (k_h) for computation of seismic lateral earth pressures and loads shall be determined on the basis of the α_{max} at the ground surface (i.e., k_{ho} = $F\alpha_{max}$ α_{max} = As, where k_{ho} is the seismic horizontal acceleration coefficient assuming zero wall displacement occurs).

The seismic vertical acceleration coefficient, kγ, should be assumed to be zero for the purpose of calculating lateral earth pressures, unless the wall is significantly affected by near fault effects, or if relatively high vertical accelerations are likely to be acting concurrently with the horizontal acceleration.

If the wall is free to move laterally under the influence of seismic loading, and if lateral wall movement during the design seismic event is acceptable to the Owner, k_{ho} should be reduced to account for the allowed lateral wall deformation. The selection of a maximum acceptable lateral deformation should take into consideration the effect that deformation will have on the stability of the wall under consideration, the desired seismic performance level, and the effect that deformation could have on any facilities or structures supported by the wall.

Where the wall is capable of displacements of 25mm to 50mm or more during the design seismic event, k_h may be reduced to $0.5k_{ho}$ without conducting a deformation analysis using the Newmark method (Newmark, 1965) or a simplified version of it. This reduction in k_h shall also be considered applicable to the investigation of overall stability of the wall and slope.

A Newmark sliding block analysis or a simplified form of that type of analysis should be used to estimate lateral deformation effects, unless the Owner approves the use of more sophisticated numerical analysis methods to establish the relationship between k_h and the wall displacement. Simplified Newmark analyses should only be used if the assumptions used to develop them are valid for the wall under consideration.

Three recommended methods to estimate wall seismic acceleration considering wave scattering and wall displacement are given in Section A11.5 of AASHTO 2016.

a) Kavazanjian et al. (1997): It does not directly address wave scattering and, since wave scattering tends to reduce the acceleration, the first method is likely conservative.
b) NCHRP Report 611—Anderson et al. (2008): It uses a simplified model that considers the effect of the soil mass, but not specifically the effect of the wall as a structure.
c) Bray et al. (2017), Bray et al. (2010), and Bray and Travasarou (2009): It provides a simplified response spectra for the wall, considering the wall to be a structure with a fundamental period. It estimates the value of kh applied to the wall mass considering both the wave scattering and lateral deformation of the wall.

The three alternative design procedures should not be mixed together in any way.

3.4. Eurocode 8 Design of Structures for Earthquake Resistance, Part 5 (2004, 2013)

3.4.1. Slope Stability

Eurocode 8, Clause 4.1.3, stipulates that a verification of ground stability shall be carried out for structures to be erected on or near natural or artificial slopes, in order to ensure that the safety and/or serviceability of the structures is preserved under the design earthquake. Under earthquake loading conditions, the limit state for slopes is that beyond which unacceptably large permanent displacements of the ground mass take place within a depth that is significant both for the structural and functional effects on the structures.

Clause 4.1.3.2 stipulates that an increase in the design seismic action shall be introduced, through a topographic amplification factor, in the ground stability verifications for structures with importance factor γI greater than 1.0 on or near slopes.

Clause 4.1.3.3 stipulates that the response of ground slopes to the design earthquake shall be calculated either by means of established methods of dynamic analysis, such as finite elements or rigid block models, or by simplified pseudo-static methods.

In modelling the mechanical behaviour of the soil media, the softening of the response with increasing strain level, and the possible effects of pore pressure increase under cyclic loading shall be taken into account. The design seismic inertia forces FH (horizontal) and FV (vertical) acting on the ground mass, for the horizontal and vertical directions, respectively, in pseudo-static analyses shall be taken as:

$$FH = 0.5\alpha * S * W$$

$$FV = \pm 0.5FH \text{ if the ratio } a_{vg}/a_g \text{ is greater than } 0.6$$

$$FV = \pm 0.33FH \text{ if the ratio } a_{vg}/a_g \text{ is not greater than } 0.6$$

Where: α is the ratio of the design ground acceleration on type A ground (rock site), a_g, to the acceleration of gravity g (α=a_g/g), a_{vg} is the design ground acceleration in the vertical direction, ag is the design ground acceleration for type A ground, S is the soil parameter of EN 1998-1:2004, 3.2.2.2, and W is the weight of the sliding mass.


ADVANCED GEOSYNTHETIC SOLUTIONS
With you every step of the way!
CIRTEX
0800 CIRTEX | WWW.CIRTEX.CO.NZ

3.4.2. Earth Retaining Structures

Section 7.1 stipulates that permanent displacements, in the form of combined sliding and tilting, the latter due to irreversible deformations of the foundation soil, may be acceptable if it is shown that they are compatible with functional and/or aesthetic requirements.

Section 7.3.1 stipulates that any established method based on the procedures of structural and soil dynamics, and supported by experience and observations, is in principle acceptable for assessing the safety of an earth-retaining structure.

The following aspects should be accounted for:

a) The generally non-linear behaviour of the soil in the course of its dynamic interaction with the retaining structure;
b) The inertial effects associated with the masses of the soil, of the structure, and of all other gravity loads which might participate in the interaction process;
c) The hydrodynamic effects generated by the presence of water in the soil behind the wall and/or by the water on the outer face of the wall.
d) The compatibility between the deformations of the soil, the wall, and the tiebacks, when present.

Section 7.3.2.2 stipulates that for the purpose of the pseudo-static analysis, the seismic action shall be represented by a set of horizontal and vertical static forces equal to the product of the gravity forces and a seismic coefficient. The vertical seismic action shall be considered as acting upward or downward so as to produce the most unfavourable effect.

The intensity of such equivalent seismic forces depends, for a given seismic zone, on the amount of permanent displacement which is both acceptable and actually permitted by the adopted structural solution. In the absence of specific studies, the horizontal (k_h) and vertical (k_γ) seismic coefficients affecting all the masses shall be taken as:

$k_h = \alpha * S / r$

$k_\gamma = \pm 0.5kh$ if a_{vg}/a_g is larger than 0.6

$k_\gamma = \pm 0.33kh$ otherwise

Where, the factor *r* takes the values listed in Table 2 depending on the type of retaining structure. For walls not higher than 10m, the seismic coefficient shall be taken as being constant along the height.

3.5. FHWA-NHI-11-032, LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations, Reference Manual (2011)

Section 6.2.2 states that most slopes can accommodate at least a limited amount of seismically-induced movement. Therefore, the seismic coefficient is always equal to less than the height-adjusted α_{max} and typically is on the order of 50% of the height-adjusted α_{max} for allowable seismic slope deformations on the order of 25mm to 50mm. Therefore, the seismic coefficient will never be more than the site-specific α_{max}. It is typically equal to or less than 50% of the site-specific α_{max}, and in some cases it can be equal to or less than 25% of the site-specific α_{max}.

Section 6.2.3 states that in contrast to the limit equilibrium approach, the displacement-based approach involves the explicit calculation of cumulative seismic deformation. The potential failure mass is treated as either a rigid body or deformable body, depending on whether a simplified Newmark sliding block approach or more advanced numerical modelling is used.

The values of k_{max} are the maximum possible values and only apply to walls of less than 6m in height that cannot

Type of Performance	Type of Retaining Structure	Factor (r)*
Can tolerate deformations/ displacements	Free gravity walls that can accept a displacement up to $d_r = 300\ \alpha$ S (mm)	2.0
Non Rigid/Elastic - Stiff Wall	Free gravity walls that can accept a displacement up to $d_r = 200\ \alpha$ S (mm)	1.5
Rigid/Elastic	Flexural reinforced concrete walls, anchored or braced walls, reinforced concrete walls founded on vertical piles, restrained basement walls and bridge abutments	1.0

Table 2: EC8 reduction factor based on type of retaining structure

* Conceptually, the factor r is defined as the ratio between the acceleration value producing the maximum permanent displacement compatible with the existing constraints, and the value corresponding to the state of limit equilibrium (onset of displacements). Hence, r is greater for walls that can tolerate larger displacements.

accommodate lateral displacement on the order of 25mm to 50mm in the design earthquake. For walls 6m tall or higher, the value of k_{max} can be reduced to account for spatial incoherence (also referred to as wave scattering), or averaging of the ground acceleration over the active (or passive) wedge. For walls that can accommodate at least 25mm to 50mm of lateral displacement in the design earthquake, another reduction for system ductility (allowable lateral displacement) can be applied. These adjustments can be applied to the maximum seismic coefficient to evaluate the design seismic coefficient, kh.

If the earth retention system can be allowed to translate laterally, the seismic coefficient can be reduced to account for the ductility of the wall system. As discussed in the Newmark method permanent seismic displacement is a function of the yield acceleration, the acceleration at which seismic displacement is initiated. For a retaining wall, the yield acceleration is the seismic coefficient at which the horizontal forces (the demand) on the wall system equal the lateral resisting forces (its capacity).

3.6 Earthquake Geotechnical Engineering Practice, Module 6: Earthquake Resistant Retaining Wall Design (2017)

Module 6 is part of a series of guidance modules developed jointly by the Ministry of Business, Innovation & Employment (MBIE) and the New Zealand Geotechnical Society (NZGS). This document deals with earthquake resistant retaining wall design only. The New Zealand Building Act considers the retaining structures as buildings which are as a consequence subject to the requirements of the New Zealand Building Code.

The module discusses in Section 5 the limit equilibrium pseudo-static analysis method and stipulates the derivation of the horizontal coefficient of acceleration, k_h, from the unweighted horizontal peak ground acceleration, α_{max}. In Clause 5.1, k_h is calculated by the following equation:

$$k_h = \alpha_{max} A_{topo} W_d$$

Where, α_{max} is the unweighted horizontal peak ground acceleration, A_{topo} is the topographic amplification factor, and W_d is the wall displacement factor used to reduce the α_{max}

The topographic amplification factor, A_{max}, is adapted from Eurocode 8, Part 5: Part 5: BS EN 1998-5: 2004 (Annex A) and is further refined.

In Section 5.3 it is recognised that designing flexible retaining walls to resist the full ULS peak ground acceleration (α_{max}) is unnecessary and uneconomical in most cases. Most retaining wall systems are sufficiently flexible to be able to absorb high transient ground acceleration pulses without damage because the inertia and damping of the retained soil limits deformations. Wave scattering effects also reduce the accelerations in the backfill to values less than the peak ground motions adjacent to retaining walls. Also, in most cases, some permanent wall deformation is acceptable for the ULS case (refer to Table 4.1).

A non-exhaustive list of performance requirements is provided in Section 4.2 Table 4.1. The wall displacement factor, Wd, is selected according to the amount of permanent displacement that can be tolerated for the particular design case. Reducing the design acceleration by Wd implies that permanent movement of the structure and retained ground is likely to occur. Several other assumptions are implied, including that:

a) The retaining structure is sufficiently resilient or ductile to withstand the movement;
b) The supporting soils are not susceptible to strength loss with straining; and
c) Any supported structures or services can tolerate the movement.

The intent of that table is to give guidance on selecting seismic design parameters for retaining structures. The movements indicated are for typical cases and represent permanent movement from a single design earthquake for selecting appropriate design acceleration coefficients. Instantaneous dynamic movements during an earthquake will be greater and there may be additional movements from gravity loads prior to an earthquake. Some buildings will be more sensitive to movement than others and it is the designer's responsibility to ensure that movements can be tolerated.

4. DISCUSSION AND RECOMMENDATION

The five documents presented in the previous sections above, and other references as well, accept that earthquakes produce ground motions that in turn induce inertia forces of an alternating nature in slopes and earth retaining structures. The alternating inertia forces are of short duration and change direction many times. Therefore, even though the C/D ratio (i.e. factor of safety, FoS) during a cycle of earthquake loading may fall below one, it will usually remain below one for only a very brief period of time, until the load reverses. During the interval when the C/D ratio is below one, permanent displacement will accumulate. However, only limited displacements will occur during any one interval because of its short duration.

In summary, the documents follow the logic shown below:

- Some types of retaining structures may be considered rigid/elastic; Eurocode 8 explicitly stipulates what these are;

- Most slopes, natural and man-made, and earth retaining structures under seismic load exhibit internal deformations (i.e. the soil mass reaches a plastic state) and do not perform as rigid – elastic structures;
- To account for this when using the pseudo-static method for analysis, the seismic coefficient used is a percentage of the αmax and is a function of the α_{max}, the magnitude of the earthquake, the geometry of soil structure, and of a certain level of acceptable displacements;
- Vertical accelerations are used where applicable and Eurocode 8 provides a straightforward calculation.

4.1 Proposed Roadmap for Decision Process on Acceptable Displacements

In many practical situations some small-to-moderate amount of permanent displacement is acceptable after the design seismic event. Small permanent displacements are not likely to be life threatening and can be repaired by removing or placing earth. However, there are locations where even small slope or embankment movements may affect nearby structures or present a risk to public safety. In such cases, even a few centimetres of movement may not be acceptable.

If permanent movement of the slope is acceptable, there are significant benefits to the Owner. However, an overriding question in any approach that involves permanent deformations is the amount of deformation that is acceptable. Ultimately, that decision belongs to the Owner, who must weigh a number of factors in reaching this decision.

Factors that should be considered when deciding on acceptable levels of permanent displacement include the implications of the movement and the likely mode of slope movement. When considering these factors, the Owner should evaluate both the relative consequences of movement and, as appropriate, the cost of designing to avoid the movement.

One of the main factors for deciding on the acceptable level of movement involves the location and function of the slope. The type of soil at a site also should be considered when establishing displacement limits. This consideration is related to both the type of failure mechanism and the response of the slope to loads.

Although NZTA BM3 in Table 6.1 specifies the maximum total horizontal displacements of soil structures, these are not directly linked to a referenced methodology that relates them with the seismic coefficients used in pseudo-static stability analysis and do not cover all potential cases and locations/significance of soil structures.

Similarly, Module 6 in Table 4.1 stipulates the performance requirements without a direct link to a referenced methodology that relates them with the seismic coefficients used in pseudo-static stability analysis.

If displacements of more than 100mm are being considered, a rigorous review of the possible consequences of movement of the slope or embankment should be conducted. Figure 1, adapted from Figure 7-22 of FHWA-NHI-11-032, shows the steps that the Owner might use to define a design approach and an acceptable limit to permanent displacement.

4.2 Advanced Roadmap for Performance Based Seismic Design

Taking into consideration the documents discussed in the previous sections, international practice and the current state of the art in Geotechnical Earthquake Engineering, the following two Performance Based Design methodologies are proposed to be used to deliver safe and value engineering solutions. The two proposed methods are a basic and an advanced one, the use of which depends on the complexity and criticality of the project. The soil-foundation-structure-interaction aspects are critical in all cases and risk levels to provide both safety and an economic design.

The basic option is adapted from Section 6.2.3 of FHWA-NHI-11-032 to cover both slopes and earth retaining structures. It is intended to be used for preliminary, feasibility, and simpler concept design phases. The advanced method was introduced and applied at NCTIR in 2017 for the design of the coastal realignment seawalls and for the concept design of the northern realignment at the Sandpit. This method follows Performance Based Design and Soil-Foundation-Structure-Interaction principles, is intended to be used in concept design phase of complex sites, and in detailed design phases, and comprises the following steps.

a) Select appropriate soil constitutive model and properties that are compatible with the stress and strain levels of each loading case. It may be required to adopt different strength and stiffness properties for each load case (static and seismic).
b) Carry out static slope stability analyses using appropriate resistance factors to confirm that slope performance meets static loading requirements;
c) In the case of MSE walls and slopes, carry out LRFD design of the internal stability in accordance with AASHTO LRFD Bridge Design Specifications (2016) for three limit states (Service, Strength, and Extreme) to assess the pull-out resistance and the tensile resistance and strength of the reinforcement;
d) Carry out two dimensional (2D) stress-strain analyses [finite element software (FE) are generally adequate


ROCKFALL DEFENCES


REMOTE DRILLING & STABILISATION


REMOTE MAINTENANCE & INSPECTION

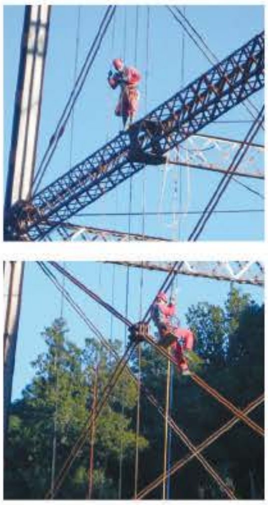
Delivering solutions & experience where they're needed!

Avalon has grown to become a well respected key play in the New Zealand high/difficult access field.

Our management style remains 'hands on' and we pride ourselves on a high degree of availability and excellent client communication.

Our reputation brings the best staff to use allowing us to deliver you a service that is second to none.


ROCKFALL PROTECTION SYSTEMS


COMPLEX INSPECTION


REMOTE MAINTENANCE

EXPERIENCE

Since 1996 we have delivered throughout New Zealand backed by engineering and difficult access competency spanning more than 30 years.

INNOVATION

A commitment to deliver optimised solutions that achieve maximum results for the client.

PERFORMANCE

Its simple to us - ON TIME, ON BUDGET; its what we deliver every time.

REMOTE HEIGHT CONSTRUCTION

GENERATION INSTALLATION & MAINTENANCE


REMOTE ACCESS CLEANING


CONFINED SPACE ACCESS


HEIGHT SAFETY SYSTEMS

We minimise your risks!

AVALON
// ROPE ACCESS SERVICES

AVALON INDUSTRIAL SERVICES LTD

PO BOX 5187, 94 HIGH STREET, FRANKTON,
HAMILTON 3242, NEW ZEALAND

P +64 7 846 1686 **F** +64 7 846 1016
E admin@avalonltd.co.nz **W** avalonltd.co.nz

for unreinforced slopes and simple retaining systems, finite difference software (FD) are generally adequate for all plus reinforced slopes and complex retaining systems] to optimise the solution and to provide a holistic, internal and global stability analysis;

e) The model should be setup with discrete approach, i.e. all structural elements and significant soil elements are modelled with appropriate interfaces, so that complete Soil-Foundation-Structure-Interactions are taken into account and can be monitored;

f) Interfaces with appropriate limiting shear strengths should be inserted between structural and soil elements, structural fill and backfill if necessary, and the foundation material;

g) Static, quasi-static, and/or time history effective stress analyses, using the 2D FD or FE approach are undertaken to assess the internal and global stability, the overall performance, and the Soil-Foundation-Structure-Interaction within the soil or earth retaining system, its comprising elements, and with the adjacent materials;

h) The time history analyses are carried out with appropriate constitutive models that are carefully calibrated and their properties reconciled, and appropriate time history records inserted as excitation;

i) The quasi-static analysis is undertaken implementing an incremental/stepwise earthquake loading procedure to assess the propagation of internal and external deformation, the yield acceleration, and ultimately the mode and magnitude of deformation;

j) The long term performance of the soil or earth retaining system is further assessed implementing stochastic analysis that simulates random degradation of the shear and tensile strength of the fill materials and the overall performance under quasi-static seismic actions;

k) Evaluate the acceptability of the displacement based on performance criteria established by the owner for the specific project site.

The advanced method can be expanded to include three dimensional (3D) numerical analysis following the same steps described above.

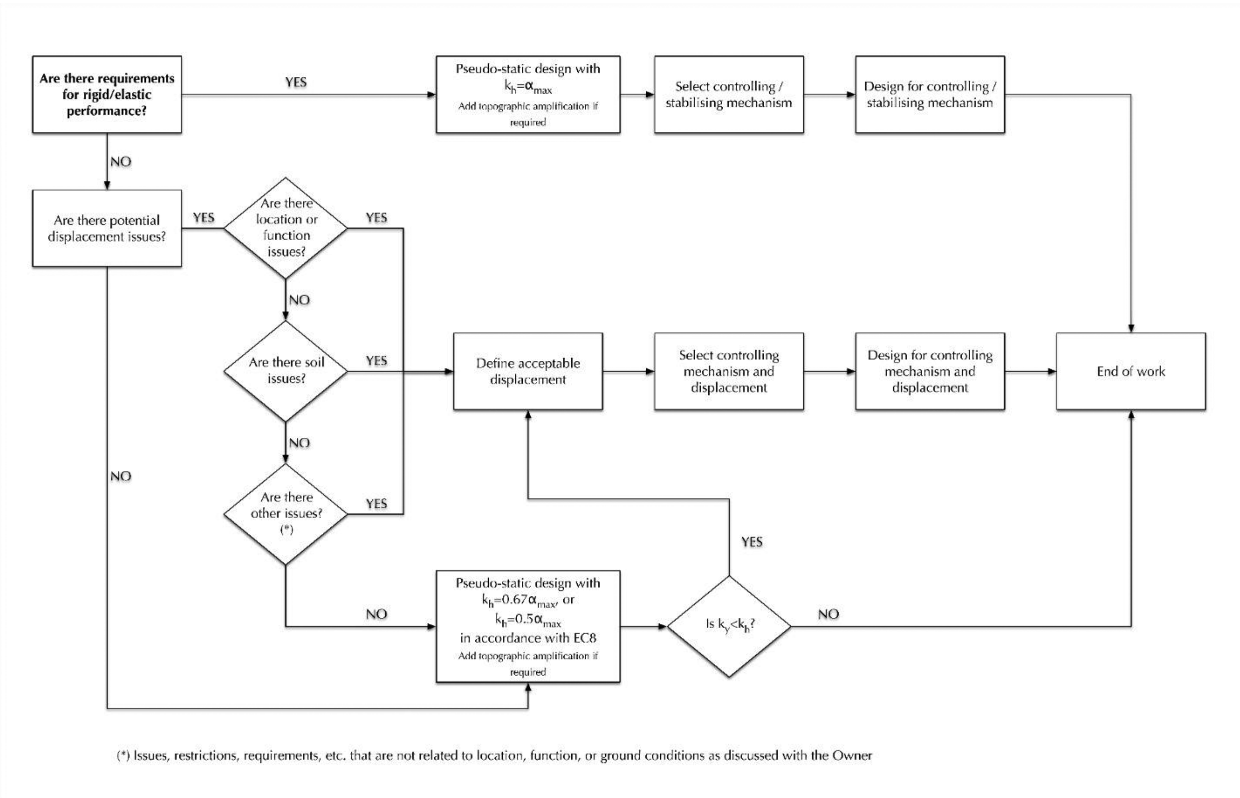


Figure 1: Procedure to define a design approach and to establish acceptable slope / earth retaining structure movement (adapted from Figure 7-22 of FHWA-NHI-11-032)

5. REFERENCES

AASHTO, 2016, LRFD Bridge Design Specifications.

Ambraseys, N.N., and Menu, J.M., 1988, Earthquake-induced ground displacements: Earthquake Engineering and Structural Dynamics, v. 16, p. 985-1006.

Bray, J.D., and Travasarou, Thaleia, 2007, Simplified procedure for estimating earthquake-induced deviatoric slope displacements: Journal of Geotechnical and Geoenvironmental Engineering, v. 133, p. 381-392.

Bray, J.D., Macedo, J., and Travasarou Thaleia, 2018, Simplified procedure for estimating seismic slope displacements for subduction zone earthquakes: Journal of Geotechnical and Geoenvironmental Engineering, v. 144(3).

Dakoulas, P., and Gazetas, G., 1986, "Seismic Shear Strains and Seismic Coefficients in Dams and Embankments." Soil Dynamics and Earthquake Engineering, vol. 5, no. 2, pp. 75–83.

Day, R.W, 2002, "Geotechnical Earthquake Engineering Handbook", McGraw-Hill, 614 p.

Earthquake Geotechnical Engineering Practice, 2017, "Module 6: Earthquake-resistant retaining wall design", NZGS & MBIE.

Eurocode 8, 2004 & 2013, Design of Structures for Earthquake Resistance, Part 1 and 5.

FHWA-NHI-11-032, 2011, LRFD Seismic Analysis and Design of Seismic Design of Transportation Geotechnical Features and Structural Foundations, Reference Manual.

Jibson, R.W., 2007, Regression models for estimating coseismic landslide displacement: Engineering Geology, v. 91, p. 209-218.

Kramer , S. L., 1996, "Geotechnical Earthquake Engineering", Prentice-Hall, Inc., Upper Saddle River, New Jersey, 653 p.

Madabhushi S.P.G., I. Thusyanthan, Z. Lubkowski & A. Pecker, 2009, Shallow foundations: Chapter 8 in Seismic Design of Buildings to Eurocode 8, Edited by A.Y. Elghazouli, Spon Press.

Newmark, N., 1965, Effects of earthquakes on dams and embankments, Geotechnique, 15(2), 139–160.

NCHRP (National Cooperative Highway Research Program), 2003, NCHRP Project 12-49, Recommended LRFD Guidelines for the Seismic Design of Highway Bridges, Part I, Specifications, Part II, Commentary and Appendices, Submitted by MCEER/ATC, Report MCEER/ATC 49, and Liquefaction Study Report, Report MCEER/ATC 49-1.

NCHRP (National Cooperative Highway Research Program), 2008, NCHRP Project 12-70, Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes and Embankments, Recommended Specifications, Commentaries and Example Problems NCHRP Report 611, Transportation Research Board, Washington, DC.

NZTA, 2016, Bridge Manual SP/M/022, Third edition, Amendment 2

Comparison of predicted and observed seismic performance of Kekerengu and Tirohanga bridges during Kaikōura 2016 Earthquake

Jawad Arefi
Jawad is a chartered geotechnical engineer at Beca in Christchurch. He is specialized in dynamic site response analyses and liquefaction assessment. He has been involved in broad geotechnical projects covering a variety of multi-disciplinary civil, industrial and building projects including roads and bridges, ports, low-rise structures, and liquefaction mitigation designs.

Richard Young
Richard is a Geotechnical Principal at Beca with over 30 years of international experience. Working closely with his structural colleagues on projects including NCTIR, Christchurch Southern Motorway and Transmission Gully, and drawing on his learnings from the Canterbury Earthquake sequence, he has developed a detailed understanding of seismic ground and structure interaction.

Nik Stewart
Nik is a Senior Associate with Beca and has over 22 years of structural experience in bridge and civil structural projects. Specialising in seismic design and assessment, he has recently worked on the North Canterbury Transport Infrastructure Recovery Project (NCTIR), SCIRT and NZ Transport Agency's seismic bridge assessments

This paper was prepared for the 12th ANZ Young Geotechnical Professionals Conference (12YGPC) in Hobart on 7-9 November 2018

ABSTRACT

On November 14, 2016 at 12:02 a.m. local time, the Mw 7.8 Kaikōura earthquake occurred along the east coast of the upper South Island, New Zealand. The earthquake affected a relatively large area and significant impacts occurred to the horizontal infrastructure in the region. This paper focuses on the effects of the ground shaking on two bridges - Tirohanga Stream Bridge and Kekerengu River Bridge along State Highway 1 (SH1). As a result of this earthquake, a new bridge and associated embankments have been constructed at the Tirohanga site and the Kekerengu Bridge has been repaired. One year prior to this earthquake, the same two bridges were the subject of a geotechnical and structural seismic assessment initiated by the New Zealand Transport Agency (NZTA) as part of a programme for assessing the seismic performance of bridges on national strategic routes. This paper compares the predicted seismic performance of the bridges prior to the earthquake and their observed performance during the Kaikōura earthquake. It provides lessons learned for young geotechnical engineers to consider for seismic assessments of existing structures and valuable insights into their performance, and how uncertainties can be accounted for.

1. INTRODUCTION

Tirohanga Bridge and Kekerengu Bridge are located on the eastern coastal margin of the upper South Island, sandwiched between the uplifting coastal Kaikōura range and the sea. Both bridges are located on SH1, Kekerengu approximately 60km northeast of Kaikōura and Tirohanga Stream Bridge a further 6km north.

Prior to the Kaikōura earthquake a series of detailed seismic assessments were undertaken for a number of bridges as part of NZTA's program for assessing and, where economically justifiable, retrofitting bridges on national strategic routes. Tirohanga and Kekerengu bridges were assessed as part of this scheme by Beca Ltd (Beca) during 2015 – 2016 (Carding & Stewart 2016, Stewart 2016). The objective of these assessments was to obtain a best estimate of the expected seismic performance of the existing bridges and, where appropriate, implement retrofitting measures. In agreement with NZTA, the performance requirement adopted for the seismic assessment was acceptable levels of damage under a 500 years return period event or damage control limit state (DCLS), and no collapse under an event with a 1,500 years return period. Under the DCLS earthquake, the NZTA Bridge Manual (BM) stipulates that the structure shall be usable by emergency traffic although damage may have occurred, and some temporary repairs may be required to enable use.

At 12:02 a.m. on 14th November 2016, an M_w 7.8 earthquake occurred in the northeast of the South Island of New Zealand causing fault rupture, ground shaking, liquefaction, and landslides. It ruptured the surface in a north eastward direction over a distance of approximately 180km crossing at least twelve faults and initiating up to 10m of vertical and 11m of horizontal movement. This event uplifted the land relative to the sea by 1 – 2m (refer to 2017 NZSEE special bulletin 2017 for details).

Following the earthquake, there was a discussion between KiwiRail and NZTA about how to get road and rail infrastructure back up and running and the question of whether to abandon the coastal road and instead develop an alternate road, was raised. During these discussions, it quickly became clear that repairing and reopening the existing coastal route was the most viable option due to its shorter distance, lower cost of upgrading and higher reliability during similar events. Hence, the New Zealand government announced the formation of the North Canterbury Transport Infrastructure Recovery (NCTIR) alliance, between NZTA, KiwiRail, and several major contractors. The purpose was to repair the road and rail networks to reconnect communities with a more resilient network. Following this, NCTIR inspected the affected road alignments, embankments and bridges including our subject bridges i.e. Tirohanga Stream Bridge and Kekerengu River Bridge and proposed rebuild and repair strategies for each structure.

In general, damage to some bridges occurred in the earthquake with some bypassed, but most were able to be opened quickly with restrictions imposed. Observations following the quake suggested that most of the structural damage sustained by the bridges was due to inertial loading with the damage resulting from geotechnical issues believed to be secondary. Many bridges suffered minor to moderate structural damage due to the development of plastic hinges at the piers. Settlement at bridge abutments was widespread and in some cases prevented access over the bridges until fill or asphaltic concrete was placed to form ramps onto the bridges (refer to 2017 NZSEE special bulletin 2017 for details).

Although not part of the initial study, observations of the performance of a KiwiRail Bridge located approximately 100m to the east of the road near the Tirohanga Bridge, the Stinking Stream Bridge located 130km south west of Tirohanga site in addition to Flaxbourne River Bridge and Needles Creek Bridge located circa 25km north east of Kekerengu Bridge, during the Kaikōura earthquake has proved informative.

This paper presents the impact of Kaikōura earthquake on the Tirohanga Stream and Kekerengu River Bridges and compares the observed performance with performance predicted by the seismic assessment, providing lessons learned for geotechnical engineers to consider for seismic assessments of existing structures.

2. PRE-EARTHQUAKE ASSESSMENT

2.1 TIROHANGA STREAM BRIDGE

The GNS QMAP indicated that the site was underlain by alluvium, comprising river gravel and sand including modern river beds. This was indicated to be dominantly gravel with subordinate sand and silt. There was no ground investigation undertaken at this site prior to earthquake, however observations made during the site walkover of the eastern abutment indicated near surface highly plastic firm silt and silty sand.

In the absence of any intrusive ground investigation data, but based on the geological map and inferred sloping bedrock topography, it was surmised that the site subclass would likely be D (deep or soft soil) but that a sensitivity check be undertaken considering it as Class C and even possibly Class E. In addition, both drained (coarse-grained) and undrained (fine-grained) properties for the abutment soils were proposed based on the site observation of surface silty material at the eastern abutment and dominant gravel given on the geological map. The intention was to estimate representative rather than moderately conservative geotechnical parameters. Without site specific data the following were applied:-

- Coarse grained soils: ϕ' = 30°, K_p = 5 and horizontal modulus of subgrade reaction = 5.z (MN/m²/m)
- Fine grained soils: C_u = 25 + 10.z (kPa), $P_p = \gamma.z + 2\ C_u$ and horizontal modulus of subgrade reaction = 15 (MN/m²/m).

The active Kekerengu fault was located only 100m south of the site and it was emphasized that the surface expression of fault rupture could affect the bridge. As the alluvium material could contain horizons of sand, and

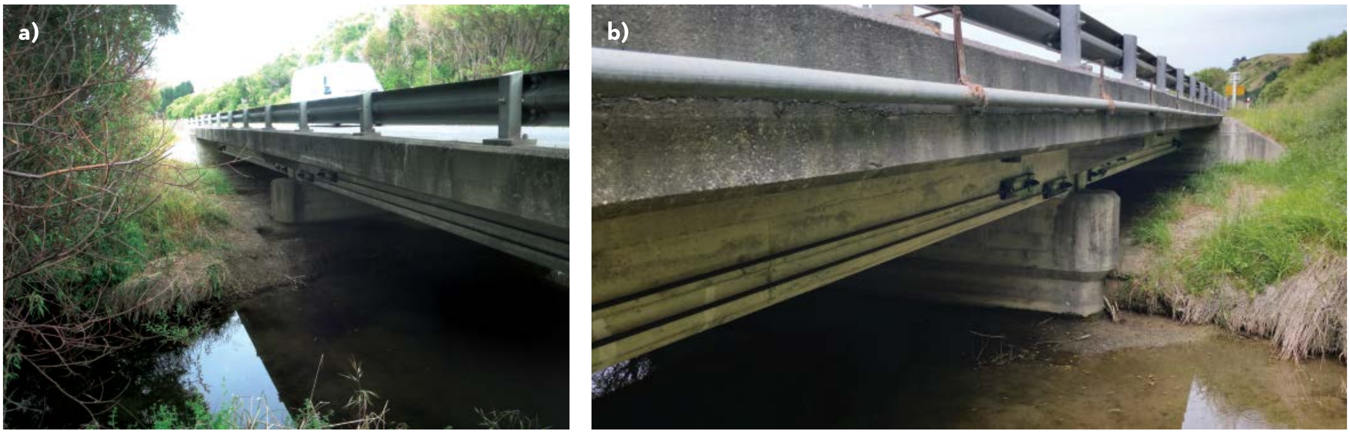


Figure 1: a) Tirohanga Bridge deck and pier, looking south, b) pier and abutment pre earthquake, looking north

because of the high hazard factor, the liquefaction risk was considered to be medium.

Tirohanga Stream Bridge comprises two spans of cast in situ reinforced concrete T-beams. The central pier and the abutments each comprise a cast in situ reinforced concrete wall supported by octagonal precast concrete driven piles, 9.75m and 11m long, respectively. Figure 1 shows the abutments and also the wall pier. It was noted that the bridge had been structurally strengthened in the past with tension bars along the deck beams to increase the vertical load-carrying capacity.

The seismic assessment identified that damage to the piles and abutment walls under the DCLS earthquake would likely comprise cracking and spalling at the top of the piles, with some deformation and settlement at the abutments. Shear failure of the piles under transverse loading under a seismic event with a return period of 1,300 years was predicted. It was surmised that this shear failure could result in loss of support, leading to local collapse or settlement of the bridge. However, as the bridge was shown to have acceptable levels of damage, and the structural components of the bridge to have adequate capacity under DCLS loading, seismic strengthening was not recommended.

2.2 KEKERENGU RIVER BRIDGE

The GNS QMAP indicated that the site is underlain by alluvium comprising river gravel and sand including modern river beds. Approximately 100m inland the ground rises with the QMAP indicating the slopes to be underlain by Waimea Formation silty mudstone. As with Tirohanga, no intrusive ground investigation was undertaken, however during a site walkover it was observed that the northern abutment was founded on mudstone and the southern on alluvial soil.

In the absence of any intrusive ground investigation data, but based on the geological map and site observations, the subsoil class was considered likely to be Class C (shallow soil) at the southern abutment and possibly Class B at the northern abutment, with a recommendation that a sensitivity check be undertaken for both. Drained (coarse-grained) and undrained rock mass properties were applied to the south and north abutments respectively, with the backfill to the north abutment assumed to be coarse-grained. Without site specific data the following parameters were applied:-

- Coarse grained soils: ϕ' = 30-34°, K_p = 5-6 and horizontal modulus of subgrade reaction = 10.z (MN/m²/m)
- Rock mass properties: K_p = 10 and horizontal modulus of subgrade reaction = 90 (MN/m²/m)

The horizontal stiffness of the rock abutment side was estimated to be between 3 and 9 larger, and the strength 2 times larger, than the south abutment.

Similar to Tirohanga Bridge, Kekerengu Bridge is located in an area of high seismicity with the Clarence Fault within 20km of the bridge and the Kekerengu Fault within one kilometre.

The seismic input parameters adopted for the assessment are given in Table 1.

Subsoil Class	1 in 500 year return period	
	PGA	Mw
Class C	0.56	7.5
Class D	0.47	7.5

Table 1: seismic input parameters adopted for the assessment

The 76m long Kekerengu River Bridge was constructed circa 1940. It is a five span, simply supported structure, the cast-in-situ reinforced concrete deck supported off four T-beams which were cast integrally with the abutments and semi integrally with the four piers. The northern abutment

Figure 2: Kekerengu River Bridge, bridge overview looking south (left); north abutment on rock (middle); south abutment on soil (right) prior to earthquake

is a reinforced concrete wall founded directly on the mudstone rock and the southern abutment is a framed structure comprising a deep capping beam with four short columns and a spread footing founded below the river bed level. The piers are founded on spread footings. The southern abutment is fronted by a sloping wire mattress faced revetment. The superstructure is monolithic with the abutments. Figure 2 provides a general view of the bridge, including the conditions at both abutments.

The bridge was assessed in both the transverse and longitudinal directions using a nonlinear static push-over analysis. The performance of the bridge was dependent on the stiffness and capacity of the soil, therefore sensitivity checks were carried out considering double and half the estimated soil stiffness and capacity. In the longitudinal direction, the seismic load was considered to be resisted by the out of plane bending of the walls and the lateral capacity of the soil surrounding the embedded portion of the walls. Transverse seismic loads were considered to be resisted by the in-plane bending capacity, shear capacity and overturning capacity of the pier and abutment walls which transfer the loads to spread footings and then to the ground through vertical bearing. Lateral transverse resistance is provided by the sliding capacity of the walls and partly by the passive resistance of the soil against the sides of the embedded walls. Comparing the results of the analyses for the expected soil properties in each of the longitudinal directions, it was concluded that the performance of the structure is similar in both directions.

In the longitudinal direction plastic hinges were assessed to form in the pier-deck connections at the top of the pier cap beams and at the top of the northern abutment. The southern abutment columns were estimated to start to yield at a return period of 200 years. The plastic rotation capacity of northern abutment wall was assessed to be exceeded at a return period in excess of 1,500 years. In the transverse direction the bridge exhibited satisfactory behaviour and met the seismic performance requirements up to a return period in excess of 2,500 years.

In general, the assessment was found to be quite sensitive to the selection of appropriate site subsoil class. If the results of any subsequent geotechnical investigation concluded it was a Class D (deep or soft soil) site a strengthening scheme would be needed to be developed. However, based on the best estimate of the ground conditions it was concluded that the bridge complies with the damage control seismic performance requirements that, under the 500 year return period design earthquake, and the risk of collapse was low and the structure would be usable after temporary repairs have been carried out.

3. SEISMIC RESPONSE OF THE TWO BRIDGE SITES

The subject sites experienced a Modified Mercalli VII (i.e. very strong) shaking level during the Kaikōura earthquake. Two strong motion stations recorded the ground shaking near this area. These were Kekerengu Valley Road (KEKS) station and Ward Fire Station (WDFS) located on site subsoil Class B (rock) and Class D (deep or soft soil), respectively (Figure 3). Reviewing the accelerograms recorded by these two stations provides valuable insights on the ground shaking experienced in this area. Figure 3 shows the locations of the two bridges in relation to nearby strong motion stations and the trace of fault rupture during Kaikōura earthquake. The two bridges are approximately 6km apart and similar a distance from the KEKS station. The WDFS station is some 20km north east of the Tirohanga Stream Bridge.

Although the epicentre of the earthquake was 100km south west of the sites, it appeared that there was dominant energy release near the subject sites (refer to 2017 NZSEE special bulletin 2017 for details). Severe damage was induced by surface rupture across SH1 in particular near to the Tirohanga site, where the road was closed.

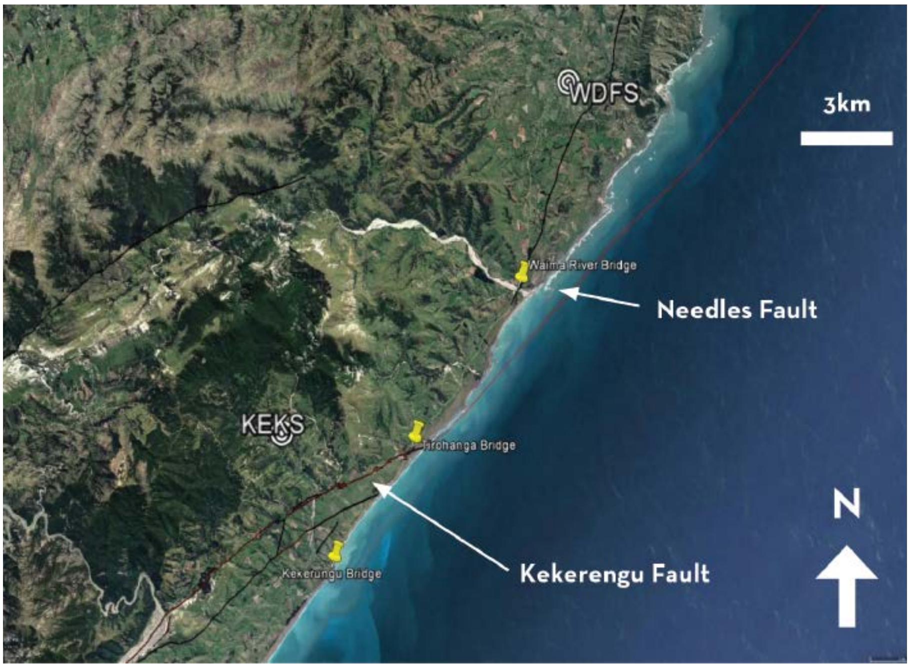


Figure 3: Aerial map of subject sites, black line is the active fault trace from GNS Science NZ active fault map, the red line are the segments that ruptured during Kaikōura earthquake, the yellow line shows the current SH1 road alignment (base map from Google Earth)

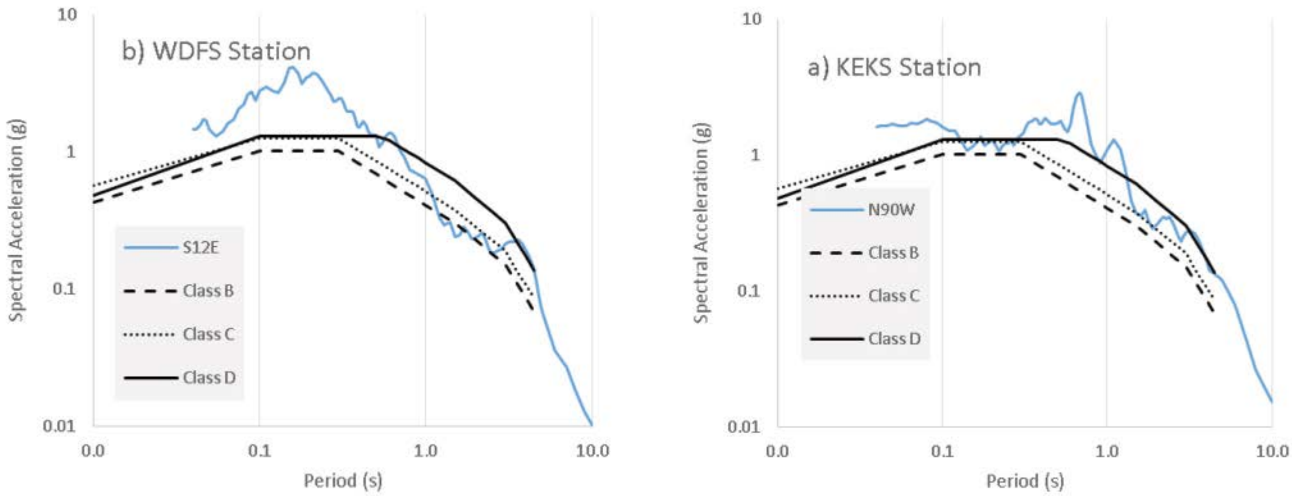


Figure 4: Horizontal 5% damped pseudo-acceleration response spectra observed in: a) KEKS station; and b) WDFS station. The NZS 1170.5:2004 design spectra are drawn for a return period of 1 in 500 years

The acceleration response spectra of the two strong motion stations, were calculated by GNS Science using the recorded accelerograms (Figure 4). It can be seen that at the KEKS and WDFS stations very large short period accelerations (including peak ground acceleration) occurred, far exceeding the design spectra. The design spectra are calculated based on NZS 1170.5:2004. This standard has produced a hazard map for New Zealand which gives an appropriate indication of the seismicity of the subject sites; the hazard factor ranges from 0.13 to 0.62 across the country and for the Tirohanga and Kekerengu bridges area, it is approximately within the range of 0.4 – 0.45. The design spectra presented in Figure 4 are evaluated assuming a 1/500 year annual probability of exceedance to be consistent with the performance requirements adopted for the seismic assessment undertaken prior to the earthquake (refer to Section 2). Another interesting point regarding the seismic demand imposed by the Kaikōura earthquake was the high vertical accelerations, which were approximately 0.40g recorded by nearby strong motions stations and more than 1g in other areas.

Although, the recorded accelerograms at the KEKS and WFDS provide insights into the response of the ground during shaking, they may not be representative based

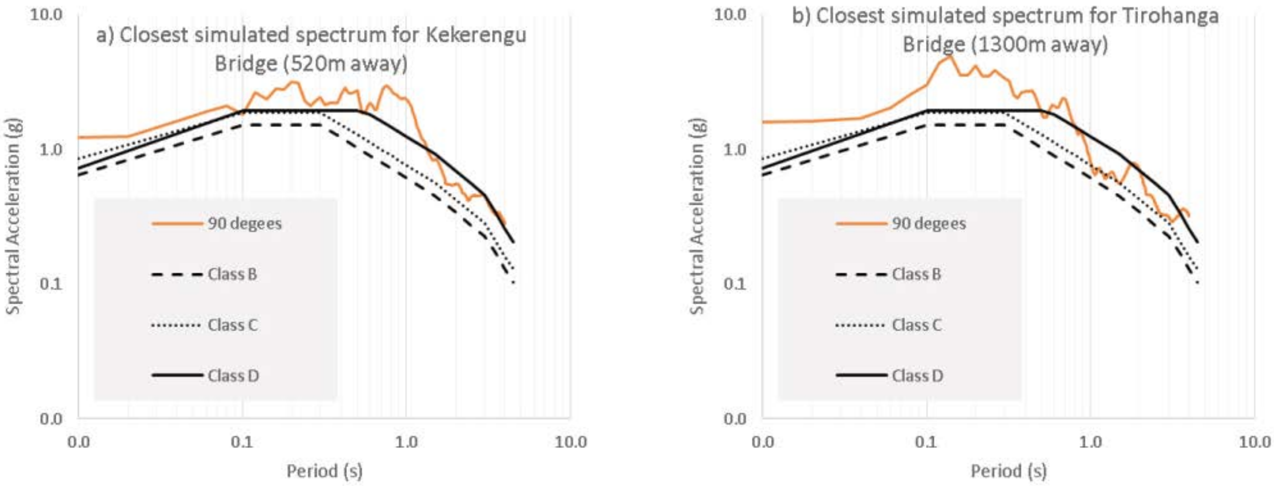


Figure 5: Horizontal 5% damped pseudo-acceleration response spectra simulated for a) Kekerengu Bridge and b) Tirohanga Bridge. The NZS 1170.5:2004 design spectra are drawn for a return period of 1,500 years

on their distance away from our subject sites. Local site effects at the strong motion stations can be significantly different from the bridge sites, hence the local ground motion at the two sites was compared with the simulated ground motion by SeisFinder (Seisfinder 2018). SeisFinder is a web application which adopts computationally-intensive earthquake resilience calculations to generate estimated local ground motion simulations.

Figure 5 illustrates the spectral acceleration plots for the nearest SeisFinder simulation stations which are 0.5km and 1.3km respectively from Kekerengu and Tirohanga bridges. By inspection it can be seen that they are broadly similar to those recorded at the KEKS and WDFS stations and, due to the lack of any closer ground motion records, are considered representative of the seismic response of the ground at our sites, for this study.

The seismic assessment of the bridges prior to the Kaikōura earthquake determined that the natural period of the two bridges depended largely on the stiffness of the soil as well as the extent of nonlinearity during the target seismic shaking. However, for the level of damage observed after the earthquake, it is surmised that the natural period of the two bridges are within the range of 0.2 – 0.4 seconds. Figure 5 illustrates that both bridges have experienced a much larger levels of shaking, including corresponding peak ground accelerations (PGA), than the design 1s corresponding to 1 in 500 and 1,500 years return periods.

4. POST-EARTHQUAKE

4.1 TIROHANGA STREAM BRIDGE

4.1.1 Ground conditions

In 2017 NCTIR commissioned seven boreholes and 14 CPTs along the road alignment, which identified that the ground comprises 6m of clayey silt over sandy gravel, with the clayey silt layer possibly being embankment fill used in the bridge approaches. It was determined that the site Soil Class is D. Under the ULS design criteria the site was considered prone to liquefaction, resulting in an estimated 25 – 100mm of free-field liquefaction-induced settlement.

4.1.2 Post-earthquake inspection

NCTIR's inspection of the bridge identified that it sustained minor damage comprising settlement of approaches and minor displacement of the piled wing walls relative to the abutments. No surface expression of liquefaction was seen anywhere at the site. In addition, there was no slumping or slope failures evident on the sand dunes on the seaward side of the site and no evidence of lateral spreading of stream banks was found (pers. comm. Nick Tunnicliffe 2018).

Most notably, around 180m south west of the bridge site, where the Kekerengu fault crosses SH1, severe road damage was observed. The fault rupture caused 2.5m of vertical and 5m of horizontal ground movement (Figure 6). The lowering of the ground level on the northern side of the fault allowed water to pond on the ground surface (Figure 7a). As such, flooding during future significant rainfall events was identified as a major risk, which contributed to the proposal to construct a new bridge to replace the existing one.

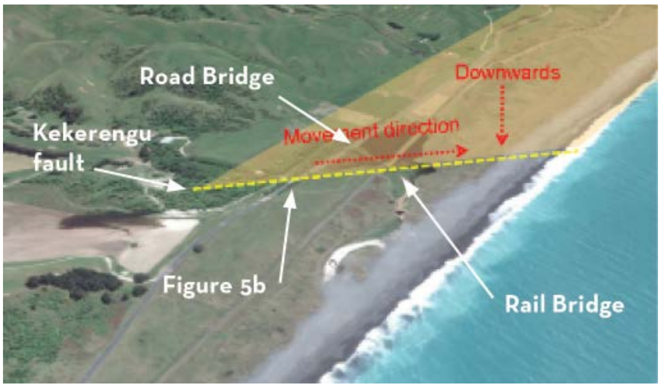


Figure 6: a) Strike-slip fault causing, ~2.5m downwards and 5m horizontal eastwards b) Road damage 185m away from bridge site (Photos courtesy of NCTIR)

Another interesting observation on site was the performance of a KiwiRail Bridge, approximately 100m to the east of the road. Although no slumping or slope failure was observed on the elevated approach embankments, which are located on similar ground conditions to the road bridge, the rail bridge sustained severe damage to the extent that girders unseated from an abutment (Figure 7b). The damage to the rail bridge could be attributed to the proximity to Kekerengu fault trace and also high vertical accelerations.

The new road bridge is shown in Figure 7c and is constructed approximately on the same alignment, but at a significantly higher elevation to avoid flooding risk.

4.2 KEKERENGU RIVER BRIDGE

4.2.1 Ground conditions

Due to the comparatively good performance of the bridge and the requirement for only minor repairs, no subsequent investigation was carried out.

4.2.2 Post-earthquake inspection

The bridge was inspected during the initial response by NCTIR (MacDonald 2018). According to the inspection report the bridge structure withstood the earthquakes well and demonstrated considerable robustness, although the bridge approaches were damaged due to settlement and horizontal movement. Minor subsidence in the approach embankment zones, surface undulations at abutments, and relatively minor approach and pavement damage in the form of longitudinal and transverse cracking was observed. The approaches were reconstructed through the placement of fill and resealing, which quickly made this road bridge usable again. Structurally, the post-earthquake inspection also concluded that there was only minor damage including some vertical and diagonal cracking to the pile cap and concrete beams at southern abutment (Figure 8). The diagonal crack in the pile cap implies a vertical punching shear failure which had begun to manifest itself due to the earthquake shaking. No cracking was observed at the top of northern abutment.

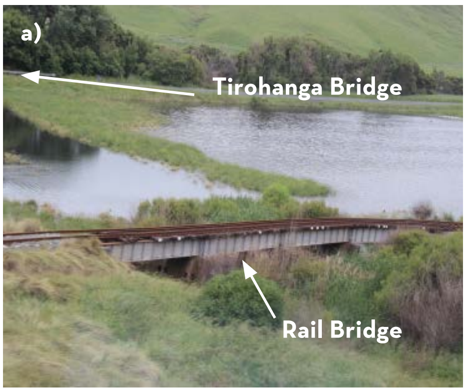


Figure 7: a) Flooding of the bridge site after earthquake, b) Rail Bridge unseated girder, c) new Tirohanga road and Rail bridges (photos courtesy of NCTIR and GEER)

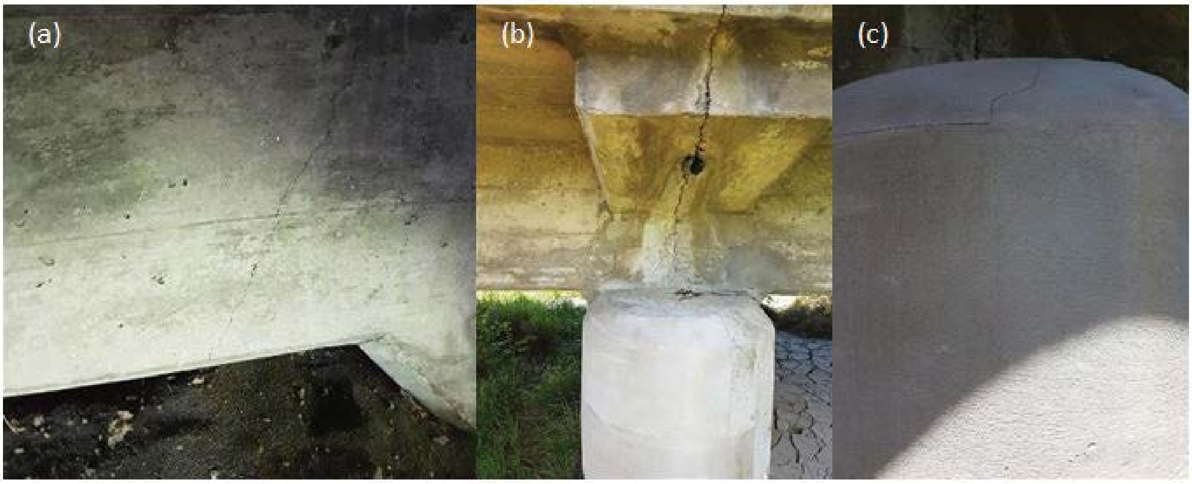


Figure 8: a) Minor cracking of capping beam at southern abutment, b) opening of pre-existing horizontal cracks of construction joints at the beam soffits at southern pier c) cracked concrete repairs at the top of pier (photos courtesy of NCITR)

The faces of the abutments and wing walls appeared to remain vertical. Hence, it was concluded that seismic vulnerabilities and severity of localised failures envisaged for the abutments and piers had not manifested themselves as suggested by the pre-earthquake assessment.

Despite the high level of ground motion experienced, observational evidence shows that neither of the two bridges showed any signs of significant earthquake damage. However, contrary to these two bridges, many other bridges farther away from the epicentre sustained severe damage. For instance, only 8km north east of the Tirohanga site, the Waima River Bridge (shown in Figure 3), sustained moderate to severe damage.

5. DISCUSSION

The Kaikōura earthquake caused severe damage to infrastructure, and particularly the road and rail transportation networks. Notably large segments of the South Island's main highway, State Highway 1 and KiwiRail's Main North Line were damaged and had to be closed between Picton and North Canterbury. Closure of SH1 increased travel time between Christchurch and Picton from around 3.5 hours to more than 8 hours. The vulnerability of the state highway network to different and cascading hazards presented a substantial risk throughout the emergency operation, and during the recovery phase. The earthquake highlighted the need for major strengthening and engineering structures along critical transport routes. Therefore, it is very important to learn from this event as well as from similar events experienced around the globe.

Despite severe damage to roads and some bridges, the majority of bridges performed well during the earthquake, including the subject bridges of this paper, which only sustained minor damage. The abutment wall at Tirohanga Stream Bridge moved marginally and no ground damage was observed. However, the wider area encompassing the bridge site settled by about 2 – 3m due to ground displacement across the fault surface. Although a seismic assessment carried out prior to the Kaikōura earthquake indicated that the bridge would meet the performance requirements set out by the Bridge Manual, when subjected to higher levels of ground shaking the level of predicted damage did not materialise.

Meanwhile, closer to the earthquake epicentre, Stinking Stream Bridge on SH7, located 130km south west of Tirohanga site, sustained more severe damage. Although Stinking Stream Bridge was closer to the epicentre, it experienced lower levels of ground shaking than the Tirohanga site. The nearest strong motion station to this site, Waiau Gorge Station (WIGC) recorded horizontal and vertical peak ground accelerations of 0.73g and 0.56g, respectively. But the interesting feature of this bridge was that not only was the geological setting similar to Tirohanga, but also the structural form was broadly comparable. However, on this site, due to liquefaction and lateral spreading effects, the approach embankment settled and displacements of the wing walls caused major cracking.

Like Tirohanga, Kekerengu Bridge also sustained much less damage than predicted, noting that greater damage was predicted than for Tirohanga Bridge. The inspection of the bridge site after the Kaikōura earthquake found only minor cracks in the concrete, most of which were deemed to be pre-existing. Although the bridge sustained minor damage, there were other bridges with similar structural configurations and geological settings. For example, farther from the epicentre, Flaxbourne River Bridge and Needles Creek Bridge located approximately 25km north east of Kekerengu Bridge on SH1 suffered moderate to severe damage. These two bridges suffered more damage, the deck of the Flaxbourne Bridge separated from its piers and abutments and plastic hinges were observed at both abutments. At Needles Creek Bridge liquefaction ejecta was observed with lateral spreading cracks near to the piers. Both abutment walls settled 150mm and plastic hinges developed at the top and bottom of the piers (GEER 2017). Considering that the level of shaking was similar to Kekerengu Bridge, it highlights there are many uncertain factors in ground behaviour which makes the assessment of existing bridges difficult.

The uncertainty in the characterization of site, soil and structure properties makes the assessment of existing structures challenging, especially in the absence of specific ground investigation information and as-built drawings. There is a risk, either consciously or subconsciously, of combining conservatism at different levels of site classification, soil stiffness and strength assessments, which can lead to an over-conservative outcome for the assessment of existing structures, especially considering the cost and traffic disruption if significant retrofitting is undertaken.

In summary there are a number of lessons learned from comparison of seismic assessment of the subject bridges with the observed performance after the Kaikōura earthquake:

- In general the seismic assessments carried out prior to the earthquake indicated none of the bridges required strengthening in order to meet the performance requirement set out by the Bridge Manual. This assessment was validated by the earthquake as the inspections showed minor damage within the structure and surrounding soil, even though the shaking experienced during the Kaikōura earthquake was larger than the targets adopted for the seismic assessment. Nevertheless, both Kekerengu and Tirohanga bridges performed much better than anticipated in terms of the extent of repairable damage.
- No obvious soil degradation, e.g. liquefaction, was observed at any of the two bridge sites, whereas other bridges, located in similar geological settings, suffered severe damage as the local soil deposits liquefied. Liquefaction susceptibility across a typical bridge site is extremely difficult to predict reliably, even with some site specific ground investigation information.
- Geotechnical engineering has to consider natural geological materials that are potentially variable both in terms of their properties and their spatial distribution. But combining conservativism in assessing strength and stiffness properties of soil, with soil class, can result in an overly conservative evaluation of the soil's performance. Conversely, it can also lead to under-estimation of structural loads.
- Whether consciously or subconsciously, there is likely to be an inherent conservatism in geotechnical assessment where there is a lack of any site specific ground investigation data, which will result in predicting more damage in the structure. Checking the sensitivity of the structure to stiffer and stronger soil properties could reduce the predicted damage quite considerably - matching that observed.
- Palermo et al. (2017) inspected many bridges after Kaikōura event and they concluded that in general wall pier bridges showed much less damage than other forms of pier structure as this form of construction has higher structural redundancy. Both Kekerengu River and Tirohanga Stream Bridges have wall-shaped piers.
- In the seismic assessment of both bridges, a pushover analysis, which is a pseudo-static structural method was combined with soil stiffness properties derived from CIRIA R103. The latter is based on pile static testing appropriate for long term assessments and not transient seismic loads. Moreover, static methods ignore the hysteretic damping inherent in dynamic soil response which reduces the estimated damage in the numerical model. In addition, the former ignores the additional damping gained as a result of yielding reinforcement and cracking concrete elements as would be captured in a time history analysis.
- "Local" effects, such as fault rupture, have potential to be more significant than seismic soil behaviour. Generally, the predicted performance of Tirohanga Bridge matched the observations better than Kekerengu Bridge, which sustained comparatively less damage. On the other hand Tirohanga Bridge had to be rebuilt because of the increased flood risk due to settlement of the entire site by 3m. This is an important factor to consider by asset owners - as well as undertaking accurate assessment evaluations, the holistic picture of the structure within a wide geological setting should not be ignored. In this case the subsequent flooding events resulted in rebuilding a bridge which was only damaged minimally.
- Although the predicted thickness, stiffness and strength of the soil strata, when combined with the pseudo-static assessment, were meant to evaluate the 'best estimate' of the seismic response of the Tirohanga and Kekerengu Bridges, the Kaikōura

earthquake highlighted that the accuracy of this evaluation is overwhelmed by the potential variability in the inputs. Nevertheless, the prediction assessment does provide invaluable insights into the performance of the structure. Despite the shortcomings, the best outcome of such seismic assessment is enhancing one's engineering judgement.

- It is imperative to have the wisdom of an experienced engineer fully involved in the modelling process, so the developing engineer's modelling skills can be 'truthed' against the experienced engineer's experience.
- All bridges influenced by fault ruptures sustained some level of damage. Given a further and possibly stronger earthquake, it is even harder to reliably predict the effects of fault rupture when undertaking future seismic assessments. Therefore, definition of damage control limit state and collapse prevention limit state needs further research in this regard.

In summary, geotechnical engineers, compared to other fields of engineering, work with materials and conditions that are difficult to characterise. Unlike many other engineering disciplines, they cannot simply specify the types of materials or geometries with which they would like to work. The development of the geotechnical input models are strongly dependent on the available data, the experience of the individual engineer and precedent of the profession as a whole. Hence the study of case histories in general, and of failed or collapsed structures in major events in particular, provide valuable insight for future designs if the observed performance is documented systematically.

6. ACKNOWLEDGEMENT

The authors would like to acknowledge NZTA and NCTIR for approval to showcase these two case studies and present the findings included in this paper. The support and collaboration of colleagues Nick Tunnicliffe, David Rowland, Tristan Hanbury-Webber and Alex Carding has been much appreciated. Moreover, the first author is grateful for assistance provided by Glenn Steetskamp and his colleagues at NCTIR. Financial support was provided by the New Zealand Geotechnical Society and Earthquake Commission to attend the 12YGP conference.

7. REFERENCES

Andrews, D. (2018). Detailed Design report – geotechnical "Road", NCTIR, 200179-DD-GT-RP-0002

Carding, A., Stewart, N. (2016). Region 11 SH1 Kekerengu River Bridge detailed seismic assessment, Beca report No. NZ1-12832118-9, 28 July 2016

Elson, W. K. (1984). Design of laterally-loaded piles, CIRIA report 103

Geotechnical Extreme Events Reconnaissance (2017). Geotechnical reconnaissance of the 2016 Mw 7.8 Kaikōura, New Zealand earthquake, 10 June 2017

MacDonald, I., (2018). Inspection summary report SH1 – Kekerengu River Bridge, NCTIR document No. 200163-DD-BR-RP-0008, Rev. A.

New Zealand Transport Agency, Bridge Manual SP/M/022, 3rd Ed.

New Zealand Society for Earthquake Engineering (2017). 2016 Kaikōura EQ special issue Vol. 50(2), June

New Zealand Standard (2004). Structural design actions Part 5: Earthquake actions – New Zealand, NZS 1170.5:2004

Palermo, A., Liu, R., Rais, A., McHaffie, B., Andisheh, K., Pamapanin, S., Gentile, R., Nuzzo, I., Granerio, M., Loporcaro, G., McGann, C., Wotherspoon, L. (2017). Performance of road bridges during the 14 November 2016 Kaikōura earthquake, Bulletin of New Zealand Society for earthquake engineering Vol. 50(2), June

Seisfinder website retrieved on 11 June 2018, https://quakecoresoft.canterbury.ac.nz/seisfinder/

Stewart, N. (2016). Region 11 SH1 Tirohanga Stream Bridge detailed seismic assessment, Beca report No. NZ1-12732064-9, 15 July 2016

The Design of Ground Anchors and Soil Nails in the Repair of State Highway 1

State Highway 1 between Christchurch and Picton was significantly damaged by the magnitude 7.8 Kaikōura Earthquake in November 2016. The North Canterbury Transport Infrastructure Recovery (NCTIR) is an alliance partnership, set-up to restore the road and rail networks in this area. Several projects undertaken by NCTIR have utilised ground anchors or soil nails in the repair or construction of structural assets, however the design used varied from site to site. This paper will discuss some types of ground anchors and soil nails. It will illustrate how they were used on this project, how corrosion protection was achieved and the design standards which were adopted.

1. INTRODUCTION

1.1 North Canterbury Transport Infrastructure Recovery (NCTIR)

The damage caused to the road and rail transport infrastructure along the north east coast of the South Island by the 2016 Kaikōura Earthquake was unprecedented in New Zealand. North Canterbury Transport Infrastructure Recovery (NCTIR) is an alliance partnership between the NZ Transport Agency (NZTA), KiwiRail, Fulton Hogan, Downer, HEB Construction and Higgins that was set-up to restore the road and rail networks. More than 1300 people from 100 organisations are part of NCTIR, working to restore the transport corridor along the Kaikōura coastline.

1.2 Typical Applications in NCTIR Structures

Ground anchors and soil nails have been utilised by NCTIR in the design of retaining walls, rock fall mitigation, tunnel repairs and various other uses on and around State Highway 1 and the MNL Railway. This paper will focus predominantly on the use of ground anchors and soil nails for retaining structures supporting State Highway 1 where such uses have included:

- Installing anchors through damaged existing gabion basket walls and kingpost walls to improve stability and/or wall capacity.
- Incorporating anchors into the design of new king post and bored pile walls in order to reduce the embedment required and limit outward displacement under future seismic loading.
- New gabion walls incorporating ground anchors rather than MSE reinforcing in order to minimise the excavation needed during construction.
- Soil nail walls designed to stabilise existing soil slopes and crib walls
- Stabilising steep slopes below new gravity walls with soil nails

2. DESIGN GUIDELINES

2.1 Standards and Codes

For the NCTIR project the design of assets supporting the state highway has been completed in accordance with NZTA Bridge Manual 3rd Edition, 2016 (Bridge Manual). In addition to this, a Design Philosophy Report for Retaining Structures was prepared by NCTIR and approved by NZTA. Between these two documents, the following list of references specific to the design of ground anchors and soil nails was developed:

- NZTA Bridge Manual 3rd Edition, 2016, Part 6 Site stability, foundations, earthworks and retaining walls
- NCTIR Design Philosophy Report for Retaining Structures on the Existing Road Network 100001-CD-RW-DP-0001[1]
- BS8081:2015 Code of practice for grouted anchors, The British Standards Institution 2017
- BS EN 1537:2013 Execution of special geotechnical works – Ground anchors
- FHWA-IF-99-015 Geotechnical Engineering Circular No. 4 - Ground Anchors and Anchored Systems - June 1999

Louise Kendal-Riches

Louise Kendal-Riches is a Chartered Professional Geotechnical Engineer. She moved to New Zealand to work for Aurecon in 2011.

Since then, Louise has been on secondments to various organisations undertaking earthquake recovery work including demolitions, repairs and rebuilds. Most recently Louise joined NCTIR as the Non-coastal Structures Lead.

- FHWA 96-069r Manual for Design and Construction of Soil Nail Walls (which has since been superseded by FHWA-IF-03-017 Geotechnical Engineering Circular No. 7 – Soil Nail Walls)

In addition to the above documents, NCTIR made two applications to the NZ Transport Agency to depart from the Bridge Manual during the design of ground anchors and soil nails. In both cases these were a departure from the approach to corrosion protection required by the Bridge Manual.

2.2 Ground Anchors or Soil Nails

As defined in Table 6.4 of the Bridge Manual, soil-nailed walls are categorised as a mechanically stabilised earth (MSE) wall. Soil-nailed walls comprise concrete and steel insertions knitting together a whole block of soil to form a mass which acts under gravity to remain stable, whereas anchored walls, as defined in the Bridge Manual, are designed to transfer some of the loads on walls to the ground outside the zone of influence of the wall. The NCTIR project used either soil nails or ground anchors at over 30 sites.

2.3 Key Guidance from the Bridge Manual for Ground Anchor Design

The following points summarise the key guidance from the Bridge Manual for ground anchor design:

- Walls that are restrained using anchors are designed to transfer some of the loads on walls to the ground outside the zone of influence of the wall.
- Anchored walls are generally rigid systems, and shall be designed to resist the full ground, groundwater and earthquakes forces on the walls.
- The anchor system shall be designed to ensure a ductile failure of the wall under earthquake overloads.
- Pull-out tests shall be carried out on trial anchors.
- On-site suitability tests shall be carried out accordance with BS EN 1537.
- On-site acceptance tests shall be carried out in accordance with BS EN 1537on all as built ground anchors.
- Long term monitoring and instrumentation should be carried out in accordance with FHWA-RD-97-130 Design manual for permanent ground anchor walls.

2.4 Key Guidance from the Bridge Manual for Soil Nail Design

The following points summarise the key guidance from the Bridge Manual for soil nail design:

- Soil nailing shall be carried out only on drained slopes that are free of groundwater
- Limited controlled block displacement may be allowed in strong earthquakes
- Pull-out tests shall be carried out
- On-site suitability tests shall be carried out on a selected number of production soil nails as per BS EN 1537(49)
- On-site acceptance tests shall be carried out in accordance with BS EN 1537(49) on at least 25% of all as built soil nails.

2. 5 Corrosion Protection for Ground Anchors

The Bridge Manual states that all anchor systems shall be corrosion protected to ensure durability over the design working life of the structure. Two classes of protection are specified. Table 6.5 and Figure 6.5 in the Bridge Manual provide a decision making process based on the aggressively of soil, consequences of failure and the cost benefit. Class I includes double corrosion protection by encapsulation of the tendon or bar pre-grouted under factory conditions inside a corrugated plastic sheath. Class II has single corrosion protection such as galvanizing or a fusion bonded epoxy-coating.

2.6 Corrosion Protection for Soil Nails

The Bridge Manual states that soil nail reinforcement shall be subject to the same corrosion protection requirements as ground anchors.

3. GROUND ANCHOR AND SOIL NAIL SYSTEMS

3.1 Systems considered at NCTIR

Both ground anchors and soli nails can be considered as a series of structural elements (rods) protruding into the ground. The rods can have various different forms depending on the system adopted. During the design process, various types of ground anchor or soil nailing systems were considered by the NCTIR designers. These included solid bars, hollow core self-drilling bars, fibre reinforced plastic (FRP) bars or direct push anchors. The following sections provide a general description of each of these ground anchor types.

3.2 Solid Bars

Solid bar anchors or nails typically comprise steel bars installed in a drill hole and grouted. These bars can be encapsulated in a grout core which is cast within a plastic sheath to provide Class I corrosion protection. In the case of construction in weak ground where drill holes are at risk of collapse during installation, temporary casing can be provided to keep the drill holes open.

3.3 Hollow Core Self-drilling Bars

Self-drilling anchors do not require a separate process to form the drill hole. A sacrificial drilling head at the tip of the anchor rods forms and governs the diameter of the anchor hole. Grout is pumped through the hollow core of the rods which eventually form the steel reinforcement component of the anchor. If necessary the grout can also be used as a drilling fluid during installation. Self-drilling anchors cannot be encapsulated in a plastic sheath to provide double corrosion protection.

3.4 FRP Bars

Fibre reinforced plastic (FRP) bars are solid and are installed using the same methods as described above for solid steel bars. A key advantage over a steel anchor is that they are significantly lighter. A lighter bar is much safer to handle on site. The other key advantage of FRP bars is that they have better corrosion resistance characteristics when compared to steel and can achieve a 100 year design life without encapsulation or additional treatment. The disadvantage is that the failure mechanism is brittle rather than a ductile plastic failure.

3.5 Direct Push Anchors

Direct push anchors typically have a flat head on the end of a series of steel rods which is rotated at the end of installation to provide a mechanical anchor. There is no grout component to these anchors and they rely on loading of the soil at the end plate to achieve their capacity. Because the anchors are pushed, rather than drilled, they are not suitable for installation in rock. These also cannot be encapsulated in a plastic sheath to provide double corrosion protection, although the specification of more corrosion resistant materials can improve the durability of these anchors. Of the systems presented here, only these direct push anchors are only suitable as ground anchors and not also suitable as soil nails. This is due to the load transfer mechanism.

4. SIGNIFICANT NCTIR GROUND ANCHOR AND SOIL NAIL SITES

4.1 The Sand Pit

4.1.1 Location and Damage

The NCTIR Sand Pit site is located on State Highway 1, just south of the Clarence River. The earthquake damage at this site consists of both fill embankment and cut slope failures along a 700 m long section of 2-lane highway. In the worst damaged area, the embankment failure destroyed the entire width of the south bound lane, as can be seen in Photograph 1. The ground conditions in this location are predominantly dune sand and the main challenges to the project were the confined nature of the site and the short time frame available for design and construction before reopening the highway. A temporary solution, comprising anchored gabion baskets, was designed and constructed to work around these constraints.

Photograph 1: Embankment failure at the Sand Pit, photograph taken January 2017.

4.1.2 The Sand Pit Wall Design

4.1.2.1 Optioneering

The design of this wall relied on the stability of the slope below the wall and analysis showed that the static slope stability factor of safety specified in the Bridge Manual was not easily achieved. It was recognised that design solutions which addressed this instability, in full compliance with the Bridge Manual, may not be achievable with in the NCTIR timeframe. However, to facilitate opening the road by December 2017, a temporary solution could be constructed. This design would have a design life of up to 5 years. In parallel, feasibility and pricing studies for a more long term solution to realign SH1 were carried out. The selected solution presents a lower cost, temporary option with a design life of up to 5 years comprising anchored gabion walls on the outside edge of the road.

4.1.2.2 Solution Overview

The advantage of an anchored solution at the Sand Pit was that it minimised the need to excavate to install a gravity or MSE solution as this would not be possible without closing the adjacent haul road which was essential to enable other critical repair works to the south. The maximum wall height was 6.0 m, and ground anchors were installed through the new gabion baskets.

The ground conditions were of Quaternary Dune Sand. Where ground anchors were installed, this was typically encountered as Sand with minor Silt or trace Gravel increasing in density with depth. Pre-construction anchor tests demonstrated that the ground was typically unstable during drilling and collapsed during open hole drilling before an anchor could be inserted and grouted. To overcome this construction issue, self-drilling (hollow core) anchors installed with a grout flush were specified. As discussed previously, it is not possible for this type of ground anchor to meet the requirements of the Bridge Manual for permanent anchor corrosion protection. A departure from the Bridge Manual was secured because these works were part of a temporary structure with a 2-5 year design life. A departure for reduced design seismic loading for this asset was also obtained because of the short design life.

During design of the Sand Pit ground anchors, a geotechnical ultimate grout to ground bond stress of 90 kPa was adopted. This bond strength was based on sacrificial anchor testing comprising three test anchors each with a bond length of 6.0 m and drill hole diameter of 0.1 m. The anchors failed between 175 kN and 255 kN. Three sacrificial direct push anchors were also installed and tested. Each direct push anchor took just 3 to 4 minutes to install and be ready for testing. The ultimate loads achieved were 66 kN, 81 kN and 90 kN for three different sizes of anchor heads. The extension recorded when loading each anchor was between 280 and 380 mm.

4.1.3 Detailing

The main design details for the gabion basket and self-drilling anchor wall at the Sand Pit comprised:

- A backing plate installed on each anchor to transfer the load from the gabion to the anchor.
- A wedge shaped bearing plate sat in front of the backing plate to allow anchors to be installed at 15° below horizontal.
- The anchor head was wrapped in denso tape to protect it from corrosion.
- A debond sleeve was installed over the hollow core anchor to create the unbounded length. This comprised denso tape and a pvc sleeve.
- A PVC pipe was installed through the gabion basket and 200 mm into the fill behind the wall to protect the anchor in this section where is is most vulnerable to corrosion, water flow and abrasion.
- Anchor nuts were to be hand tightened, rather than having a lock off load applied.

Figure 1 provides further details of the Sand Pit Ground Anchors.

4.1.4 Testing

Testing of the Sand Pit ground anchors was completed in accordance with the recommendations that are published in FHWA-IF-99-015 Geotechnical Engineering Circular No. 4 - Ground Anchors and Anchored Systems. During construction 5% of the anchors were performance tested and the remaining 95% were loaded for proof testing. The results are summarised in Table 1.

Photograph 2 shows the Sand Pit retaining wall during November 2017 when construction of this structure was approximately 80% complete.

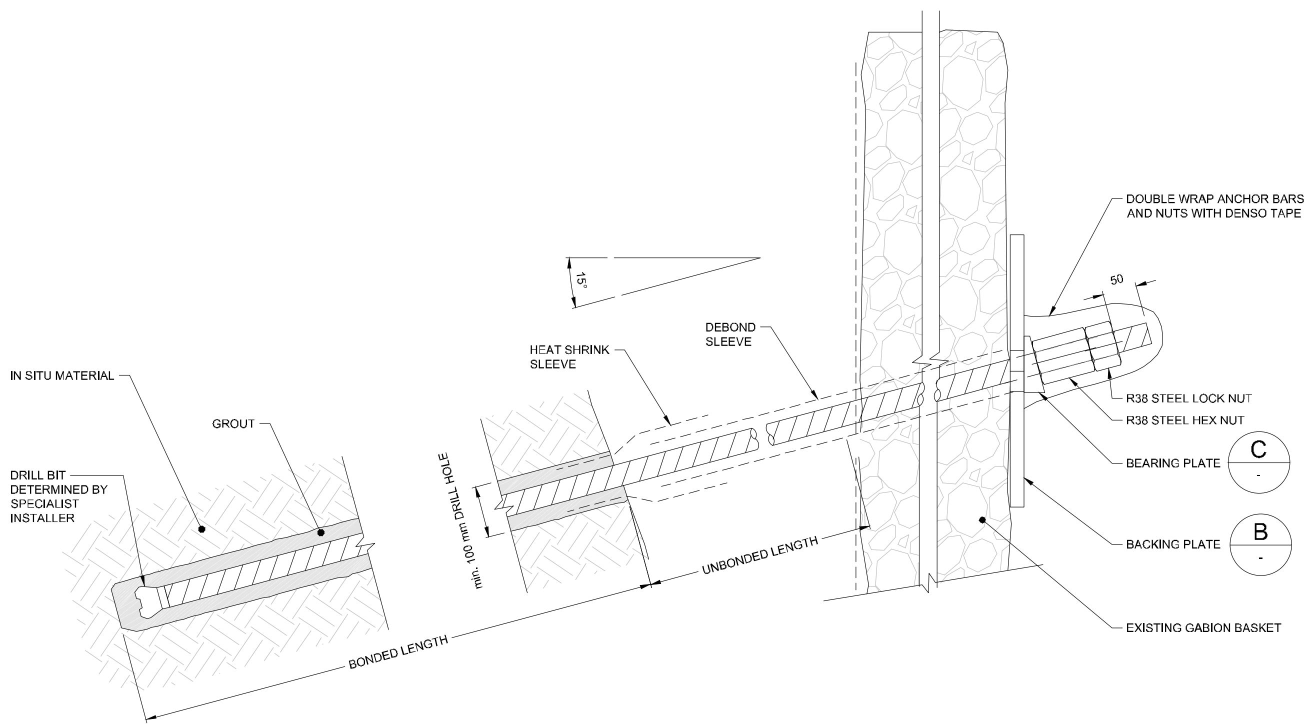


Figure 1: The Sand Pit ground anchor details.

Anchor Bond Length (m)	Design Test Load (kN)	Maximum Test Load (kN)	Minimum Calculated Bond Strength (kPa)	Displacement during 1-10 Minute Creep Test (mm)
5.0	65	86.5	55	0.12
7.0	110	146	66	0.11
8.0	110	146	58	0.3

Table 1: Summary of anchor test results at the Sand Pit

Photograph 2: Anchored gabion retaining wall at the Sand Pit during construction.

Photograph 3: Shoulder failure at the site of Wall 375.

4.2 Wall 375

4.2.1 Wall Location and Damage

Wall 375 is located in the Hundalees, approximately 30 km south of Kaikōura. Many of the NZTA retaining wall assets in this area were badly damaged during 2016 Kaikōura Earthquake. Some walls rotated and settled, while others completely collapsed. At this particular location a significant portion of the slope supporting the road shoulder failed which left the north bound lane constrained and the location subject to a 30 km/h speed restriction. Subsequent to the Kaikōura Earthquake, heavy rainfall and concentrated stormwater flow caused further damage. Ultimately the road geometric design at this location called for a raised road level and improved stormwater drainage. Each of these factors compounded to drive the need to construct a new retaining wall at this location. Photograph 3 shows the Wall 375 site a few weeks after the 2016 Kaikōura Earthquake.

4.2.2 Wall 375 Design

The NCTIR design for Wall 375 comprised a gabion basket wall atop a soil nailed slope. The geometry of the site meant that a soil nail slope alone could not recreate the new road height that was required for the geometric design and therefore additional gabion baskets were required to allow the shoulder to be built up. Analysis indicated that the steep slope below the proposed baskets did not provide sufficient bearing support and soil nails were designed and installed to provide a stable formation. The total height of both the gabion basket and soil nail structures is approximately 6.0 m.

The ground conditions through the Hundalees area typically comprise of Gravel fill underlain by Loess Colluvium over firm to extremely weak Mudstone of the Pahau Terrane. Sacrificial testing was undertaken in both Colluvium and Mudstone.

In Colluvium, with N60 of typically between 6 and 20, two tests were undertaken and both achieved a load of 400 kN. These tests had to be terminated before failure due to the test load reaching 80% of the bar capacity. Two further tests in Mudstone, with N60 > 50, failed at 300 and 325 kN. All of these tests had a bond length of 2 m and a drill hole diameter of 100 mm. Based on failure at 300 kN, a conservative ultimate grout ground bond strength of 480 kPa was adopted in design for both ground types in the Hundalees. A factor of 3 was applied to this value in design.

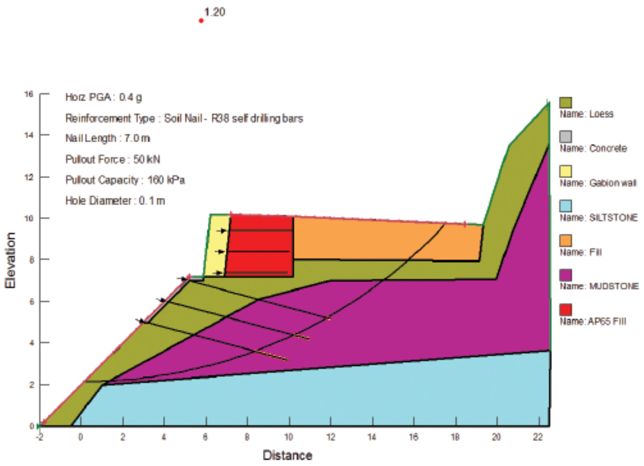


Figure 2: SlopeW output of the pseudo-static analysis of Wall 375.

The analysis of the gabion basket wall atop a soil nailed slope included a slope stability model analysed using SlopeW from GeoStudio 2016 under static and pseudo-static conditions. An extract from the analysis including a peak ground acceleration of 0.4g is shown in Figure 2.

Figure 3 provides a cross section and details of the structure which has been constructed at the site of Wall 375.

The construction sequence for Wall 375 was as follows:

1. Clear all vegetation and loose material from the slope
2. Install the soil nails inn a top down sequence and position the drainage strips and reinforcing mesh
3. Shotcrete the slope face followed by installation of the nail heads
4. Apply a final coat of shotcrete to cover the nail heads
5. Excavate for and construct the gabion basket wall section
6. Place backfill behind the gabion baskets
7. Construct road pavement, barriers, stormwater and other associated elements

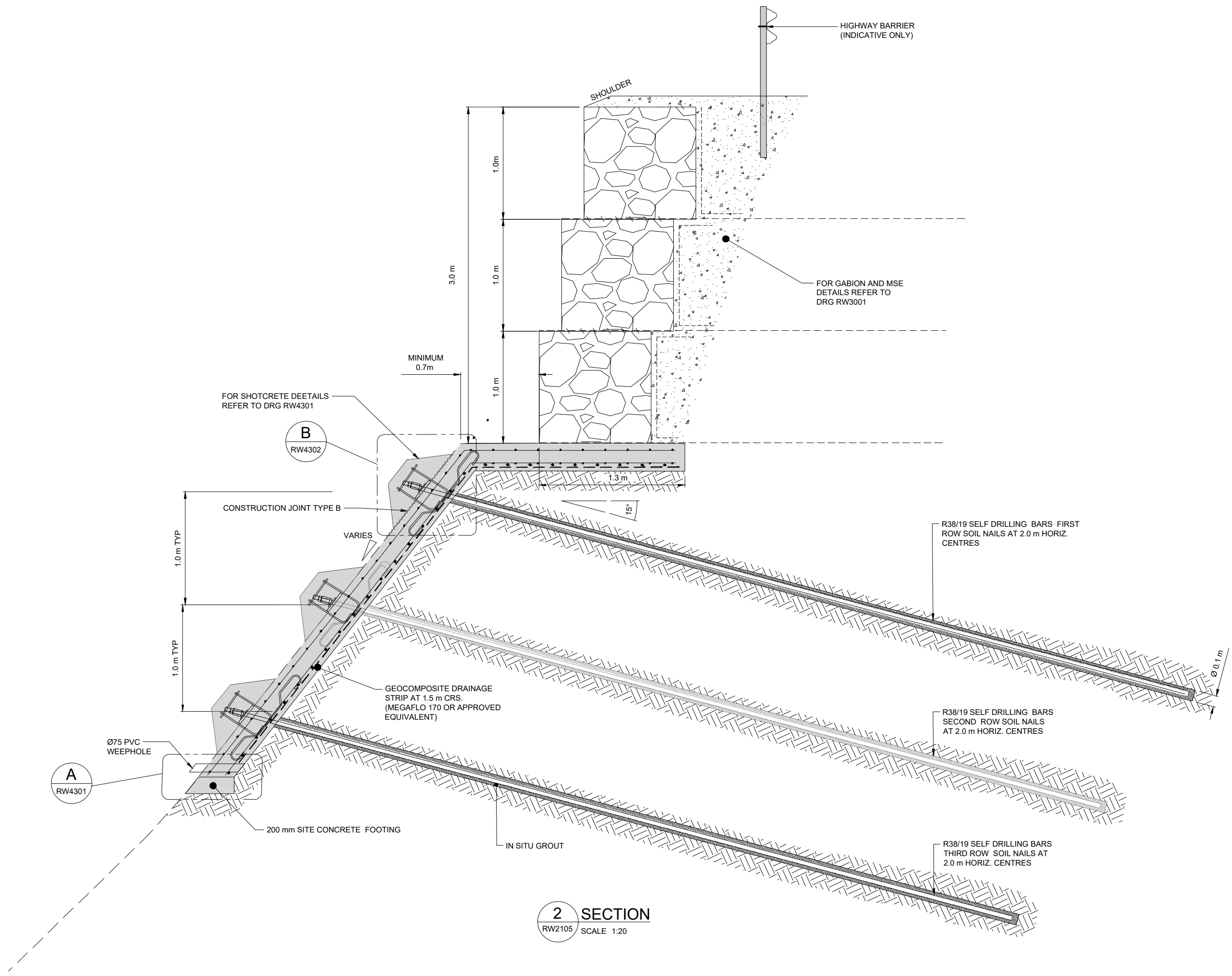


Figure 3: Typical cross section through Wall 375 and details.

4.2.3 Wall 375 Detailing

As shown in Figure 4, the design details for Wall 375 include:

- Three rows of 7 m long soil nails constructed using hollow core anchors on a diamond pattern spaced at 1.0 m vertically and 2.0 m horizontally.
- A 100 mm diameter grouted drill hole. A small amount of Shotcrete was also used to completely fill the top of the drill hole and ensure complete coverage of the bar.
- A 250 mm minimum thickness of shotcrete facing reinforced with steel mesh. Such shotcrete was applied to the upper slope face and wrapped over the top of the slope to minimise water ingress (see Figure 3).
- Geocomposite drainage strips behind the shotcrete connect to pvc weepholes at 2.0 m centres to address the risk of excess pore water pressure developing behind the shotcrete facing.
- A soil nail head detail comprising horizontal and vertical bearing bars, a trimmer bar to form a mechanical connection between the reinforcing mesh in the Shotcrete face and the nail, L-bars to reinforce the final face Shotcrete, an anchor plate, a domed base plate and nuts to match the anchor bar.

4.2.4 Wall 375 Soil Nail Testing

Testing of the Wall 375 soil nails was completed in accordance with the recommendations published in FHWA-IF-03-017 Geotechnical Engineering Circular No. 7 – Soil Nail Walls. 10% of production nails in each row were to be subjected to proof testing with a design test load of 180 kN. This meant a total of 2 production nails were tested. They both satisfactorily supported the maximum test load of 270kN which is 150% of the design test load. This indicates that a bond strength of greater than 106 kPa was achieved. Photograph 4 shows the first row of soil nails for Wall 375 immediately after installation.

4.3 Wall 433

4.3.1 Wall 433 Location and Damage

Wall 433 is an example of an existing retaining wall in the Hundalees area which was damaged by the 2016 Kaikōura Earthquake. It comprised a steel kingpost wall with timber lagging. The steel posts generally settled and/or displaced out of alignment with some movement at the base of the retained height evident. Due to the rake of the wall, it is not clear if any outward rotation of the wall occurred as a result of the 2016 Kaikōura Earthquake. Photograph 5 shows the damaged wall.

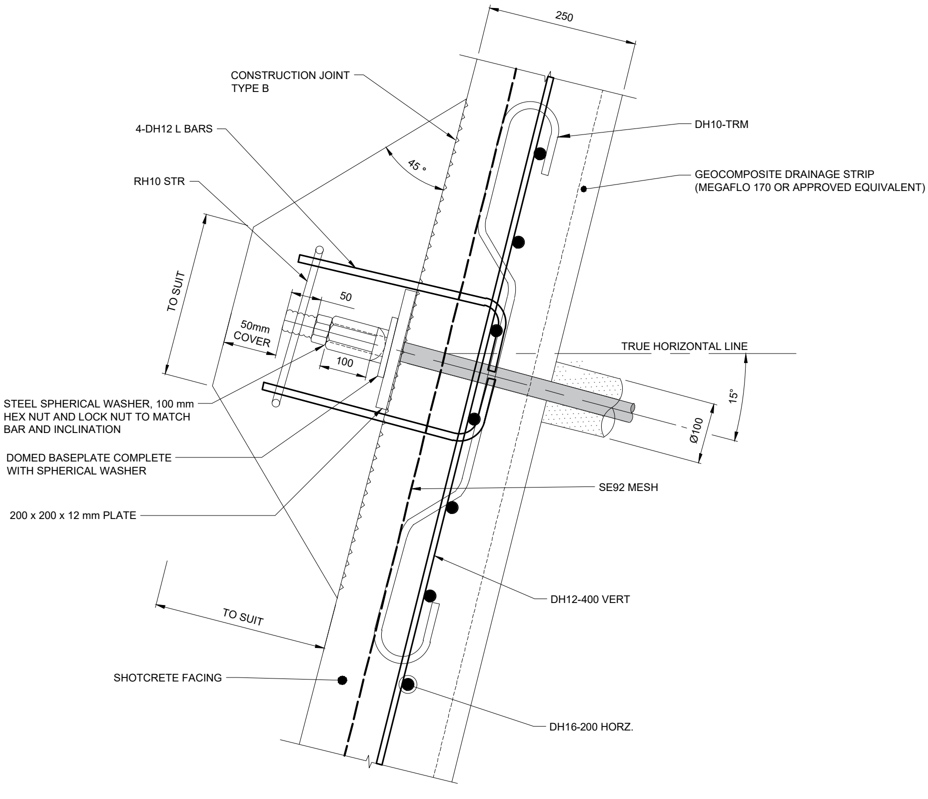


Figure 4: Soil nail head and shotcrete detail for Wall 375.

Photograph 4: Photograph taken May 2018 showing the first row of nails at Wall 375 immediately after installation.

Photograph 5: Damage to Wall 433 due to the 2016 Kaikōura Earthquake, photograph taken early 2017.

4.3 Wall 433 Design

To completely remove and rebuild Wall 433 would require significant excavation and back fill works to be undertaken. This would have been expensive and require a road closure during construction. It was therefore judged by the NCTIR Design Team that the optimum solution comprised a retrofit of the existing wall with ground anchors. Such ground anchors were designed to prevent any future rotation of the existing wall and improve the overall stability of the wall. In the design calculations, a minimal embedment of the kingposts was assumed due to the uncertainty around the as-built structure. The maximum retained height of the wall was 2.0m. Figure 5 shows an elevation of Wall 433 at the location of the retrofit ground anchors and walers.

Separate sacrificial anchor tests were not undertaken for this site because of the proximity to other sites where tests had already been undertaken in the Hundalees. The bond strength value presented in the previous section for Wall 375 was adopted for wall 433 as well.

For comparison, direct push anchors were installed and tested at the same sites as previously discussed. In colluvium the direct push anchor failed at 312 kN and 329 kN in Mudstone. Installation of the direct push anchor in Mudstone was possible by predrilling a pilot hole. While the direct push anchors had sufficient capacity, the anchor head dimensions needed to achieve this capacity were considered to be too large to easily install through the existing wall and therefore self-drilling anchors were specified here.

4.3.3 Wall 433 Detailing

The key components of the repair works design comprised:

- The single staggered row of hollow core ground anchors had a horizontal spacing of 1.5 m to match the exiting post spacing.
- Ground anchors had a bonded length of 3.0 m and an unbonded free length of 3.0 m.
- The ground anchors were staggered vertically to allow space for the whalers to overlap fully over each pair of kingposts.
- Whalers, comprising galvanised back to back PFCs, were designed to transfer the load from the existing posts to the ground anchors. The PFCs had stiffener plates added and backing plates provided at each anchor head.
- Custom packing wedges were provided between the posts and the whalers had to be used to ensure

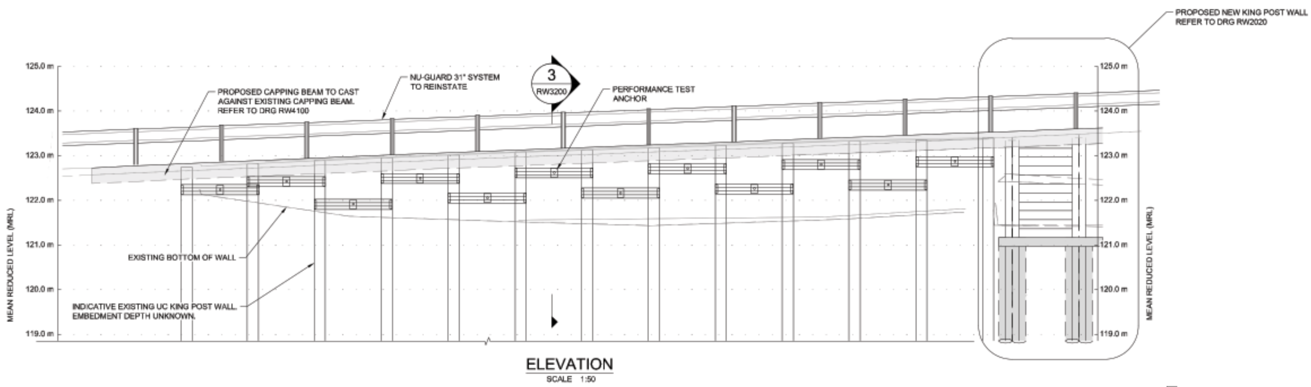


Figure 5: Extract from the construction drawings for Wall 433.

uniform load transfer due to the uneven post alignment. This is also why whalers were only ever designed to span across two posts.
- In addition to the backing plates, the anchor heads were also provided with a wedge bearing plate to accommodate the angle of the anchors.
- All anchor heads were wrapped in two layers of denso tape to protect them from corrosion.
- A debond sleeve was installed over the hollow core anchor to create an unbounded free length of 3.0 m. This debond section also comprised denso tape and a pvc sleeve. It extended through the whaler to provide protection to the otherwise exposed section of new ground anchor.
- Anchor nuts were to be hand tightened, rather than having a lock off load applied.

Figure 6 shows further detail of the ground anchor head detail that was constructed at Wall 433. Photograph 6 shows Wall 433 during construction of the NCTIR ground anchor retrofit works.

4.3.4 Wall 433 Ground Anchor Testing

Testing of the Wall 433 ground anchors was completed in accordance with the recommendations published in FHWA-IF-99-015 Geotechnical Engineering Circular No. 4 - Ground Anchors and Anchored Systems.
During construction one anchor of the 12 installed was performance tested and the remainder were proof tested. All ground anchors satisfactorily supported the maximum test load of 153 kN. This indicates that a bond strength of greater than 160 kPa was achieved. Creep testing recorded displacements of up to 0.65 mm between minute 1 and minute 10 of the test.

Photograph 6: Photograph of Wall 433 before anchor heads were installed, taken May 2018.

5. TESTING SUMMARY

Table 2 and Table 3 summarise the sacrificial testing discussed in this paper. Table 2 presents results undertaken on hollow core ground anchors, while Table 3 presents results from direct push anchor tests.

6. CONCLUSIONS

In each of the case studies presented in this paper, hollow core self-drilling type ground anchors or soil nails were constructed. These were found to be significantly faster to install than the alternative solid bar solution given the collapsing ground conditions encountered on all three of the sites. Hollow core anchors do not provide Class I corrosion protection as required in the Bridge Manual for permanent structures. In these particular situations, departures were granted by NZTA to allow the use of hollow core anchors. In each case the size of the hollow core bar specified was greater than required. This was in order to give the desired capacity at the end of 100 year design life assuming a loss of material through corrosion over time. In the disaster recovery environment, the advantages afforded by speed of construction was

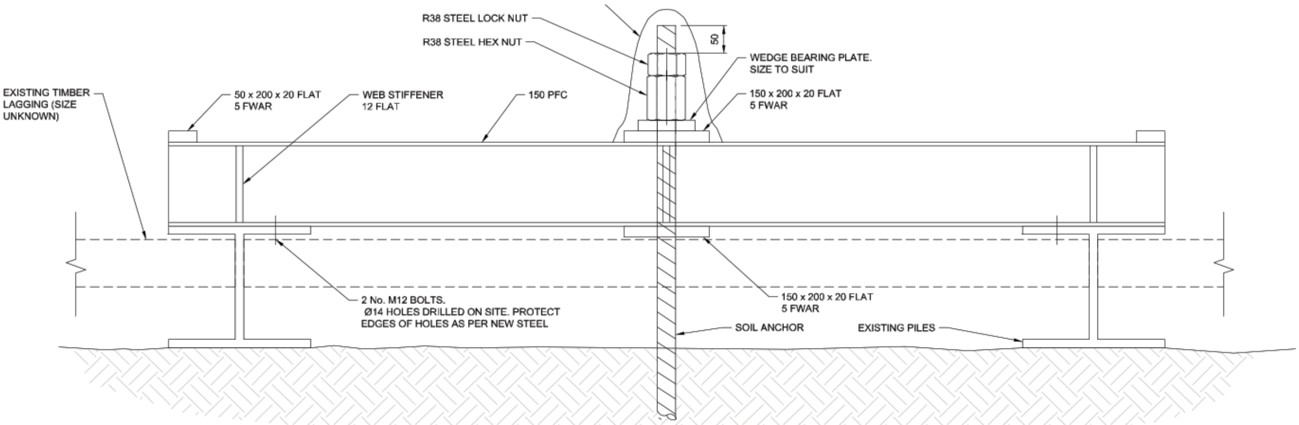


Figure 6: Plan of the whaler and anchor head details for Wall 433.

significant and hollow core bars proved much faster to install than solid bars in the conditions encountered.

Direct push anchors were considered and tested during design. In Loess colluvium and very weak Mudstone, the capacity of the anchor was similar to drilled anchors. However in sand the capacity was much lower. Direct push anchors were very fast to install and test, but significant deformation of the anchor has to occur in order to engage the soil to provide a reaction load.

One of the key debates throughout the design process was classifying whether retaining wall assets were to have soil nails or anchors. Construction, installation and detailing often looked similar, but the two systems differed in terms of assumed behaviour during the life of the asset, the design analysis and the testing requirements.

Much attention was given during the NCTIR project to detailing for corrosion protection. The most vulnerable section of the anchors and nails is assessed in many published references and by the NCTIR Design Team as being just behind the head. This area is more likely to be exposed to air and water attack, which in many cases was exacerbated by the presence of salt. This section of the anchors and nails may also be subject to changing loads or twisting as the wall facing and components could displace in the future. These issues were addressed in a number of ways as detailed in the case studies presented in this paper, including continuing the debond sleeve to the back of the nuts, wrapping vulnerable elements in denso tape, providing pvc sleeves and encapsulating soil nail heads in Shotcrete.

The testing regimes recommended by the FHWA for ground anchors and soil nails are very different. Typically all ground anchors are subject to some level of as-built testing, whereas only up to 25% of soil nails may be tested. This is assessed by the author as a reflection of the different levels of criticality of any given bar in each system.

NCTIR initially operated in an environment where much of State Highway 1 was closed. This meant that passing traffic volumes were low or absent and closing the road was relatively easy. Once the road reopened to the public immediately prior to Christmas 2017, it became much more difficult to form large excavations which impacted the road and enable the construction of gravity or MSE walls. In this restricted post December 2017 environment, installing ground anchors or soil nails was preferred over excavation.

A cost comparison of the two methods was undertaken and construction costs were found to be comparable. However, with due consideration of the whole life cost, anchored structures had potential to be more complex and therefore expensive.

In general, NCTIR were encouraged by NZTA to suggest and apply for departures from the Bridge Manual. It was recognised that careful consideration of each individual site and detailed analysis would lead to potential solutions, not in line with the Bridge Manual, where cost savings could be achieved. On this project several such cases were realised including case specific reductions in anchor corrosion protection.

7. ACKNOWLEDGEMENTS

I would like to acknowledge NZTA as the owners of the assets discussed in these case studies. In addition, the case studies presented were designed and constructed by NCTIR and involved a team of designers, constructors and others. In particular Alex Park, Sam Glue, Kiran Saligame and Andrew Awad were heavily involved in these designs.

Material Description	Grouted Hole Diameter (mm)	Anchor Free Length (m)	Anchor Bond Length (m)	Anchor Rod diameter (mm, external/ internal)	Measured Ground Anchor Displacement (mm) at				Ultimate Load at Failure (kN)	Recommended Ultimate Design Grout to Ground Bond Strength (kPa)
					50 kN	100 kN	150 kN	200 kN		
Loose to medium dense Sand with minor Silt	100	0.7	6.0	38/19	0.14	0.44	1.16	-	180	90
Medium dense Sand with minor Silt	100	0.7	6.0	38/19	0.1	0.27	1.5	1.6	255	130
Loose to medium dense Sand with minor Silt	100	0.7	6.0	38/19	0.22	0.46	0.63	-	175	90
Loess Colluvium with trace gravel	100	3.0	2.0	38/19	1.64	7.12	7.41	7.94	>400	475
Loess Colluvium with trace gravel	100	3.0	2.0	38/19	1.36	4.76	5.22	6.59	>400	475
Weak to very weak Mudstone	100	3.0	2.0	38/19	4.1	8.4	9.5	12.9	325	475
Weak to very weak Mudstone	100	3.0	2.0	38/19	3.54	7.15	10.6	12.21	300	475

Table 2: Anchor test results from testing of hollow core ground anchors

Material Description	Anchor Head Width (mm)	Anchor Bar Length (m)	Measured Ground Anchor Displacement (mm) at				Ultimate Load at Failure (kN)
			50 kN	100 kN	150 kN	200 kN	
Loose to medium dense Sand with minor Silt	206	6.0	400	-	-	-	66
Loose to medium dense Sand with minor Silt	259	6.0	310	-	-	-	83
Loose to medium dense Sand with minor Silt	335	6.0	300	-	-	-	91
Loess Colluvium with trace gravel	259	2.5	25	45	100	150	312
Weak Mudstone	206	2.5	20	40	125	100	329

Table 3: Anchor test results from testing of direct push ground anchors

Kaikōura Earthquake Repair and Recovery Programme – South Bay Marina and Coast Guard Station

Jodi Edwards

Jodi has worked as a Geotechnical and Civil Engineer in the Christchurch office of Tonkin & Taylor Christchurch since 2007. She was educated at the University of Canterbury and is currently part of the NCTIR team that is working to "Move Mountains and Reconnect Communities.

Tony Fairclough

Tony is a Christchurch based University of Auckland educated Geotechnical and Civil engineer with over 28 years' experience. He is the current Chair of the NZGS and has worked throughout NZ and the wider Asia-Pacific region including Malaysia (resident 4.5 years) and Hong Kong (resident 1.5 years).

Tony is currently employed by Tonkin & Taylor Ltd, who he joined in 2000, and was part of the NCTIR structures design team from January 2017 to June 2018.

INTRODUCTION

This paper describes the damage that was caused to the South Bay Marina and Coast Guard Station by the 2016 Kaikōura earthquake, and, the associated impact on the local community and economy. An overview of the "fast track" design and construction process that was undertaken to complete the marina earthquake repair program is also provided.

Kaikōura is a small semi-rural community with a district population of 3,552. The local economy has a strong reliance on the marine environment as fishing and marine based tourism are the prime drivers. Figures published by the Ministry of Business, Employment and Innovation (MBIE) indicate the pre-earthquake total annual tourism spend in Kaikōura was $120 million while discussions with local fishing operators indicated the value of the annual fishing catch was between $25 and 30 million per season.

Reliance of the Kaikōura economy on tourism and fishing was borne out by the following 2015 Statistics New Zealand data:

Employment Sector	Percentage of Kaikōura workforce	Percentage of New Zealand population working in sector
Accommodation and food services sector	26.1%	7.1%
Retail sector	14.3%	9.8%
Agriculture and forestry sectors	10.8%	10.8%
Fishing industry	1.6%	0.09%

Table 1: Summary of Kaikōura Employment by Sector

Marine based tourist activities are the main drawcard for tourists to Kaikōura. A study undertaken in 1998 indicated that the main motivations for visitors to Kaikōura were whale watching and dolphin swimming, with over 50% of overnight visitors participating in whale watching and over 30% in dolphin swimming.

South Bay is located on the southern side of the Kaikōura Peninsula, approximately 2.7 km due south of the Kaikōura Village centre. The South Bay Marina is an important part of the Kaikōura community and economy as it is the departure point for most of the local marine

[1] 2013 Census, Statistics New Zealand

[2] Summertime Visitors to Kaikōura: Characteristics, Attractions and Activities, Simmons, D.G et al. 1998. Lincoln University.

Figure 1: Location of the South Bay Marina and Coast Guard Station, Kaikōura.

tourism operations (including Whale Watch Kaikōura, Dolphin Encounters and Seal Swim) and is the base of operations for almost all of the commercial fishing fleet. This infrastructure is also crucial to the regional recreational boating community and is the site of the safest public launching ramp and the South Bay Coast Guard station.

Figure 1 shows the location of Kaikōura Village, the South Bay Marina and the South Bay Coast Guard Station.

The M_W7.8 Kaikōura Earthquake of 14 November 2016 resulted in an average of 0.93m of tectonic uplift in the South Bay area. This had serious consequences for the Kaikōura coastline and the associated marine industry. In particular; many new navigation hazards were created and the depth of water associated with the existing marina facilities was significantly reduced.

Photograph 1 illustrates the significant tectonic uplift and change to the coastal environment that occurred as a result of the 2016 Kaikōura earthquake.

In the days immediately following the Kaikōura earthquake, Government New Zealand quickly identified the importance of the South Bay Marina and Coast Guard Station to the wider Kaikōura community and economy. As such, steps were taken to expedite construction to reinstate the preearthquake level of service of these facilities. As an interim measure, while the North Canterbury Transportation Infrastructure Recovery (NCTIR) Alliance was being finalised, Government New Zealand appointed Environment Canterbury (ECan) to co-ordinate a stakeholder group and construction of slipway and launching ramp emergency repair works commenced immediately prior to Christmas 2016.

NCTIR took over the management, design co-ordination and delivery of the marina and coast guard station earthquake repair program during February 2018. This phase of the repair works was the subject of a meticulously planned construction plan and program to ensure the fastest-possible high-quality project delivery was achieved. The repaired marina and Coast Guard facilities were officially opened on 14 November 2017, exactly one year after the Kaikōura earthquake event. As described below, this was an incredible outcome that exceeded stakeholder expectations and demonstrates the value and advantages that an alliance type delivery structure can achieve.

Photograph 1: South Bay Marina and Coast Guard Station. This photograph was taken on 25 November 2016, within approximately 30 minutes of high tide.

GEOLOGY

The South Bay area is predominantly underlain by Amuri Limestone. This group is described on published geologic maps[3] as '*Hard, siliceous, micritic limestone locally interbedded with siltstone, marl, sandstone, chert or green sand. Includes intrusive and extrusive igneous rocks of the Grasseed Volcanics member with dikes, flows, sills, pillow lavas and agglomerate of peridotite, gabbro, dolerite and basalt,*' from the Muzzle Group approximately 55 million years old.

[3] Rattenbury, M.S.; Townsend, D.; Johnston, M.R. (compilers) 2006: Geology of the Kaikōura area: scale 1:250,000 geological map. Lower Hutt: GNS Science. Institute of Geological & Nuclear Sciences 1:250,000 geological map 13. 70 p. + 1 folded

Field observations at the South Bay Marina indicated that the bedding was the dominant defect in the rock mass at this site. The bedding layers were very thin and moderately inclined, dipping unfavourably towards the mooring basin, due to folding in historic seismic events (refer to Figure 4). The rock joints were very widely spaced and sub-vertically inclined towards the north-west.

EARTHQUAKE DAMAGE AND CONSEQUENTIAL EFFECTS

Somewhat surprisingly, considering the magnitude and duration of earthquake shaking experienced at the site, no significant structural damage was apparent to the Coast Guard and marina infrastructure after the magnitude 7.8 earthquake. However, the tectonic uplift of the South Bay area due to the earthquake created many new navigation hazards and the depth of water associated with the marina facilities was significantly reduced.

The reduced water depth adversely affected use of the marina approach channel, the mooring basin and all three of the local launching ramps: the Coast Guard slipway, the marina launching ramp and the Kaikōura Boating Club launching ramp (refer to Photograph 1).

Post-earthquake, most commercial vessels could only use the marina launching ramp during a 2 to 4 hour window over high tide and this placed severe restrictions on their operations. The limited tidal windows also had a brutal impact on tour operations, that were the subject of additional constraints such as DOC marine mammal permits or regulations on deployment times of fishing gear. The extremely narrow operational window prevented the ability to provide a good quality tourism experience at regular and convenient times during daylight hours. Preliminary estimates indicated that, prior to completion of the marina earthquake repair works, the tour operators could only provide between 10 and 20% of the pre-earthquake tour capacity.

After the Kaikōura earthquake the Coast Guard slipway was subject to water depth issues an hour either side of low tide, in particular during times of bad weather or heavy seas. In the low-tide situation the Coast Guard vessel had to be transported by road to the adjacent marina launching ramp resulting in a minimum 20 minute delay to emergency response.

As the nearest alternative Coast Guard stations are located in Picton to the north and in Lyttelton to the south, both at least 6 hours sailing time from Kaikōura in fair weather, it was imperative that the South Bay Coast Guard facilities were made fully operational as quickly as possible.

OVERVIEW OF THE PERMANENT REPAIR WORKS PROGRAM

The works required to reinstate the pre-earthquake functionality of the Coast Guard and South Bay Marina facilities predominantly comprised excavation of the seabed to correct and compensate for the tectonic uplift rather than the specific repair of structural damage to the harbour facilities. This included excavation of the approach channels and mooring basin.

The marina excavation and dredging works had a significant adverse impact on the existing seawall, jetty and mooring pile structures as it undermined their foundations. The launching ramps also needed to be extended as, post-earthquake, the bottom of the ramp was too high relative to the low to mid-tide sea level. In total, approximately 14,500 cubic metres of seabed material was excavated from the marina approach channel and mooring basin as part of the NCTIR earthquake repair program. Most of this excavated material, which generally a combination of

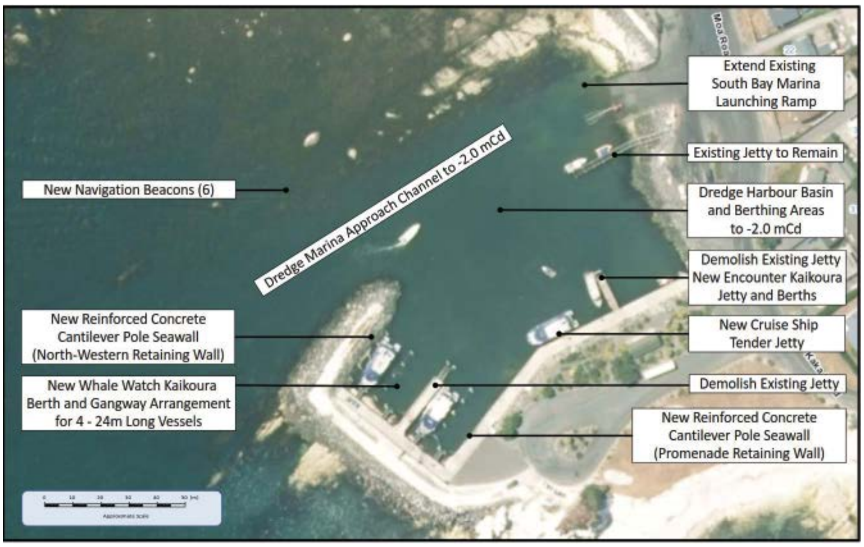


Figure 2: Overview of the works that were completed at the South Bay Marina site as part of the NCTIR program.

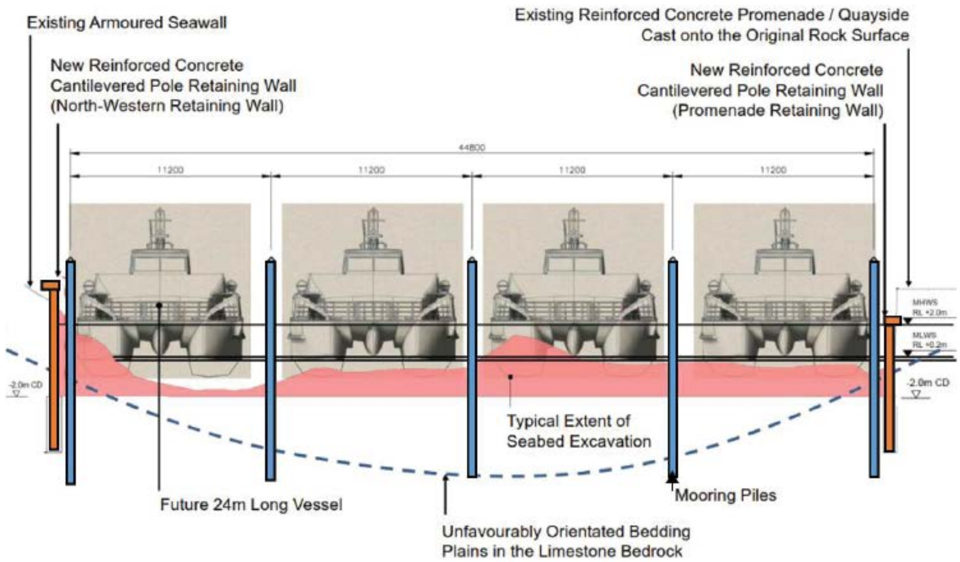


Figure 3: Typical cross-section through the Whale Watch Kaikōura berthing area.

silt, sand, gravel and cobble sized material derived from the limestone bedrock, was used to construct temporary working platforms prior to being placed in an approved, environmentally appropriate offshore disposal area.

Figure 2 provides an overview of the earthquake repair works program that was constructed under NCTIR.

Due to the presence of unfavourably orientated bedding plane defects (refer to Figure 3) and secondary joints in the basement rock, a risk was identified that blocks of rock or crushed zones could move or become unstable as a result of the seabed excavation works. This, in turn, had the potential to undermine and damage the existing breakwater and

Figure 4: Overview of the works that were completed at the South Bay Coast Guard Station site as part of the NCTIR program.

promenade structures. This risk was assessed to be highest during excavation construction and the Ultimate Limit State (ULS) seismic event scenarios. A combination of retaining walls, benching and reconfiguration of the marina layout was used in conjunction with a carefully designed staged construction sequence to mitigate these risks.

Some stakeholders used the earthquake repair program, and the presence of an unprecedented variety, size and number of construction resources in the Kaikōura region, as an opportunity to invest in upgrades and future proof their facilities. For example, Kaikōura District Council invested in a new tender jetty to allow cruise ship passengers to safely transfer from ship to shore, via tenders, in all tide conditions.

Whale Watch Kaikōura (WWK) also invested in the construction of a new vertical sea wall on the north-western side of their berthing area (refer to Figures 2 and 3). This enabled the length of vessel which could be accommodated in all of their berths to be increased from 18 m to 24 m. In conjunction with a staged replacement of their existing fleet, this enabled the total number of passengers that could be carried by WWK to be approximately doubled, in an environmentally sensitive manner.

Environment Canterbury invested in the installation of 6 new lighted navigation beacons. This significantly improved channel visibility and user safety at night and during times of poor weather conditions.

The type of repair work that was undertaken for the Coast Guard Station was similar to the adjacent marina site, but was significantly smaller in scale. In total, approximately 5,500 cubic meters of seabed material was excavated from the Coast Guard Station approach channel and turning basin as part of the NCTIR earthquake repair program.

Figure 4 above provides an overview of the earthquake repair works program that was completed by NCTIR at the South Bay Coast Guard station site.

OVERVIEW OF THE EMERGENCY REPAIR WORKS PROGRAM

Emergency repair works were undertaken by Downer Construction Ltd during December 2016 to improve the safety and functionality of the Coast Guard Station, marina and Kaikōura Boat Club (KBC) launching ramps over the 2016/2017 Christmas period. The scope of these works comprised:

- Partial excavation of the Coast Guard Channel.
- Placement of temporary hard fill at the end of the marina and KBC launching ramps (this enabled trailers to reverse further down the launching ramps), and,
- Excavation of a small basin in the seabed immediately in front of the marina launching ramp (this allowed a limited number of vessels to sit within the marina harbour while waiting to use the ramp at times of low tide).

During the 2016/2017 Christmas period the KBC allowed non-members with vessels that were less than 8m long to use their private launching facilities. This significantly reduced the demand and congestion at the marina launching ramp and reduced operation risks for the commercial and tourism vessels.

ENVIRONMENTAL MANAGEMENT

An environmental management plan (EMP) was prepared and submitted for the project during December 2016, prior to commencing the physical construction works. This EMP was developed in accordance with the Contractor's Environmental Policy and AS/NZS 4801:2001 –

"Occupational Health & Safety Management Systems".

The primary potential environmental risks which were identified at this stage of the project were:

- Discharge of seabed material to Kaikōura Harbour during channel excavations.
- Discharge of construction materials during reconstruction.
- Excessive disturbance to the coastal marine area.
- Spillage of hazardous substances.
- Construction noise and vibration.
- Fumes and smoke from machinery, and,
- Disturbance of archaeological remains.

All of the above issues had the potential to adversely impact the environment and wildlife, in particular:

- Penguins (which nested and moulted in the existing breakwater armour rock around the sites during November).
- Sea slugs (which were known to be present on the sea bed within the marina basin).
- Seals (which often rested on the breakwater rocks around the sites), and,
- Whales and dolphins (which were known to occasionally swim within a few hundred metres of the sites)

Several mitigation and control measures were identified and implemented to minimise or eliminate the potential adverse impact of the proposed works on the environment. In addition to the use of common techniques, such as the construction of bunds to control silt runoff, several less common measures were deployed such as:

- The University of Canterbury was engaged to relocate sea slugs and other fauna, as instructed by ECan, from the marina area in the week prior to commencing construction.
- Construction was programmed to minimise disturbance to Penguin nesting or moulting periods, and,
- NCTIR archaeologists assessed the footprint of the construction works and established exclusion zones which were clearly marked out on site. An On-Call Procedure (OCP) was established for notification of any suspected archaeological discoveries.

ENGINEERING DESIGN

The key elements of the earthquake repair design program comprised:

- Dredging and demolition works.
- Extension of the reinforced concrete Coast Guard

and marina launching ramps.
- Construction of two new reinforced concrete cantilevered pole retaining walls to support the vertical edges of the basin dredging works.
- Supply and installation of 34 new mooring or gangway support piles.
- Supply and installation of four new floating gangway systems.
- Supply and installation of Six navigation beacons, and,
- Construction of two new jetties

Figures 2 and 4 show the location of the above works.

The design of the width and depth of the approach channels and basins were discussed and agreed with the key stakeholders and checked to ensure compliance with Australian Standard AS3962:2001. The mooring piles, jetties, retaining walls and navigation beacons were all designed to comply with the appropriate New Zealand standards and Harbour Master (ECan) requirements.

A considerable amount of effort was expended to optimise the WWK mooring basin layout, and; the final design of the two retaining walls. As a result of stakeholder engagement, WWK made a significant financial contribution which enabled the construction of both retaining walls.

The following options were considered for the Promenade and North-western retaining walls:

a) No retaining walls (provide a batter around the entire basin excavation perimeter).
b) Shotcrete facing with rock anchors.
c) Reinforced concrete "L- wall".
d) Cast in-situ reinforced concrete cantilever walls, with or without tiebacks, and,
e) Pre-cast reinforced concrete pole walls with shotcrete, cast in-situ or precast panel lagging, and, with or without tiebacks.

Preliminary analysis, assessment and stakeholder engagement confirmed that the "no retaining wall" option was unacceptable as excavation was required immediately adjacent to the existing promenade or breakwater to enable the construction of a WWK berthing arrangement similar to the preearthquake situation.

Primarily due to the expected cost of the shotcrete and rock bolt solution, and potential maintenance and corrosion issues at the rock bolt heads, this option was judged to be technically and economically undesirable. Cost and constructability issues, in particular safety during construction and inadequate cut support during basin excavation, resulted in a cantilevered pole solution being strongly preferred over the L-wall options.

The cantilever pole wall solution had several key benefits over the other options, including cost, and could be constructed in such a way that it supported the existing promenade or breakwater structures before construction of the basin dredging works commenced. The use of precast poles was identified as the preferred design as it eliminated the durability and constructability concerns associated with the cast in-situ alternative. Discussions with the delivery team confirmed that this solution was likely to be the easiest and cheapest to construct. As such the "pre-cast reinforced concrete pole wall" option was collaboratively selected by the design and delivery team as the preferred retaining wall design.

Specialist finite element modelling (FEM) software was used to analyse and estimate the static and seismic soil and rock loads, deformations and structural demands which were associated with the precast reinforced concrete pole walls. At the location of the north-western retaining wall, the rock mass was observed to have bedding planes dipping at approximately 20° towards the basin excavation. Secondary rock Joints were also observed in the field to dip towards the north-west. Two failure block configurations were assessed to be the most critical kinematically-feasible failure mechanisms and were adopted as the 'worst case' design scenarios.

In total, three retaining wall height and embedment configurations were analysed and optimised during the detailed design process. Wall Type A had a maximum retained height of 3.0 m and an embedment of 3.0 m; Wall Type B had a retained height of 5.0 m and an embedment of 3.5 m; and, Wall Type C had a maximum retained height of 4.45 m and an embedment of 3.0 m. The apparent friction angle on the bedding plane was assessed to be 44° and a sensitivity analysis was also undertaken via a 2° reduction along the bedding plane.

The preferred wall layout was assessed to comprise 450 mm square piles at 1.3 m centres. The rock (and soil if present) between the poles was therefore assessed to be supported by arching effects between the poles.

In accordance with the requirements of NZS 3101-2006 Chapter 3, the walls were designated to be located within the tidal zone and were designed to have a minimum cover of 60 mm, a cement binder incorporating 8 % Amophous Silca (Microsilica, MS), a total binder of equal to or greater than 350 kg/m^3, and a water to binder ratio of less than 0.45. The minimum specified compressive strength was 50 MPa.

The final retaining wall design incorporated a cast in-situ reinforced concrete capping beam running across the top of all the wall piles. The original pre-cast pile design specified starter bars projecting from the top of the piles. This detail was found to result in the starter bars being vulnerable to damage during pitching and installation of the piles, in particular when working adjacent to the existing promenade structure. To address this constructability issue, the starter bar detail was amended to comprise a Reid bar with coupler cast into the pile. Once the pile was cast in place threaded rods, which were designed to act as the starter bars, were screwed into the coupler as part of the capping beam construction sequence. A significant improvement in the speed and quality of construction was immediately observed after this amendment had been made, which more than off-set the additional cost associated with the Reid threaded inserts and rods.

The pile starter bars extended across the pile/capping beam joint. This joint was identified as being potentially vulnerable to corrosion, in particular due to its vicinity to the tidal and/or splash zones. To improve the level of corrosion protection at the joint location, Sikadur 32 Normal was applied to the top of the precast concrete piles. This product allows for wet concrete to be poured against the precast concrete and effectively form a wet joint and provide full cover to the starter bars.

Finally, the Promenade retaining wall design utilised a cast in-situ reinforced concrete lagging. The Delivery Team requested that the lagging for the north-western retaining wall be amended to a pre-cast panel system to better support the breakwater armour rock during excavation. This change in lagging system resulted in a different set of construction challenges, in particular around lagging installation, backfill placement and aesthetics if the wall piles are not installed to an extremely tight line, level and spacing. In hindsight, the designers believe that the cast in-situ lagging system resulted in less construction challenges, and, a slightly better final finish.

CONSTRUCTION

It was initially proposed that a temporary bund be constructed across the marina harbour entrance and the basin pumped dry to enable completion of the launching ramp, retaining wall and jetty construction. This would have resulted in the marina facility being completely closed to the public and commercial use for a period of between 7 and 9 months.

The stakeholder engagement process identified that the best overall solution, which resulted in the lowest adverse reputational, commercial and financial impact across all stakeholders, was to maintain at least partial access to the launching, mooring and berthing facilities during the construction works. As a result, the construction methodology and program were redesigned so that launching facilities were available at the marina and at least one berth was available for WWK operations at all times except for a period of four to six weeks at a time when the north western retaining wall was being constructed. The revised construction program had a total duration of 10 to 11 months.

Photograph 2: South Bay Marina, March 2018. Construction of the temporary working platform is complete and excavation to deepen the marina approach channel is well advanced.

In summary, the revised construction sequence and methodology comprised:

1.) Progressively excavate the Coast Guard station turning basin and approach channel.
2.) Progressively construct a temporary working platform adjacent to the Coast Guard approach channel using material excavated during Task 1 above.
3.) When the Coast Guard channel excavations were complete, remove the temporary working platform and relocate the excavated material to construct a temporary working platform adjacent to the marina approach channel (see Photograph 2 below).
4.) Complete all "land based" excavations for the marina approach channel. Use excavated material to improve the marina channel working platform, or, commence construction of a temporary access road and working platform for the Dolphin Encounter Jetty.

Photograph 3: View showing the south-eastern temporary working platform. Note the barge-mounted excavator visible in the centre-top portion of this photograph.

Photograph 4: Cruise Ship Tender Jetty construction. All construction for this and the Dolphin Encounter Jetty was completed within the temporary working platform.

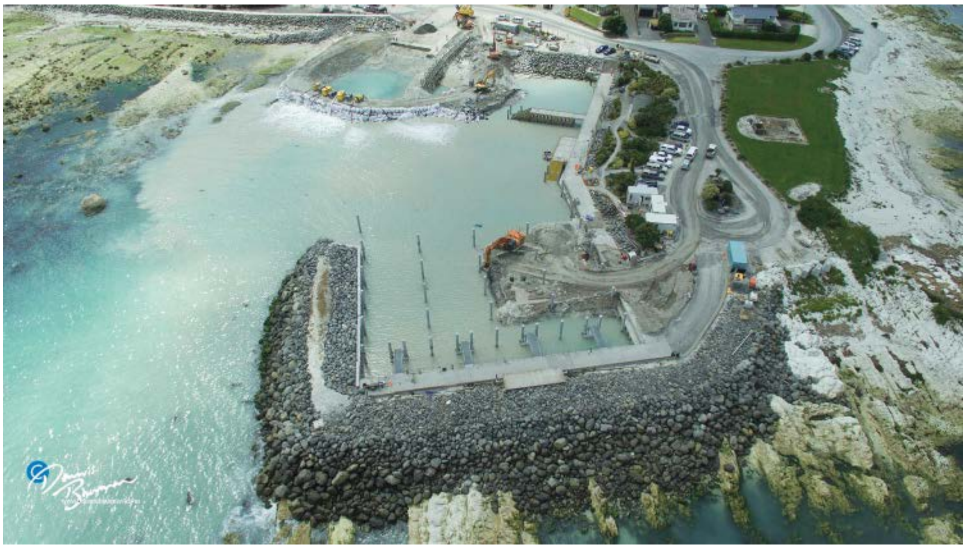
Photograph 5: Promenade retaining wall construction. This wall design comprised precast reinforced concrete poles and cast in-situ infill panels. Its construction was completed within the temporary working platform.

5.) When the marina approach channel "land-based" excavations were complete, remove the temporary working platform and relocate such material to complete the temporary working platform for the Dolphin Encounter Jetty, the Cruise Ship Tender Jetty and the Promenade Retaining wall (see Photograph 3).
6.) Undertake additional "ship-based" dredging works for the approach channel and harbour basin using a barge-mounted excavator (see Photograph 3). Barge-mounted equipment was used for all areas where land-based excavators could not safely reach without the need to close the approach channel or launching ramp.
7.) Construction of temporary works platform from limestone material that was recycled from the channel deepening works. This enabled safe construction of the promenade retaining wall and Cruise Ship Tender Jetty. (Photo 3)
8.) Commence demolition and construction of the Dolphin Encounter Jetty and construction of the cruise ship jetty and promenade retaining wall structures (see Photographs 4 and 5). The pile construction for these structures was completed from the temporary working platform surface, with careful staging and shoring of future excavations to ensure the highest possible level of geotechnical stability and worker safety was maintained at all times.
9.) At a time that suited WWK operations, extend the temporary working platform within the marina across the full width of the WWK berthing area to enable construction of the north-western retaining wall to commence. During this phase of the construction WWK adopted an alternative, less efficient methodology to embark and disembark their passengers which made use of the launching ramp or the adjacent existing jetty.
10.) Commence and complete construction of the north-western retaining wall as quickly as possible.
11.) Install WWK berthing area mooring and gangway piles.
12.) Install WWK floating gangway system.
13.) Excavate and remove the temporary working platform for the north-western retaining wall, WWK berthing area and Dolphin Encounter Jetty. Simultaneously complete the final seabed excavation and dredging works in these areas. Transport spoil material from these temporary working platforms to the marina launching ramp and construct the temporary coffer bund (refer to Photograph 6).
14.) Complete construction of the promenade retaining wall. Remove temporary working platform and complete final seabed excavation and dredging works in this area. Transport all excavated material to the approved disposal area (two berths were made available for WWK to use upon completion of this task).
15.) Complete construction of the marina launching ramp extension. Remove the temporary coffer dam and complete final seabed excavation and dredging works in this area. Transport all excavated material to the approved disposal area.
16.) Disestablish from site and admire the final outcome (refer to Photograph 7). Note the north-western retaining wall is visible in the top-right corner of this photograph.

CONCLUSIONS

From a programming perspective, this was an extremely challenging project which required the highest levels of stakeholder engagement, responsiveness and collaboration from the Delivery and Design teams to ensure its success.

Stakeholder, environmental and Health and Safety requirements combined in a way that meant the Design Team had to be flexible, work extremely closely with the Delivery team, and, be willing to discuss and modify



Photograph 6: South Bay Marina, 24 October 2017. Construction of the temporary coffer dam around the marina launching ramp is complete (top left-hand corner of photograph).

their designs, as appropriate, to ensure best-for-project outcomes were achieved. An example of this was the change to the retaining wall starter bar design to reduce the risk of reinforcing steel damage during pile installation and rework.

This project also reminded the authors that the best overall design and delivery program is not always the one which results in the quickest and cheapest construction. For this project the best construction methodology, sequence and program was one that enabled restricted use of the marina facilities by commercial stakeholders, thus allowing them to earn some income while the repair works were being constructed. In the case of this project, a relatively modest increase in construction cost enabled many millions of income to be earnt by the tourism and commercial fishery sectors while the repair works were being completed. This, obviously, had significant direct and indirect benefits such as reducing the impact of the earthquake to local businesses, allowing some contimuity of service to their key Clients, enabling some ongoing local employment, with the associated downstream benefits to the local economy.

Discussions with the wider stakeholder group indicate they unanimously agree this project was a great success. Many have indicated they believe the repaired facility is better than the original, as it incorporates their feedback and suggestions for improvement. Most of these improvements were achieved for no additional cost to the original budget. Those improvements which resulted in additional cost were discussed by the stakeholder group, and the primary beneficiaries made financial contributions to enable them to be constructed as part of the NCTIR repair program without unfairly disadvantaging any of the other stakeholders.

In the opinion of the author's, this project is an excellent example of the program efficiencies, value engineering and best for project and stakeholder outcomes which can be achieved by an alliance type project.

Photograph 7: Completed Whale Watch Kaikōura berthing area and north-western retaining wall. (Photograph taken at approximately high tide on 14 November 2017).

ACKNOWLEDGEMENTS

We would like to acknowledge the stake holder working group, Environment Canterbury, Kaikōura District Council, NCTIR and the New Zealand Government for all your contributions to throughout the planning, design and construction of the South Bay Marina and Coast Guard harbour earthquake repair works.


WE'RE OFTEN TOLD
WE'RE THE BEST AT WHAT WE DO
CALL US AND WE'LL PROVE IT
PERRY GEOTECH
Mobile 0274 304554
Phone (07) 5780072
EXTRACTING THE PAST
shaping the future
PHILLIP FALCONER
0274 304 554
JOSH KENDRICK
0272 855 004
EMAIL: OFFICE@PERRYGEOTECH.CO.NZ
37 GLENLYON AVENUE, P.O. BOX 9376,
TAURANGA 3142, NEW ZEALAND
WWW.PERRYGEOTECH.CO.NZ
PERRY GEOTECH

Rockfall Modelling for Coastal Transport Corridor Recovery following the November 2016 M7.8 Kaikōura Earthquake

1. ABSTRACT

On November 14th 2016, a Magnitude 7.8 (Mw) earthquake occurred in the South Island of New Zealand, centred approximately 60km south-west of the coastal town of Kaikōura. The complex sequence of ruptures resulted in significant damage to a number of major roads and transport corridors, including the Inland Kaikōura Road, State Highway 1 (SH1) and the adjacent Main North railway Line (MNL), cutting off all land routes into Kaikōura.

The mountains of the Seaward Kaikōura Range rise abruptly from the adjacent coastline, with a limited platform along the coast forming the narrow transport corridor. Significant damage was incurred from large scale rockfall and rock avalanches triggered by the earthquake. Initial recovery efforts focussed on removal of the nearly 1 million cubic metres of rock and debris which inundated the road and rail, in order to allow access for the repair and improvement of the transport corridor.

Rockfall modelling encountered a number of major challenges in the initial recovery phase, with a significant number of large complex slips, limited site information, and rapidly evolving rockfall problems. Considerable efforts were focussed on gathering specific rockfall information from site, which in turn was used to calibrate 2D rockfall models and inform the type and extent of temporary protection needed to ensure the safety of staff working in close proximity to the slips. This paper summarises the key challenges and innovations in understanding, modelling, and effectively mitigating rockfall risk during the initial recovery phase of the project.

2. INTRODUCTION

Over 80 landslides occurred along the section of the transport corridor surrounding Kaikōura following the initial earthquake, with numerous additional failures observed in the weeks and months afterwards. The scale and number of these landslides severely impacted State Highway 1 (SH1) and KiwiRail's Main North Line (MNL), with approximately 10km of the transport corridor to the south of Kaikōura and 14km to the north of Kaikōura the most heavily affected. Among these landslides, a number of major landslides failed resulting in extensive debris cones inundating the road and rail corridor.

As well as the initial earthquake-generated mass failures, the significant shaking experienced during the earthquake resulted in extensive fracturing and ground damage in the colluvium and rock forming the slopes above the transport corridor. Associated ground damage effects of the earthquake included tension cracking and dilation of rock outcrops, as well as large scale removal of soil and vegetation leading to increased potential for further instability. This resulted in numerous large scale active rockfall source areas, as degradation of the damaged slopes released material downslope. These failures were observed to occur both as a result of rainfall and aftershocks, as well as continued "fair weather" releases of material.

The highly active nature of the rockfall source areas resulted in continued progradation of the debris cones at the base of slopes, cutting off access to recovery efforts, as well as causing serious risk to workers attempting to access site. In order to allow removal of the debris to begin design and construction of temporary protection measures needed to be undertaken, to reduce the risk posed from unexpected rockfall generated from the slopes above.

Tiarnán Colgan

Tiarnán is a Geotechnical Engineer who has worked for AECOM New Zealand for the last 5 years. Tiarnán has been part of the NCTIR project since February 2017, working across a range of roles mainly focused on 3D photogrammetry, rockfall modelling, and the design of mitigation measures and protective structures for multi-hazard sites.

Figure 1: Ohau point landslide debris covering SH1

3. INITIAL RECOVERY PHASE

In order to begin the process of clearing debris from the transport corridor, the first step involved creating safe access for staff and heavy plant to the slip sites along the coastline. Due to the linear nature of the transport corridor in this area, and the narrow coastal platform this corridor inhabits, large scale landslides at numerous locations completely inundated this platform, effectively blocking access to the sites further along the coast. As such, temporary protection was required in order to allow safe travel past active rockfall sites, in order to begin recovery works at multiple areas along the affected section of coastline.

As well as the development of large debris fans at the base of slopes, the extensive damage to the rock mass within the failure areas resulted in a considerable amount of debris remaining on the slope itself. In order to remove material from this on-slope source extensive helicopter sluicing was carried out to encourage the movement of this material downslope. As a result of this, the profile and location of the debris fan continued to change as more material was deposited at the slope toe. Rockfall modelling therefore required frequent updates to account for the changing behaviour of the hazard with time. Furthermore, once the material had been transported downslope, coordinated efforts between the design and site teams were needed to develop plans for the safe removal of the deposited material, in a way which limited exposure to the risk of rockfall from above. As the shattered ridgeline and colluvium deposits continued to generate rockfall as the debris fan was gradually removed, further updates were required for rockfall modelling in order to take into account the changing nature and material parameters of the newly exposed areas of the slope face.

A range of short term protective measures were used, tailored to the specific nature of each slip and the available space at the slope toe. Larger sites with significant debris fans used this deposited material to create catch benches and protective bunds, which were able to be deconstructed and reformed as earthworks progressed with the removal of debris; while smaller sites which produced less material incorporated a range of short term measures including shipping containers, water filled barriers and concrete block walls. At sites where the rockfall risk could not be demonstrably reduced to an acceptable level to allow contractors to work below the slips, remotely controlled excavators were used to carry out work until the appropriate level of protection could be

Figure 2: Inundation of SH1 and MNL

Figure 4: Rockfall catch ditches and earth bunds, forming safe access around Ohau Point

Figure 3: Helicopter sluicing

achieved. Operation controls were also heavily engaged in order to protect staff travelling below rockfall sites, with the use of spotters, traffic control, and remote monitoring to identify changing hazards and coordinate with teams working on the slopes; as well as strict controls on work in response to rainfall or aftershock events.

4. SIGNIFICANT CHALLENGES FOR MODELLING AND DESIGN

Due to complexity of the hazards present across the wider project area, the timeframe within which work was being carried out, and the separation between site and the design office, a number of significant challenges were encountered in the development of designs for protective measures in the early phases of the project. The NCTIR programme of works was a disaster recovery effort. This required decisions to be made early without the level of supporting information or investigation which would normally be undertaken on the projects if they were pre-planned and done in isolation.

Complex source areas, and sites which exhibited multiple hazards such as rockfall, debris flows, and shallow landslides meant that, in many cases, protective measures were required to cope with a number of different hazards at the same time. Continued deposition of material downslope meant that surveys of slope profiles became out of date within days, and the processing time required to get up-to-date survey information meant that often this information was obsolete by the time it reached the design office.

Complicated source areas and the variations in the source rock/colluvium between individual sites meant that it was not possible to develop consistent rockfall material parameters for use across the project. The rapidly evolving slopes meant that both on-slope material parameters, and the distribution of rock sizes modelled, needed to be frequently updated. The separation between the design office (based in Christchurch), and the site itself meant that it was not possible for the design team to witness these changes in person, and so were reliant on the accurate translation of this information from site based teams. With limited time for rigorous testing of design solutions, verification of the effectiveness of the protective

Figure 5: Abseil scaling and targeted sluicing

measures needed to be carried out on site, with results relayed back to the design team and used to inform the next round of modelling.

The particular challenges of this project required innovative solutions and sources of information in order to ensure the accurate modelling and design of protective structures. The greater understanding of the on-site hazards this developed allowed for rapid assessments of changing sites, and the effective communication of the risks posed to staff and equipment. This allowed for the most appropriate and informed decisions on risk and protective measures to be made by the site team, in order to ensure the safety and efficiency of recovery works, and the protection of site staff working in hazardous areas.

4.1 From challenges to design inputs

Whilst a number of features of the recovery effort acted to constrain the level of detail which could be afforded to the design process, namely the complexity of the site and the speed at which recovery works were undertaken, other aspects of how this work was undertaken acted to facilitate the development and testing of solutions within the time frames required.

Efforts at removing unstable rocks from the source areas were concentrated initially on the use of intensive, targeted helicopter sluicing, made possible by the close proximity of the work area to the ocean. This allowed teams of 3-4 helicopters to work in rotation on a single slope to saturate and dislodge unstable sections of the on-slope debris. Once the most hazardous sections of the slope and the larger scale features had been treated to the point where the slope was considered safe enough to allow abseil access from the slope crests, targeted removal of individual loose rocks was undertaken by crews of abseil teams using hand tools, airbags, and explosive and non-explosive charges. The combined approach of helicopter sluicing and targeted scaling resulted in a huge volume of material being transported down the slope in a semi-controlled environment; with the operational controls in place to facilitate safe access to site meaning that these falling rocks were able to be directly observed and recorded by spotters and on-site geologists. The observations of these induced rockfall events acted to significantly increase the understanding of the ways in which rocks travelled down the slope, reflecting both wet weather and dry weather releases in the use of sluicing and scaling, respectively.

In the initial months after the earthquake, the large landslides which cut off road access to the majority of the project area meant that helicopters needed to be used to shuttle staff between work sites. The unstable nature of the slopes meant that abseil work and inspections of the slope needed to be completed from above, with site geologists and abseil teams flown to landing platforms upslope of the major slips and scaling work coordinated from there. These helicopter flights provided numerous opportunities every day to carry out aerial inspections of source areas and slopes, as well as facilitating the gathering of useful photos and videos of the source and deposition areas of slips whilst travelling between work sites. This information, along with the rockfall observations discussed previously, contributed significantly to the design process, allowing for a much greater understanding of the behaviour of falling rocks across variable sites, and so greater accuracy and reliability in the development of predictive rockfall models within a short timeframe.

5. KEY ASPECTS OF THE DESIGN PROCESS

A number of important aspects and sources of information were identified throughout the initial recovery stages which contributed greatly to the overall rockfall modelling and design process. Many of these are of particular importance and relevance to the specific nature of the Kaikōura earthquake recovery works, however the lessons learned from this process present significant benefits to future disaster recovery works. The main inputs and information sources for this process are discussed in this section.

5.1 Baseline slope surveys

Initial slope surveys were generated from LiDAR information, with pre-earthquake profiles generated from a LiDAR Digital Elevation Model flown in 2012. Post-Earthquake LiDAR and aerial photography capture was flown in November 2016, allowing the generation of post-earthquake cross sections from this source. The comparison of cross sections generated from these two surveys was used to determine the significant areas of slope failure and material deposition, as well as the type of mass failures exhibited at certain sites. In areas where significant material deposition inundated the lower slopes,

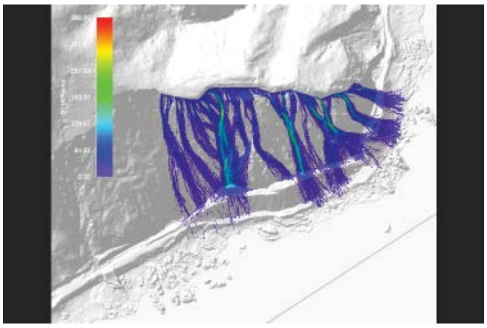


Figure 6: 3D rockfall modelling to determine critical rockfall pathways

the pre-earthquake cross sections were used to infer the approximate location of historic rock and talus surfaces, informing initial targeted clearing works and the estimation of debris volumes. In the initial months of the recovery effort, slope profiles for rockfall modelling were based on the post-earthquake LiDAR surfaces, with profiles for upslope source areas used directly from this survey. As the continued deposition of material downslope acted to change the profile of the lower slopes, the profile, slope angle and gradual progradation of the debris lobe at the slope toe was captured in site notes from on-site geologists overseeing recovery works. These notes were used to infer the development of surface profiles for the debris lobes used in rockfall modelling.

Post-earthquake surveys were used in a number of ways to generate cross sections for rockfall analysis. At complex sites, qualitative 3D rockfall modelling was used to infer the most probable rockfall pathways downslope using RAMMS:Rockfall. The nature of this software was not suited to the wide variation in material parameters observed on site, so this program was used as an initial screening tool to determine the most critical areas of slope for modelling. At less complex sites, where the main rockfall chutes and pathways are more obvious, or open slopes mean there are no obvious concentrations of rockfall trajectories, adjusted water drop analysis was carried out along with site notes and observations on observed rockfall pathways, in order to develop 2.5D or "quasi-3D" modelling. This involves determining the horizontal rockfall trajectory on the X-Y plane by identifying preferential pathways rocks are observed to travel down, and then translating this into a 2-D profile for use in the traditional 2D modelling program RocFall.

Throughout the project, project-wide LiDAR imagery was updated at approximately 6 month intervals. Sites which exhibited significant changes in the intervening periods due to earthworks or natural failures, rendering the survey information out of date, had individual surveys carried out using UAV scans. Due to the lead time involved in getting this survey information (approx. 2-4 weeks for UAV due to demand and processing requirements), often it was not possible to work with the most up to date surveys from site. As such, alternative methods were used to infer changes in the slope profile, including site measurements and 3D photogrammetry modelling.

5.2 Detailed site notes and recording of rockfall information

During the initial recovery works, concentrated sluicing and scaling efforts meant a large number of "rockfall events" were being artificially created in a semi-controlled environment. Combined with the presence of spotters at either side of the base of slips, and on-site geologists positioned at slope toe and crest, this allowed for the opportunity to observe numerous rockfall trajectories on-slope and at the slope toe. The large volume of data gathered during this phase allowed for the development of dependable rockfall models, calibrated to the observations of rockfall behaviour at each individual site. Additionally, the use of helicopters for site flyover inspections allowed for detailed descriptions of source areas and the potential range of rock sizes within these sources. A number of key observations, and their inputs to the design process are described below.

5.2.1 Rockfall source areas and slope characteristics

A key component of the development of rockfall models and their relevance to the problem in reality is the recording of accurate and up to date site mapping. This included a wide range of observations, translated to the design team through site notes, and annotated photos and site maps. The type of information gathered included:

- Location and activity of source areas, including the size and frequency of released material, the trigger events causing these releases, the dominant mode of failure at an individual source/site (i.e. discrete rockfall, debris flows, mass slides etc.)
- Dominant rockfall pathways, with estimations for the percentage of rocks observed to travel down individual pathways
- Key on-slope features such as chutes which concentrate rockfall pathways, or launching points which significantly affect rockfall trajectories
- Ongoing development of slope features, such as progradation of the debris fan, the change in material types on the surface of this debris fan following sluicing and scaling, or the movement of islands of vegetation in the upper source areas
- Material characteristics across the slope, with descriptions of roughness and material types for debris, colluvium, soil cover and exposed rock

5.2.2 Rockfall characteristics

Due to the wide variation of materials and rockfall characteristics across the greater project area, it was not possible to develop consistent material parameters for use across multiple discrete sites. As such, rockfall models were individually calibrated to the direct observations of rockfall at each site. This required specific detailed information on rockfall behaviour, which allowed for comparison and calibration between modelled and observed behaviour. Specific measurements and observations included:

- Bounce heights of rocks at various positions on the slope, including: in the upper debris fields, at the location of any significant launching points (where the location of the first bounce after this point was also noted), along the debris fans at the slope toe, and at the original road location
- Stopping points of falling rocks, with general indications of the percentage of rocks noted to come to rest at various sections of the slope and debris fan, translated as "contour lines" of percentage of rocks passing a certain point, annotated on site photos and aerial images
- Any observations on rockfall translational or rotational velocity, with specific notes on if/how these factors affected the travel of the rocks downslope or their final resting point, and any observed links between these velocities and the size/shape of rocks or trigger event which caused them to fall down the slope (e.g. rainfall, sluicing, scaling, or impact from rocks launched from upslope entraining debris as they travel downslope)

5.2.3 Boulder size estimates

Estimates of boulder sizes forms one of the most critical components of rockfall modelling, as this is a critical component in determining rock impact energy. Specific information recorded included:

- Estimated distribution of boulder sizes remaining in the source area (size range, 50th, 95th and 99th percentile boulder sizes, as well as the weighting of the distribution);
- Estimates of boulder sizes and coarseness of material forming the debris cone at slope toe (this parameter is related to the roughness of the debris slope, and is a controlling factor in rockfall trajectories along the runout area);
- Estimates of boulder sizes reaching the toe of slope, and any observations which indicate different behaviour for different rock sizes (e.g. larger rocks are observed to roll with low bounce heights and high rotational velocity, smaller rocks are observed to have higher bounce heights, with lower rotational velocity)
- Rock sizes at source and at point of deposition, to determine whether the rock quality in this source area means large rocks travel down the slope intact, or fragment as they travel

Where possible, boulder sizes were measured to give accurate volume estimates. Initial attempts at recording slope observations indicated that due to the high activity levels of the rockfall sources, it was neither practical nor safe to measure the rocks in the runout zone by hand. It was also found that estimations of individual boulder volumes were highly subjective, with little frame of reference for site staff to estimate volumes against. As such, falling rocks were grouped into more easily identifiable sizes, in order to bring more consistency to the process. This involved developing a range of equivalent boulder size estimates allowing for much more rapid, repeatable estimation of volumes. This allowed for rapid estimation of boulder sizes and size ranges, when individual large releases upslope could result in 10-100 boulders reaching the slope toe at the same time. For more accurate calculation of volumes for the larger, more critical boulders, this could be carried out after sluicing and scaling had been halted at the end of the day, as these larger boulders were unlikely to be covered by debris during the subsequent releases of material downslope. In less active areas these measurements could be carried out by hand, however in more exposed areas this could be carried out using methods such as 3D photogrammetry modelling or measured from calibrated scaled photos using apps like Split-Engineering (discussed in Section 5.3 below).

5.3 Visual media

One of the methods of capturing site information which proved to be of the greatest benefit to the design process was the effective use of visual media. This took on a variety of forms, based on the specific information which needed to be conveyed to the designers, and could be transmitted quickly from site to design office, facilitating quicker turnaround times on individual designs, as well as developing a greater understanding of the scope of site hazards, and the changes to these hazards on a daily basis. The analysis of this information also provided robust, dependable information for design inputs, and allowed design teams to spend time analysing individual aspects of an event or site in order to extract the information they needed. The applications and methods used are summarised in the following sections, with a brief description of some of the lessons learned through the early stages of the project.

5.3.1 Daily flyover photos

Part of the operational controls installed to reduce the

GEOTECHNICS

Testing

Environmental

Geotechnics provide water quality testing, contaminated land monitoring (visual inspections and the monitoring of soil, surface water, groundwater, trade waste, landfill gases and VOCs) and vibration monitoring.

Geotechnical & Construction Materials Testing

Field - Geotechnics has a dedicated team of geologists and technicians who undertake earthworks and civil testing. These teams perform quality control testing on earthworks and pavements for many large development projects. We can operate from our main offices or set up on-site testing facilities to service large projects.

Lab - Geotechnics has well equipped modern laboratories, providing excellent laboratory testing services for the civil engineering industry. Our scope of testing covers aggregates, concrete, rock and soil..

Instrumentation

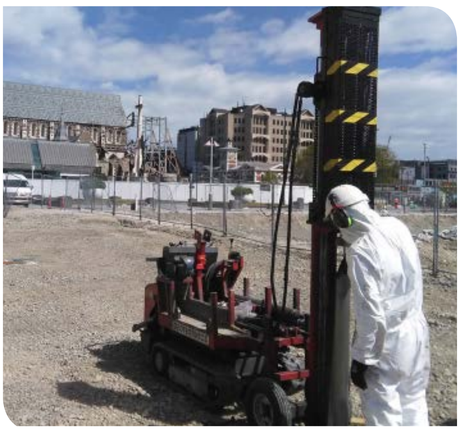
Our specialist technicians can assist in identifying the most suitable instrumentation and layout for an application along with installation, ongoing monitoring and reporting. Specialist instrumentation includes ShapeAccelArrays, Telemetry Systems, Inclinometers and Vibrating Wire Piezometers.

Pavements

Geotechnics offers a comprehensive road testing service, covering a wide range of applications. These include single lane unsealed access-ways to multi-lane highways, airport runways, container terminals, wharfs, crane pads and building platforms.

Site investigations

We have a team of geologists/technicians who undertake geotechnical investigations throughout New Zealand, including Hand Auger investigations, Terrier Rig drilling, Window Sampler drilling and CPT tests.



Underground Services Location

Geotechnics provides a one stop shop for your Underground Services Location needs. We can obtain service plans and provide on-site services location using Electromagnetic Induction tools, Ground Penetrating Radar, manhole lifters and Hydro-excavation to confirm services. Our utility service location procedures have been fine-tuned to minimise your risk when performing ground penetrating activities.

Sales

Geotechnics offer a full range of field and laboratory testing equipment for soils, concrete, aggregates, asphalt and other civil engineering materials testing. We also offer an extensive range of equipment for environmental testing applications.

Hire

We have a range of civil and environmental equipment available for hire on short or long term basis. All equipment comes fully calibrated, cleaned and checked.

Measurement & Calibration

The Measurement and Calibration Centre (MCC) is a leading provider of measurement calibration, dimensional analysis, alignment and repair solutions both on site and in our laboratory. MCC is an IANZ accredited laboratory with many years experience in dimensional measurements.

MCC

exposure of site staff to risk involved carrying out daily flyover inspections of the individual sites along the project area. These flyover enabled site geologists to observe the changes to individual sites on a daily basis, as well as having the ability to assess the site for unexpected overnight instability or slips before allowing staff onto the work site. Initially this flyover was documented by taking photos with a smartphone, and relaying these images to the design office. The images however were often of low quality, partially obscured by reflections in the helicopter windows, or out of focus. The relatively narrow field of view and low resolution on fixed focal length camera phones meant that in order to capture sharp images of the source areas and slips, photos often had to be taken from quite close to the area of slope face in question, so lacking the context of the wider slip. Significant improvements were made to this process by assigning a DSLR site camera with high resolution, a wide angle to telephoto zoom lens, and a large sensor size. This enabled higher image quality to be captured in wider ranging light conditions and from further away from slope, allowing for greater context in site photos, and the ability to zoom into fine details on the slope. The camera used for this application came with built in GPS tagging capabilities, allowing photos to be included in GIS maps and accurately placed on site, so that individual photo locations and dates can be catalogued for future reference. Additionally major improvements were made to the quality of helicopter based photos by simple changes to the process of capturing photos, such as making sure to use helicopters for the flyover inspection which allowed for shooting out the open window side window, rather than through the glass, eliminating problems with the camera focussing on the glass instead of the slip.

5.3.2 Videos of rockfall generated from sluicing and scaling

Due to the presence of spotters and site geologist around the slips, and the fact that the majority of people carry relatively capable digital cameras in their mobile phones, it was possible to capture numerous videos of rockfall generated during scaling and sluicing efforts from a wide range of different angles and aspects. Instances where multiple rocks fall at once as part of a larger release have too much information to be captured in site notes, but with time to analyse a video, the behaviour of these individual rocks can be assessed. This enabled designers to visualise the rockfall at multiple points along the slope, and further analysis of these videos proved to be a significant source of reliable information to inform the design process.

For example, a particularly useful rockfall video would show the progress of a falling rock down the lower debris fan, travelling out to reach the road at the slope toe. By identifying features on the slope which are present in both the video and the aerial site imagery from the post-earthquake LiDAR survey, it is possible to determine the approximate distance the rock travels down the slope over a given portion of the video clip. By analysing the location of this rock at various time-steps within the video (or the number of frames it takes for the rock to travel a certain distance, for a known framerate), it is possible to estimate the velocity at which the rock is travelling at various points on the slope. If it is possible to measure the rock after it has come to rest, or if in the video the rock passes an object of a known size such as a roadside barrier or shipping container, it is also possible to estimate the size of the rock relative to this object. This information then allows for: the estimation of bounce heights on the slope; distance covered between bounces, and time spent in the air; and given some assumption on rock volume and density, the approximate kinetic energy the rock has as it travels down the slope. All of these inputs are invaluable in the design of effective protective measures.

5.3.3 In-photo measurements using scaled imagery

Figure 7: Rockfall debris covering SH1

In locations where it was not possible to measure rocks due to safety concerns, photos could be analysed using the program Split-Engineering. This involves analysis of photos taken on a mobile phone using the Split-Camera app. This uses the known field of view of the camera being used, and measures the distance to objects using the built-in autofocus in order to generate scaled photos which can be analysed using the Split-Desktop software. This allows for detailed measurements of individual aspects of a photo, including individual boulder sizes, or the distribution of particles sizes within a debris pile. Using phone cameras from the distances required to safely separate staff from the rockfall hazard above, this was less accurate, and was eventually superseded by the use of scaled 3D photogrammetry models.

5.3.4 3D photogrammetry modelling

3D photogrammetry modelling involves capturing images

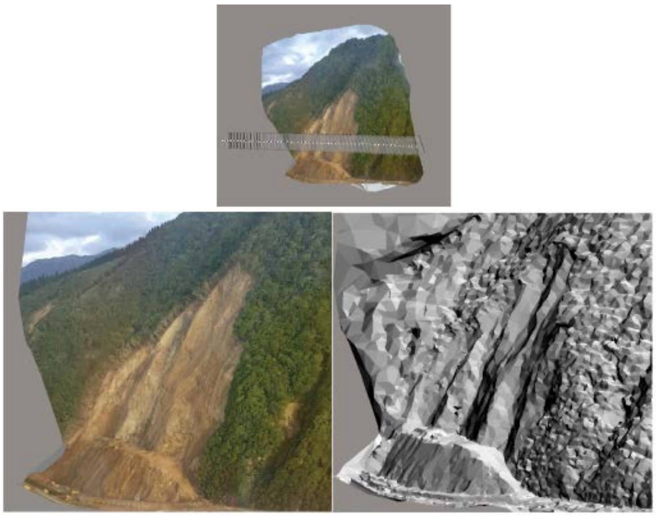


Figure 8: A basic 3D model created with frames from a single flyby video, taken with a mobile phone camera. Frame locations shown in top image

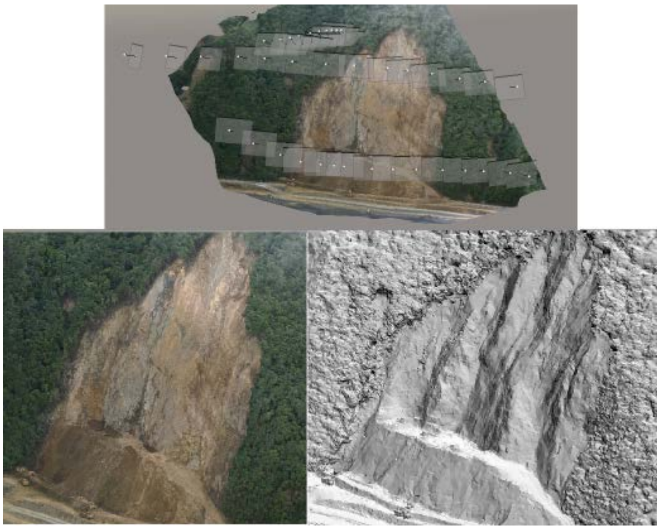


Figure 9: A more detailed 3D model, created from individual photos shot with a DLSR style camera. Photo locations shown in top image

of an object from a range of different angles, and then processing these photos with a program (Bentley ContextCapture in this instance) which identifies matching pairs of pixels between adjacent images. By mapping how these pixel locations changes across multiple photos, a very accurate 3D representation of the object can be formed. This process was used to create 3D models of entire slip sites with a very high degree of visual resolution, as well as a surprisingly high level of detail and accuracy in mapping the 3D structure of the sites. A wide range of inputs could be used to generate 3D models, depending on the level of accuracy needed in the final product. Simple models of slopes, intended to give a broad overview or a quick update of the site in question, could be captured from as little information as a 10 second video of the slip, taken from a helicopter window while travelling to another site. These 3D models could be processed in a matter of minutes, and gave a significantly better overview of the whole slip than could be conveyed using photos alone, as they allow the viewer to zoom into individual features of the slope, or to rotate and move around the slip to view features from different angles.

For more detailed models, the GPS enabled DSLR site camera was used to capture source images. These typically consisted of approximately 50-400 individual photos depending on the slip size, and were taken over the course of 2-3 passes of the slip in a helicopter at different elevations, capturing images of the slope and its features from a range of oblique and acute angles. The approximate flight time required to capture these images varied with the size of site, but was typically less than 5 minutes spent taking photos, with processing in the office typically carried out on the same day the images were captured.

The high resolution sensor of the DLSR camera allowed for the generation of incredibly detailed 3D images, with very precise texture mapping in the background model and a high degree of visual accuracy. The end product was a precise 3D model which could be moved and manipulated to see slopes from a multitude of different angles and levels of zoom, with each view possessing the image quality of the individual source photos. This was particularly beneficial when discussing clearing and mitigation strategies with on-site staff, especially abseil teams, as individual features which required attention could be viewed from the point of view of an abseiler on the slope, allowing for much more productive direction of work, compared to pointing out features on a slope.

Automatic GPS tagging of captured photos allowed the photogrammetry models to be fully scaled, with accuracy in the range of <10cm. On sites with specific features of interest, broad overview photos of the slip could be combined with highly detailed, zoomed in photos of the specific feature in question. This approach allowed for a wide range of applications including very precise measurements of rocks within a source area or at the point of deposition, dilation of cracks behind a significant rock feature, measurement of distances between slope features or downslope infrastructure, or the estimation of volumes of debris in a debris fan or stockpile. Many of these measurements would be impossible to measure on-site without putting staff in positions of extreme risk, and the rapid turnaround time from photos taken on site to the generation of a useable 3D model meant that this information could be analysed without having to wait up to 4 weeks for a UAV survey.

The fact that photos could be taken with little to no lead time required meant that 3D models of slopes could

Figure 10: Spray-painted rocks in a rockfall source area for tracking during rock-roll trials

Figure 11: Recovery works underway, beneath temporary rockfall protection measures, formed from a combined catch bench and ballasted shipping container wall

be created based on photos taken directly before and after a significant rainfall event, allowing direct comparison of the before and after states, and the attribution of individual slope movements to these trigger events. Due to the frequency and unpredictability of these events, relying on intermittent LiDAR scans or UAV surveys would have missed the effects of individual events occurring within the period between surveys.

The high degree of visual accuracy in these models meant that they formed a critical component of the rockfall modelling process. Instead of dealing with individual site photos of areas of a slope with no overall context, the 3D models could be used to assess the surface materials at every section of a rockfall pathway in one image, as well as allowing for the accurate measurement of individual rock sizes in the source area, final resting points of rocks at the slope toe, and the deposition or removal of material from the debris fan. Crucially, the quick turnaround time of the models allowed for frequent updates of the clearing process as contractors worked to remove the massive debris piles at the base of the slips. This imagery allowed for the analysis of the final slope profile and materials as the debris cover was gradually removed, allowing modelling efforts to keep up with developments on site.

These 3D models were stored on the project drive, and could be accessed using free viewing software installed on each computer, allowing the models to be used by a wide range of disciplines within the office concurrently.

6. TESTING AND CONFIRMATION OF DESIGNS

The final part of the design process during the initial recovery phase involved assessing the effectiveness of the designed temporary protection structures, and as such, the accuracy of the developed rockfall models. During the design process, a number of sites were analysed to check the actual trajectories and end points of rocks compared to the prediction in the model for that site. Typically this involved "rock-roll trials" where abseilers spray painted a number of rocks in the source areas so that they could be easily seen in videos, and easily located once they had come to rest. In instances where site observations indicated different behaviour for different rock sizes, rocks to be tracked were grouped into size ranges and spray-painted different colour to differentiate between behaviour of rocks of different sizes. These rocks were then scaled from the source area, and their trajectories downslope and final resting points observed and compared to predictions in the rockfall models to ensure accuracy.

After a design was finished for a temporary protective feature such as a catch bench or fill bund, details were relayed to the contractors on site for the construction of this feature. Once construction was completed, rockfall was induced through either targeted scaling or sluicing, and the effectiveness of the feature at stopping falling rocks was assessed. Once this had been completed, the dimensions of the feature could be adapted as required, and provided the level of protection afforded was deemed adequate, work could continue behind this protective feature, subject to the appropriate operational controls.

A case for updating NZGS (2005)

ABSTRACT

The New Zealand Geotechnical Society's "Guideline for the field classification and description of soil and rock for engineering purposes" (NZGS, 2005) has for 13 years been the standard by which soil and rock logging has been undertaken in New Zealand. NZGS (2005) does not go into exhaustive detail on technical matters regarding soils, but instead refers the reader to the Unified Soil Classification System (USCS) and recommends that its general principles be followed. The limited extent of both in-depth narration in NZGS (2005) and understanding of the USCS in New Zealand has created uncertainty as to which approach should be adopted on a number of important technical matters. This in turn has resulted in an inconsistency in output and interpretation. Using examples of where change would be beneficial, a case is made herein that NZGS (2005) should be updated to present clearer guidance on a range of subjects. Furthermore, it is argued that NZGS (2005) and the USCS are sufficiently different on fundamental issues that the NZGS should develop its own standalone soil and rock logging guide and that reference to the USCS be consigned to history.

Kevin Hind

Kevin is an Engineering Geologist at Tonkin & Taylor. After completing his M.Sc. in Earth Sciences at Waikato in 1985 he worked overseas for 15 years specialising in the geotechnical engineering of oil & gas structures. Since returning to New Zealand Kevin has focused on natural hazards and water/wastewater infrastructure. He has been with Tonkin & Taylor for 12 years and is a Technical Director in their Auckland office.

The New Zealand Geotechnical Society's *Guideline for the field classification and description of soil and rock for engineering purposes* (NZGS, 2005) is the standard New Zealand reference when it comes to geotechnical logging procedures. Its approach is broadly similar to those used in long-established foreign standards such as BS 5930: 2015 *Code of Practice for Site investigations*, AS 1726: 2017 *Geotechnical investigations* and ASTM D2487 *Unified Soil Classification System* (ASTM).

NZGS (2005) refers to the Unified Soil Classification System (USCS) as "*the basis for systematic [soil] classification*" and states that although it is not intended for the USCS to be "*followed to the letter*", its general principles should be adopted. NZGS (2005) is a short document, intended apparently to build on the USCS by providing definitions, brief insights into some technical matters and highlighting where New Zealand practice differs from the USCS, rather than being a standalone New Zealand classification system. In practical terms, the reader of NZGS (2005) needs to be somewhat familiar with the USCS in order to understand the complete classification process. The limited extent of in-depth narration within NZGS (2005), accompanied by a generally limited understanding of the USCS in New Zealand has, in the authors view, resulted in widespread uncertainty within the geotechnical industry as to how to approach the logging of soils. This in turn has resulted in inconsistent interpretations across the New Zealand Geotechnical industry.

In this article the author sets out a case for the NZGS to update NZGS (2005). It is argued that a substantially expanded document is needed, firstly to provide greater clarity on some technical matters and secondly to allow the New Zealand guide to become fully independent of the USCS. Examples of why this update is required are presented.

The case for an expanded document

The fundamental purpose of a classification system is to place something into one of a finite number of categories based on defined criteria. When a classification method is neither clearly defined nor widely understood, confusion and misclassification are likely to result. One area of geotechnical engineering that suffers from a lack of clarity is the classification of soil plasticity.

Despite plasticity being the single most important characteristic of a fine-grained soil, Section 2.3.4.2 "Plasticity" of NZGS (2005) dedicates a total of only seven lines to the subject. The terms "*low plasticity*" and "*high plasticity*" are introduced at this point, however no complete plasticity classification system is described. Example soil descriptions use the additional terms "*slightly plastic*" and "*some plasticity*". Despite being widely used in New Zealand practice, the category of "*medium plasticity*" is not mentioned. A list of acceptable plasticity categories is not presented.

With little clear guidance on what terminology to use, readers of NZGS (2005) are required to either accept the categories included in the document "as is"; to interpret the USCS for themselves; or to follow what they were taught at university or by senior colleagues in the expectation that this will be adequate. People who learnt their profession within a British or Australian context (the author included) may choose to fall back on the general principals of either BS5930: 2015 or AS1726: 2017, potentially unware of the sometimes significant differences between these systems and the USCS/NZGS (2005).

Many geotechnical practitioners believe that only low plasticity and high plasticity should apply in New Zealand, either because these are the only two terms used in Section 2.3.4.2. and/or because these are the only terms supposedly allowed by the USCS plasticity chart. The absence of any mention of "medium plasticity" in NZGS (2005) does nothing to dispel this impression.

If readers of NZGS (2005) are seeking guidance on soil plasticity from the USCS, they should refer to ASTM D2488 *Description and identification of soils (visual-manual method)* and not the actual definition of USCS, the laboratory-based ASTM D2487 *Classification of soils for engineering purposes (Unified Soil Classification System).* ASTM D2488 defines four states of plasticity: nonplastic, low plasticity, medium plasticity and high plasticity (Table 1). Given that NZGS (2005) is a field-based system that refers specifically to ASTM D2488-00, these same four plasticity categories should apply in New Zealand. Most people however are not familiar with ASTM D2488 and assume that the applicable plasticity terms are defined by the USCS plasticity chart (ASTM D2487), which is typically interpreted to have just two plasticity categories: low and high.

ASTM D2487 has four non-organic soil groups: lean clay (CH), fat clay (CH), silt (ML) and elastic silt (MH) as defined by the plasticity chart. These are almost universally referred to in New Zealand as low plasticity clay, high plasticity clay, low plasticity silt, and high plasticity silt respectively i.e. there are only low and high categories of plasticity and these are denoted within USCS by the letters L and H respectively. A recent survey undertaken by the author amongst nearly 70 colleagues returned a 98% response along these same lines. This interpretation of the USCS is however incorrect.

Description	Criteria
Nonplastic	A 1/8-in (3 mm) thread cannot be rolled at any water content
Low	The tread can be barely rolled and the lump cannot be formed when drier than the plastic limit
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit

Table 1: Criteria for Describing Plasticity (ASTM D2488)

From the very beginning of the USCS, the soil group codes L and H referred to liquid limit and not plasticity. This was clearly stated by USACE (1953) - "*The symbols L and H represent low and high liquid limits, respectively*". The initial version of the USCS (USACE, 1953; USBR, 1953) identified the following five plasticity categories: nonplastic, slightly plastic, low plasticity, medium plasticity and high plasticity. These were associated with the relevant soil groups as shown in Table 2. This same information appeared in one form or another in the USCS through multiple editions of the USACE Technical Memorandum No. 3-357, the USBR "Earth Manual", as well as the first 20 years of ASTM D2487.

It needs to be acknowledged here that Casagrande (1948) could get a bit loose with terminology, for instance describing "*the entire range of plasticity is at present is divided into two main groups designated with the letters L and H.*" Here Casagrande was referring to the possibility of introducing an intermediate liquid limit range of 35 to 50 to what was then simply a low and high liquid limit system. We know that Casagrande was not referring to two classes of plasticity because he describes the four plasticity classes presented in Table 2. Basically he is acknowledging that the two liquid limit classes cover the entire suite of four plasticity categories. This language was clarified in the USCS (USACE, 1953; USBR, 1953) with the statement given earlier. The USCS never adopted an intermediate category, although BS5930 and AS1726 subsequently did.

The reasons for the misconception that only low and high plasticity grades are allowed under the USCS are probably threefold. The first and most obvious reason is that many geotechnical textbooks straight-out state that the USCS uses L and H to define plasticity. This appears to be most common amongst British and European authors who may be conflating their own systems with the

USCS. Wikipedia, everyone's first port of call to plug an information gap, continues to spread this fallacy (https://en.wikipedia.org/wiki/Unified_Soil_Classification_System).

Secondly, the USCS (ASTM D2487) does not actually discuss plasticity grades, not even with respect to the plasticity chart. All that it provides is a soil group name and group code. The only time plasticity is mentioned in ASTM D2487 is to state that plasticity is a putty-like property of clay and that silt is "*nonplastic to very slightly plastic*". The reader of ASTM D2487 is left to their own devices when it comes to assigning a plasticity class to a cohesive soil, which usually means interpreting that L and H in the soil group code refer to the only two valid plasticity categories. Prior to a major revision of ASTM D2487 in 1983, the information on the various plasticity categories presented in Table 2 was included in the USCS to provide the reader with guidance on the matter of plasticity. The revision committee apparently considered plasticity to be a descriptor and not a classifier of soils and it was best left to ASTM D2488. ASTM D2487-83 bears a lot of responsibility for the loss of knowledge in this area.

The third likely reason is the long-standing British influence on geotechnical engineering in New Zealand. Under BS5930, L and H do actually refer to low and high plasticity respectively (as they do in AS1726). It would be natural to assume that this also applies to the USCS, especially when many available references tells us that it does. BS5930 is free to define soil plasticity differently to USCS if it likes, however given that the British Code of Practice CP2001 (1957) included what they called the "*modified form of this ["Casagrande scheme"]*" in which "*fine-grained soils are subdivided into soils of low, medium and high plasticity (suffices L, I and H respectively*", the author suspects that rather than deliberately going its own way with respect to using the Casagrande plasticity chart, the authors of CP2001 (1957) misinterpreted the USCS when they directly correlated plasticity to the liquid limit. This typically does not affect soils that plot above the a-line or are parallel to it, as plasticity index (and plasticity) will increase with liquid limit in such cases. Those soils that plot below the a-line however may exhibit very different liquid limits for the same plasticity index, meaning plasticity and plasticity index are poorly correlated, which does not seem right. Casagrande never assigned high plasticity to MH soils, choosing to call them "*elastic*" instead. High liquid limits are not always associated with high plasticity or even a high plasticity index.

It is apparent from the above assessment that NZGS (2005) should, as a minimum, define the categories of plasticity that are allowed, which in the authors opinion should be nonplastic, low plasticity, medium plasticity and high plasticity. Slightly plastic is also a possible additional term, given its historical use in the USCS, however low plasticity is probably sufficient on its own. If indeed this was not the intent of NZGS (2005), then the required classification should have been clearly presented.

As an aside, the evaluation of plasticity set out in Table 1 essentially describes the Plastic Limit test (NZS 4402 Test 2.3). It would therefore be possible for a soil's "field" plasticity to be determined in the lab by an experienced technician. This would serve as a useful means of evaluating the accuracy of the field determinations presented on the logs. Currently only some laboratories provide descriptions of plasticity as part of their Atterberg Limit reporting.

The case for creating a standalone system

The primary purpose of the USCS is to assign soils to one of 15 soil groups, each of which has a unique name and code e.g. CL – lean clay. NZGS (2005) states that "*the use of these* [i.e. soil group codes] *is not encouraged in this guideline as this tends to force rather narrow, artificial limits to the classification process*. In this respect NZGS (2005) is probably referring to the fact that the New Zealand approach to soil classification allows for the use of transitional soil types such Silty CLAY and Clayey SILT, not just SILT and CLAY to which the USCS is limited.

Despite this advice, it is not uncommon to see the USCS soil group codes used on logs in New Zealand without apparent recognition that the two systems are incompatible at a rather fundamental level. According to NZGS (2005) it takes a fines content of only 35% for a soil to be considered fine-grained, whereas under the USCS

Major Divisions	Soil Group Symbol	Typical Name
Silts and Clays **Liquid limit less than 50**	ML	Inorganic silts....***slight plasticity***
	CL	Inorganic clays ... ***low to medium plasticity***
	OL	Organic silts...***low plasticity***
Silts and Clays **Liquid limit greater than 50**	MH	Inorganic silts...***elastic silts***[1]
	CH	Inorganic clays of ***high plasticity***
	OH	Organic clays of ***medium to high plasticity***

Table 2: USCS classification of fine-grained soils (partial reproduction of USACE, 1953)

Notes: 1: No plasticity is assigned to MH soils in the USACE table it requires a minimum of 50%. Soils with fines contents of between 35% and 50% will therefore be considered fine-grained under NZGS but coarse-grained under the USCS. The potential impact of this will vary according to geological environment, however it is worth investigating what the outcomes might be if we consider the database of Auckland soils previously reported by Hind (2017).

All of the soils in the database are fine-grained according to NZGS (2005) and each has returned a valid Atterberg Limit test result. Of these demonstrably cohesive soils, 16% classify as coarse-grained (Silty SAND) according to USCS. Yet if we consider the Atterberg Limit data for these Silty SAND's, the most common classification according to the USCS plasticity chart is CH or high liquid limit CLAY (Figure 1). Clay is the third most abundant material in these soils after sand and silt, however the quantity and mineralogy of the clay is sufficient for most of them to be classified as CLAY according to the USCS. If one was using the USCS system correctly, such soils would not have even been submitted to the laboratory for the Atterberg Limit testing as they must be classified as SM based purely on their particle size distribution alone. Using the USCS and NZGS (2005) systems together in this manner results in the geotechnical log describing a fine-grained soil (probably Sandy CLAY) together with a USCS group code for Silty SAND (SM).

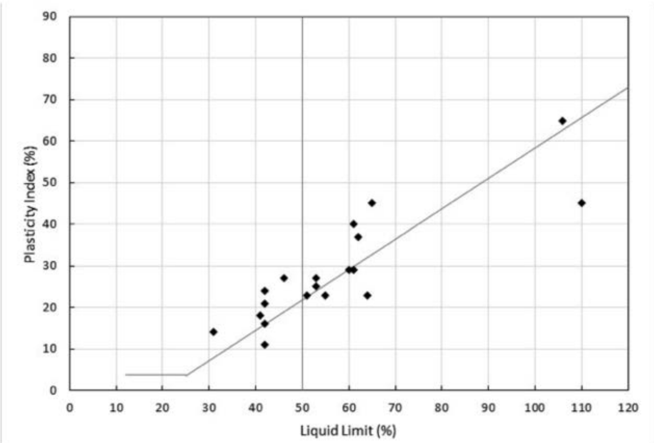


Figure 1: Cohesive Auckland soils that classify as coarse-grained (Silty SAND) according to USCS (data from Hind, 2017)

The example given above is without question an undesirable outcome, not only because of the clear contradictions it presents but also the potential impact on geotechnical assessments such as susceptibility to liquefaction, depending on which classification one chooses to believe. Some may argue that the two systems should not be used together in this manner and the author agrees. However until a NZGS (2005) update mandates that USCS codes not be used, then this practice will continue to occur. Furthermore, it is inevitable that laboratory results are compared to the field data when assessing geotechnical properties and performance of the soils, so we are inevitably forced to consider NZGS (2005) and USCS classifications together.

In an analysis of soil classifications undertaken for the entirely of Auckland soils, Hind (2017) presented evidence of a wide gulf between field and laboratory-derived classifications. The data indicated that field classifications were on the whole much more reflective of the actual composition of the soil (i.e. field classifications were silt-dominated, as were the soils) than the behavioural characteristics supposedly reflected in the plasticity chart (Figure 2). The latter classified the vast majority of Auckland soils as CLAY, and a large majority as CH.

A recent reassessment of the same database has indicated a relationship between field classifications and the actual clay content of the soil (Figure 2). It appears that as the clay content of a soil reduces, it is far more likely to be classified in the field as silt-dominated (e.g. Clayey SILT) than a CLAY. There are very few, if any, soils in Auckland that get classified in the field as low plasticity CLAY. This appears to be on account of a lower plasticity being seen as the product of an elevated silt content rather than a less active clay mineralogy.

The evidence presented by Hind (2017) and Figure 2 below suggests that the field logger is able to determine that the non-clay component is dominant (in a volumetric sense) and hence gives the soil a name such as Clayey SILT, yet the soil will generally be sufficiently plastic to plot above the a-line and be classified as a CLAY according to the USCS. In an environment such as Auckland, where the cohesive soils often have a subordinate clay fraction but one that is formed from high activity (swelling) clays, this may well be what is being observed. It needs to be remembered that many of the "clays" that Casagrande (1948) used to define the a-line actually had a significant non-clay component (e.g. Boston Blue Clay, London Clay), so the fact that many of the silt and sand-rich Auckland soils plot above the a-line should not at all be unexpected.

With field classifications and the plasticity chart sometimes resulting in different classifications for the same soil, it can be difficult to use both the field logs and laboratory data in a single coherent geotechnical assessment. There is often pressure to change logs to better reflect the results of the Atterberg Limit test results, without recognising that the two do not really tell the same story and are not measuring the same thing. The fact that NZGS (2005) and the USCS don't use the same soil names or even the same definition of what a fine-grained soil is, it must be acknowledged that the two systems really don't have that much in common. Using both systems together is bound to result in widespread inconsistencies and contradictions.

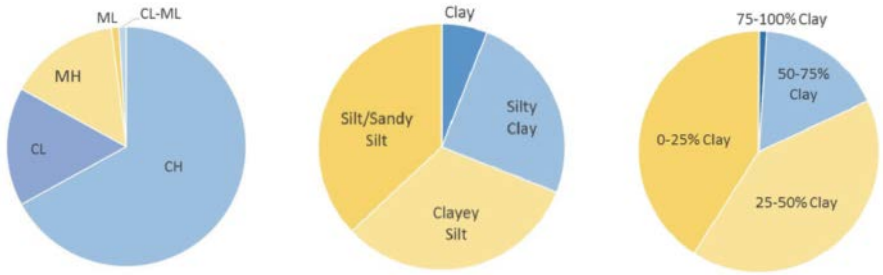


Figure 2: Auckland soils classified according to USCS (left) and NZGS (2005) (middle). The percentage of clay in the soils is summarised in the chart on the right. The field classifications (centre) are much more closely aligned to the proportion of clay in the sample (right) than the USCS basis of classification

The issue here goes well beyond the USCS constraining the classification process. The author maintains that the USCS is incompatible with NZGS (2005) to such an extent that all links to it, actual or purported, should be removed in favour of a complete and standalone New Zealand system.

CONCLUSIONS

A case has been made for the New Zealand Geotechnical Society's *Guideline for the field classification and description of soil and rock for engineering purposes* to undergo an extensive update and expansion. It is based on the author's opinion that reliance on the USCS has resulted in the existing document being insufficiently detailed to adequately inform the reader of the necessary approach to take in classifying soil in New Zealand. NZGS (2005) attempts to remain close to the USCS even though there are clear incompatibilities between the two.

It has been shown that some commonly held beliefs with respect to the classification of soil plasticity in New Zealand are incorrect. This could be addressed by the NZGS guidelines clearly stating what plasticity categories are allowed, together with presenting Table 1 (above) which will be the definitive means of determining plasticity in the field. Although NZGS (2005) was specifically developed for fieldwork, an update should also address the sometimes problematic issue of comparing field and laboratory classifications. Although this is potentially a bit of scope-creep for the document, it would nevertheless be very beneficial to the geotechnical practitioner.

The recommended approach going forward is to acknowledge that the USCS is the origin of the various soil classification systems that we are familiar with, but that the modifications NZGS (2005) has introduced has made the two incompatible. The update should be expansive enough to not only provide sufficient explanatory text but also that it becomes entirely self-contained, with no need for the reader to refer to other documents, be it the USCS or anything else. Relying on outside documents simply results in misinterpretation and contradictions. At this point there seems to be no benefit at all of NZGS (2005) being aligned with the USCS. If parts of the USCS were to remain valid for the NZGS update (such as the plasticity chart?) then these should be adopted fully into the document.

This article has touched on only a couple of issues in and around soil classifications in arguing for an update to NZGS (2005). There are a number of additional areas of contention, such as whether the recording of RQD should continue or not. These and many other topics will need to be addressed in any update which the author hopes the NZGS will commission.

REFERENCES

ASTM D2487-83. 1983. Classification of soils for engineering purposes (Unified Soil Classification System). American Society for Materials and Testing

ASTM D2487-00. 2000. Classification of soils for engineering purposes (Unified Soil Classification System). American Society for Materials and Testing.

ASTM D2487-17. 2017. Classification of soils for engineering purposes (Unified Soil Classification System). American Society for Materials and Testing.

ASTM D2488. 2017. Description of identification of soils (visual-manual procedure). American Society for Materials and Testing.

AS 1736. 2017. Geotechnical site investigations. Standards Australia.

BS 5930. 2015. Code of practice for site investigations. British Standards Institute.

CP2001. 1957. Site Investigations. British Code of Practice. British Standards Institution.

Casagrande, A. 1948. Classification and identification of soil. Transactions of the American Society of Civil Engineers, Vol 113, Issue 1, 901-930

Hind, K.J. 2017. The Casagrande plasticity chart – does it help or hinder the NZGS soil classification process?. Proc. 20th NZS Geotechnical Symphosium. Eds. GJ Alexander & CY Chin, Napier.

USACE. 1953. Unified Soil Classification System (1953a). Technical Memorandum No. 3-357. U.S. Army Corps of Engineers. Office of the Chief of Engineers, Waterways Experiment Station, Vicksburg, Mississippi, Vol 1 to 3.

USBR. 1953. Unified Soil Classification System. A supplemental to the Earth Manual. U.S. Department of the Interior, Bureau of Reclamation. Denver, Colorado.


New automated
consolidation and
triaxial equipment

Global Survey on The State-Of-The-Art and The State-Of-Practice in Geotechnical Engineering

– Hugo Acosta-Martinez[1], Pierre Delage[2], Jennifer Nicks[3], Kim Chan[4], Peter Day[5]

ABSTRACT

This paper presents the results of a global survey on the State-of-the-Art and State-of-Practice in geotechnical engineering initiated by the ISSMGE **Corporate Associates Presidential Group and the Technical Oversight Committee in March 2017. It also summarises the discussions held on the topic during the 19th ICSMGE in Seoul on 20 September 2017.**

1 INTRODUCTION

The International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) is the pre-eminent professional body representing the interests and activities of Engineers, Academics and Contractors all over the world that actively participate in geotechnical engineering. As a truly global organisation, the ISSMGE provides a focus for professional leadership to some 90 Member Societies and around 20,000 individual members. Further details on the activities of the ISSMGE can be found at www.issmge.org.

One of the objectives of the Corporate Associates Presidential Group (CAPG) of the ISSMGE is facilitating the uptake of geotechnical research in practice thereby narrowing what is referred to as the "research-practice gap". To this end, the CAPG in conjunction with the Technical Oversight Committee (TOC) initiated a worldwide survey on the state-of-practice and the state-of-the-art in geotechnical engineering. The results of this survey were presented at a workshop at the 19th International Conference of the ISSMGE in Seoul on 20 September 2017. The workshop was organised jointly by the CAPG and the TOC, both of which are Board-level committees of the ISSMGE.

The purpose of this paper is to present a summary of the survey results and of the discussions held at the Seoul Workshop. The paper then identifies potential follow-up actions required to maintain the momentum of this initiative.

2 THE CAPG, TOC AND TECHNICAL COMMITTEES

2.1 Corporate Associates Presidential Group

In his introduction to the Seoul Workshop, Karel Allaert (Jan de Nul) described the CAPG as an ISSMGE Board-level committee which comprises representatives drawn from the Corporate Associates (CAs) of the ISSMGE. At the time of the Seoul Workshop, there were thirty-one CAs including corporations, consultancies, contractors, equipment and product manufacturers, as well as one university. CA logos, with links to their company web sites, are displayed prominently on the ISSMGE web page and included in each issue of the ISSMGE Bulletin. The CAs list as at September 2017 is presented in Figure 1.

The number of CAs peaked at 43 and it is clear that more members are required. It is believed that a stable platform of CAs could be about 60.

The key purpose of the CAPG is to assist the ISSMGE in developing actions and activities that will enhance the commercial sector of the geotechnical profession. Among these, identifying and helping to bridge the gap between the State-of-the-Art (SoA) and State-of-Practice (SoP) in geotechnical engineering has been a key activity for CAPG during the last term (2013-2017).

2.2 Technical Committees

The mission of Technical Committees (TCs) is to provide a forum for active participation by the individual members of ISSMGE, and to promote the objectives, activities and results of the technical committees throughout the ISSMGE membership. The TCs are a meeting arena for discussing, developing and applying specialist geotechnical knowledge related to the behaviour of geo-materials, geotechnical engineering and engineering for society.

Hugo Acosta-Martinez

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

1 Aurecon, Australia
2 Ecole nationale des ponts et chaussées, France
3 Federal Highway Administration, USA
4 GHD, Australia
5 Jones & Wagener and University of Stellenbosch, South Africa

Figure 1: List of Corporate Associates (September 2017)

There are 33 technical committees of the ISSMGE divided into three categories, namely Fundamentals, Applications and Impact on Society (Delage, 2017). These technical committees are listed in Table 1. Technical committees may be removed or added in the future depending on the interest and activity of the members. For example, a new TC309 is currently being created on Machine Learning and Big Data in Geotechnics.

2.3 Technical Oversight Committee

The Technical Oversight Committee (TOC) is in charge of supervising and coordinating the activities of the TCs of the ISSMGE.

The TOC is managed by a Chair and a Secretary. Its members are the six Vice-Presidents of the Regions. Each Vice-President follows the activities of the TCs from his/her region (http://bit.ly/2D5xfyx).

3 GLOBAL SURVEY

3.1 Background

In late 2013, the core group of the CAPG, with the support of the then President of the ISSMGE, Prof. Roger Frank, embarked on a project to work towards improving the understanding of the SoA and SoP in geotechnical engineering. The Chair of the TOC, Pierre Delage joined this working group in late 2014 and was pivotal in engaging in regular communications with all of the TCs. As a result, a mini survey on the SOA and SOP was conducted involving all of the TCs in early 2015, culminating in a discussion session during the European Conference in Edinburgh in September 2015.

Encouraged by the success of the mini survey and the discussion session, the CAPG core group and TOC decided to undertake a global survey inviting the TCs to develop the specific survey questions considered as "hot issues" in their field.

FUNDAMENTALS
TC101 - Laboratory Stress Strain Strength Testing of Geomaterials
TC102 - Ground Property Characterization from In-Situ Tests
TC103 - Numerical Methods
TC104 - Physical Modelling in Geotechnics
TC105 - Geo-Mechanics from Micro to Macro
TC106 - Unsaturated Soils
TC107 - Laterites and Lateritic Soils
APPLICATIONS
TC201 - Geotechnical Aspects of Dykes and Levees and Shore Protection
TC202 - Transportation Geotechnics
TC203 - Earthquake Geotechnical Engineering and Associated Problems
TC204 - Underground Construction in Soft Ground
TC205 - Safety and Serviceability in Geotechnical Design
TC206 - Interactive Geotechnical Design
TC207 - Soil-Structure Interaction and Retaining Walls
TC208 - Slope Stability in Engineering Practice
TC209 - Offshore Geotechnics
TC210 - Dams & Embankments
TC211 - Ground Improvement
TC212 - Deep Foundations
TC213 - Scour and Erosion
TC214 - Foundation Engineering for Difficult Soft Soil Conditions
TC215 - Environmental Geotechnics
TC216 - Frost Geotechnics
TC217 - Land Reclamation
TC218 - Reinforced Fill Structures
IMPACT ON SOCIETY
TC301 - Preservation of Historic Sites
TC302 - Forensic Geotechnical Engineering
TC303 - Coastal and River Disaster Mitigation and Rehabilitation
TC304 - Engineering Practice of Risk Assessment and Management
TC305 - Geotechnical Infrastructure for Megacities and New Capitals
TC306 - Geo-engineering Education
TC307 - Sustainability in Geotechnical Engineering
TC308 - Energy Geotechnics

Table 1: List of ISSMGE Technical Committees

SurveyMonkey was selected as the tool for hosting the survey questions. Sam Mackenzie (GHD) kindly offered to implement and administer the survey. The global survey was subsequently launched in March, 2017.

The main aims of the CAPG/TOC global survey were to gain a better understanding of the state-of-practice in the geotechnical profession, to identify areas for improvement and to provide feedback from the profession to the Technical Committees.

The survey was divided into three sections. The first section included general questions regarding the demographics of the survey respondents. This section also allowed the respondent to identify the Technical Committees they were interested in within the ISSMGE. The second section consisted of targeted questions compiled by each of the technical committees. Most of these questions were aimed at ascertaining the extent to which existing knowledge is being applied in practice, and the needs of industry and practicing geotechnical engineers. The final section invited respondents to provide general feedback on ways of narrowing the gap between the SoA and the SoP.

3.2 Respondent Demographics

The survey drew 1,295 responses from 68 countries. 84% of the respondents were male and 16% were female.

Figure 2 shows the number of responses received from the various participating countries. The majority of responses (56%) came from the European Region followed by 13% from Asia and 12% from Australasia.

Figure 3 shows the sectors of the industry in which the respondents are employed. Clearly the survey has achieved its objective with about 70% of the responses being from practitioners. One response was received from lawyers with none from insurers.

Figure 4 shows the distribution of respondents' number of years of experience in the industry. Overall, a wide range of experience was represented, helping to provide different viewpoints.

Figure 5 shows the percentage of respondents interested in each technical committee of the ISSMGE. Twenty eight percent of the respondents are members or corresponding members of technical committees. Seventeen percent of the respondents attend TC meetings and 25% attend TC-related conferences.

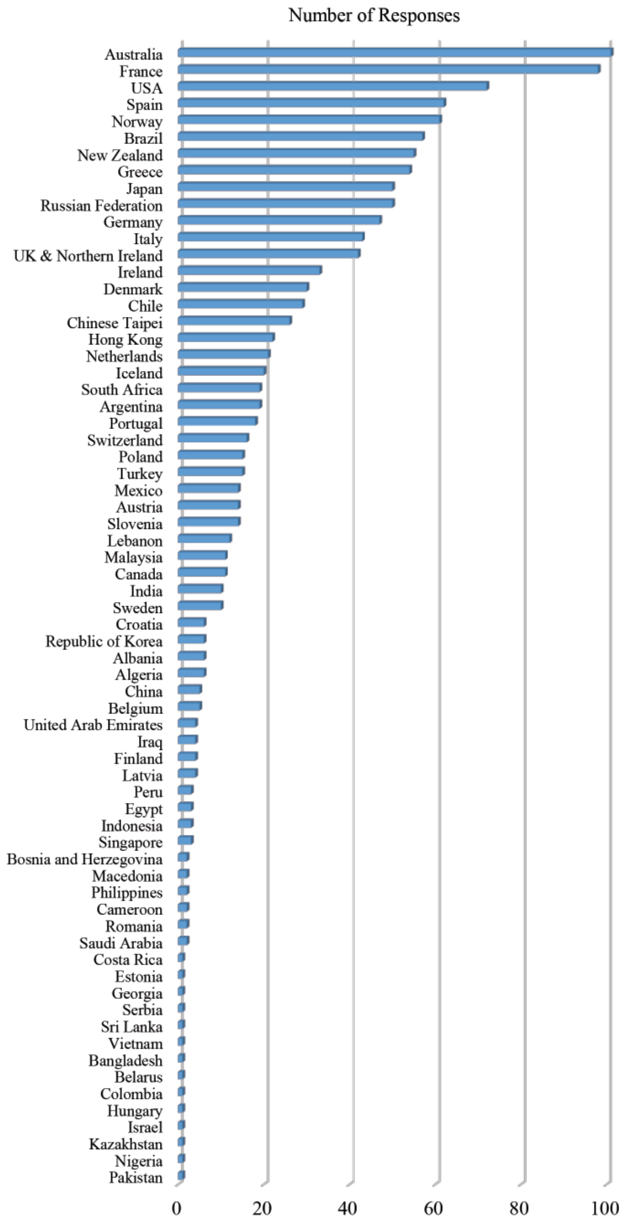


Figure 2: Number of responses by country

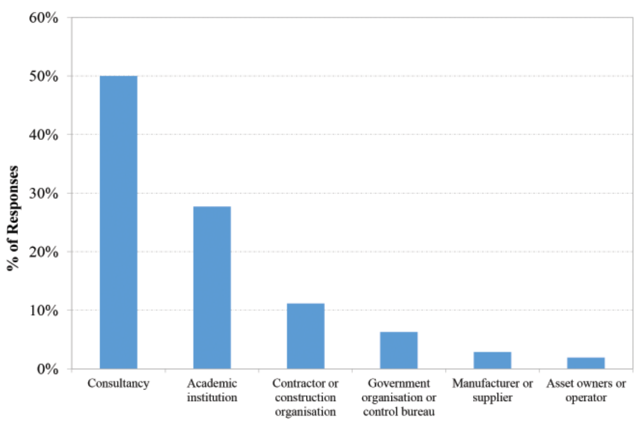


Figure 3: Industry sector of respondents

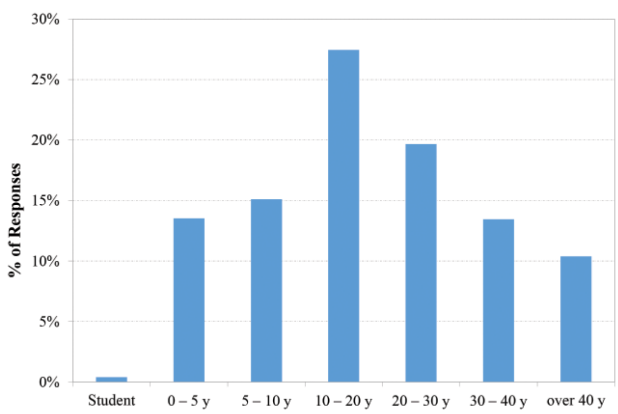


Figure 4: Number of years of experience of respondents

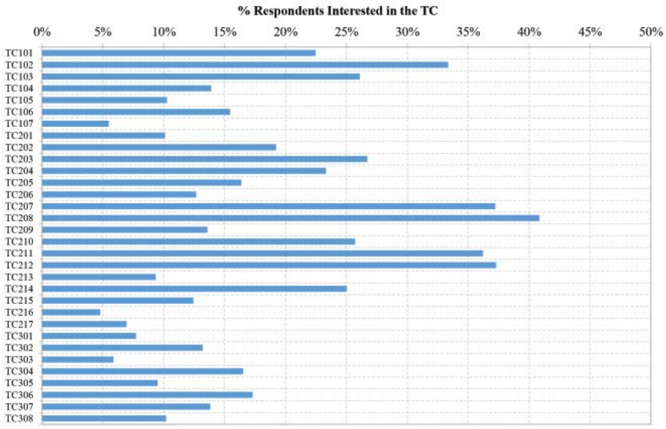


Figure 5: Respondents' interest in various Technical Committees

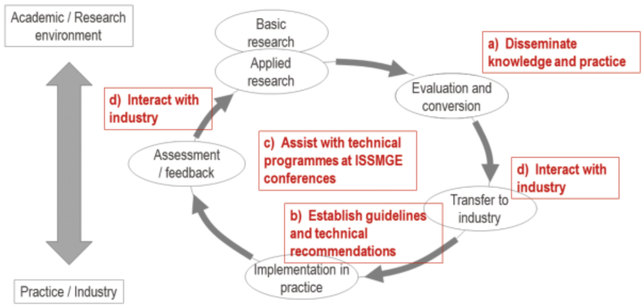


Figure 6: Integration of TC objectives in the knowledge development and implementation cycle

3.3 Response to TC Survey Questions

Twenty nine of the 33 Technical Committees listed in Table 1 provided a total of 232 questions for inclusion in the global survey. Respondents could contribute to the sections of the survey relating to the technical committees of their choice. A complete list of the questions and the responses received is available on the CAPG web site at CAPG/Downloads http://bit.ly/2mkAalj. Note that, in each case, responses have been numbered sequentially and that the numbers bear no connection to individual respondents. This summary is an adaptation of the analysis of survey results produced by Jennifer Nicks (FHWA, USA). Figures 2 to 5 come from this analysis.

In his capacity of Chair of the Technical Oversight Committee, Pierre Delage presented feedback on the survey to the Seoul Workshop with particular reference to the roles of the technical committees. A copy of the presentation is available on the CAPG web site at http://bit.ly/2mkAalj. A summary of the salient points is given below.

3.3.1 Role and objectives of Technical Committees

The objectives of the TCs as per ISSMGE guidelines are:

a) To disseminate knowledge and practice within the TC's subject area to the membership of the ISSMGE.
b) To establish guidelines and technical recommendations within the TC's subject area.
c) To assist with the technical programs of international and regional conferences organized by the ISSMGE.
d) To interact with industry and overlapping groups working in areas related to the TC's specialist area.

These objectives are closely aligned with the objectives of the CAPG. Furthermore, all these objectives form a part of the knowledge development and interaction cycle (Day, 2017) as illustrated in Figure 6.

As shown in Table 1, the ISSMGE technical committees are divided into three groups. The Series 100 committees deal with fundamentals such as soil properties and calculation / test methods. The Series 200 committees deal with application of knowledge in practice. The Series 300 committees deal with impact on society. These three groups of committees, although positioned differently as shown in Figure 7, each play a role in the knowledge development and implementation cycle.

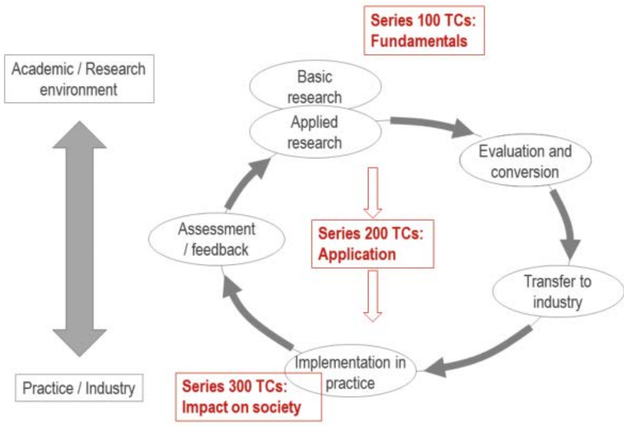


Figure 7: Role and positioning of the three groups of technical committees in the knowledge development and implementation cycle

Pierre Delage stressed that the vast majority of technical committees, particularly the Series 200 TCs, comprised a good balance of academics and practitioners, with approximately half of the participants from industry. The majority of the TCs hold regular activities aimed at transferring knowledge into practice, i.e. reducing the gap between the SoA and the SoP.

3.3.2 General feedback from global survey

Pierre Delage noted that the survey was an ambitious and difficult project and thanked all involved including the organisers, technical committees and respondents. A lot of effort and thought by the TCs went into preparing the survey questions and analysing the results.

The survey produced many interesting contributions, thoughts and feedback, providing new insights into the professional practice and technical committee activities. It is clear that academics, practitioners and contractors often think in different ways and may have divergent interests.

While the survey was a success, some of the TCs expressed frustration in that there was no information in the feedback they received on the origin of the responses and disappointment at the limited number of responses received. The timing of the survey may also not have been ideal for certain member societies with respect to the timing of their own activities.

Certain TCs expressed an interest in getting more responses to the questions asked, possibly by way of a follow-up survey. However, the results of the current survey should be evaluated first.

It is clear that the gap between the State-of-the-Art and the State-of-Practice requires careful consideration by the TCs and should receive further consideration in the planning of future TC meetings and activities.

3.3.3 State-of-the-Art and State-of-Practice

The State-of-the-Art (SoA) is the theoretical basis of the subject matter and is generally provided by the relevant technical committee, particularly TCs dealing with fundamentals (Series 100). This then needs to be incorporated into the State-of-Practice (SoP) in conjunction with the practical TCs (Series 200). The SoP represents a synthesis and analysis of practical experiences at any particular time in the light of the SoA. The SoP may be national, regional or international in application. Among the difficulties faced are that existing regulations may not be consistent with the current SoA and the time it takes for advances in the SoA/SoP to be incorporated into codes and standards. Often, no SoP documentation exists, and practice is based on personal experience of successes and failures.

In many instances, the SoA in the subject area of the TCs is contained in papers published at speciality conferences (e.g. the series of in-situ site characterisation conferences hosted by TC102). Certain TCs, such as TC215 (Environmental Geotechnics), TC301 (Historical Sites) and TC304 (Risk Assessment) disseminate this information in dedicated journals, books or working group reports.

The SoP in the subject area of many TCs is encapsulated in national, regional or international design codes and guidelines. These include EN, ASTM, ASCE, AASHTO, API, DNV, FHWA, CIRIA and other codes or documents. TCs that fall into this category include TCs 201, 203, 205, 209, 211, 213 216, and 218.

In the survey, respondents expressed the need for further guidelines (TCs 202, 208, 209, 211 and 304) while others requested better inclusion of the SoA in existing codes (TC212).

3.3.4 General feedback

The final section of the survey dealt with general feedback of respondents on ways of narrowing the gap between the SoA and the SoP. This general feedback was presented at the Seoul Workshop by Kim Chan (GHD) who summarised the main opinions as follows:

- Compulsory professional accreditation is seen as a key step in narrowing the gap between the SoA and the SoP.
- TCs should interact more with industry and the public sector so that the TCs are exposed to more real needs.
- Data interoperability and the establishment of pre-competitive data federations (such as those used in Australia and Canada to federate groundwater data) could assist in closing the gap. The application of the SoA requires the SoP practitioners to have access to such data.
- Academia should sometimes focus more on "practical questions" in their research. Research in geotechnical engineering must seek an application in practise.
- Coming up with a set of guidelines for each sub-discipline within Geotechnical Engineering and making these available to the ISSMGE community will go a long way to bridging the gap between SoP and SoA.
- Increase the number of symposia focusing on the case studies in geotechnical engineering to assist researchers in understanding the real behaviour of structures in order to model them in a better way.
- The gap between SoA and SoP can be bridged with continued professional education and involving practicing engineers in specific geotechnical committees.
- In the steering / drafting committees of regulations such as Eurocodes, a better balance between academics and practicing engineers should be sought.
- Often, SoA and SoP are both used for solving practical problems, SoA for more demanding problems vs SoP for more common problems.

Photo 1: Panel discussion session in full swing

4. SEOUL WORKSHOP PANEL DISCUSSION

4.1 Discussion Topic

In the spirit of the Seoul conference theme "*Unearth the Future, Connect Beyond*", two questions were formulated for the panel discussion:

a) Q1: How should we 'unearth' this material for the future to serve the geotechnical community?
b) Q2: How should we 'connect' this work to the 20th ICSMGE in Sydney in 2021?

Members of the audience were invited to come forward, join a circle of their colleagues and express their views. One such group is shown in Photo 1.

4.2 PANEL DISCUSSION CONTRIBUTIONS

A summary prepared by Hugo Acosta-Martinez (AECOM) of the main comments and discussions is given below.

Peter Day, University of Stellenbosch / Jones & Wagener Consultants, South Africa

- The application of new technology is often limited by the availability of the data required to apply the technology.
- Universities should consider asking industry which topics they wish to have researched.
- Universities should involve members from industry in both teaching and research activities.
- Discussion documents, such as the TC205/304 (2017) report made available at the conference, are valuable as they contain practical guidance and have been compiled by both practitioners and academics.
- We need to improve the quality and sufficiency of site investigation data by clearer specification of minimum requirements.

Kenichi Soga, UC Berkeley, USA

- Work with companies, invite them, have open discussions about how to work together.
- Organise sessions with companies.
- Foster company-university interaction.
- Bring infrastructure owners, contractors and clients to ICSMGE-Sydney-2021.

Jay Ameratunga, Golder, Australia

- Contractors and owners are the missing link. Work with them to further development of profession.

Marcelo Sanchez, Texas A&M University (USA) and Chair of TC308 (Energy Geotechnics)

- There were no surprises in the survey outcome.
- Ask ourselves where we want to be in four years.
- The diagnostics are there in survey
- Define milestones and objectives for Sydney-2021.

Peter Van Impe, Jan de Nul (Belgium)

- Transfer of knowledge is an issue.
- Specific knowledge is not always easy to find.
- Academia is controlled by the need to publish as this is often linked to research funding and career advancement. This has a perverse effect on the profession.
- It is impossible to follow up on everything that is being published.
- The need to publish to survive in academia is killing applied research.

Graham Scholey, Golder Associates (Australia)

- There is an opportunity to synthesise and address these concerns in ICSMGE-Sydney-2021.
- Identify the key people to address the conference.
- How to balance the number of papers with the quality of conference is an issue that needs careful consideration.
- An important question for the CAPG to answer is "What is in it for me as a Corporate Associate?".
- Corporate Associates need to receive tangible benefits.
- Important to increase number of CAs.

K.K. "Muralee" Muraleetharan, University of Oklahoma, USA

- Repeat the survey among chosen respondents.
- Analyse regional differences.
- Consider carefully the issue of sampling.
- Need 'far thinking' clients to support improvement.

Jennifer Nicks, Department of Transportation's FHWA (USA) and Chair of ISSMGE's Young Member Presidential Group

- Researchers are not rewarded for doing better.
- There are risks associated with trying something new.

Walter Paniagua, Chair of TC214 Soft Soils, Mexico

- Organise special sessions for commercial services / products.

- Contractors to be encouraged to present case histories in association with consultants.
- Research-to-practice papers should be encouraged at conferences.
- Acknowledgement that bridging the gap and managing expectations of commercially-oriented members are not easy tasks.

Soheil Nazarian, University of Texas at El Paso, USA

- Mutually exclusive expectations from academic and practitioners.
- Dissemination of research findings may not be permitted. Universities are funded by government.
- Invite the right people to write specifications and guidelines. Where no specifications exist for the application of new techniques, these techniques will not be used by designers for fear of litigation.
- Do not invite managers; they will not transfer knowledge; bring in young active engineers instead.

Ana Heitor, University of Wollongong, Australia

- Referred to experience at the University of Wollongong which has deep involvement with contractors and development of practical solutions to specific problems.
- Bridge the gap between SoA and SoP with education and training.
- Access to journals and databases is expensive.
- Suggest creating a platform for review of papers from the last few years on specific topics.
- Look further than Sydney-2021.

Ken Ho, Government of Hong Kong

- Reference to SoA could be ambiguous; for a given topic different answers from different universities are possible
- Add an assessment process before transferring knowledge. The GEO Office (Hong Kong) attempts to fulfil the role of assessing research findings and transferring relevant findings into practice by producing practical guidelines.
- Proposal to consider a Technical Review Board and consultation at local and international level.
- There is not always consensus among researchers on the value of new knowledge.
- Bring the right stakeholders to the table.
- 'Technovation forums' are suggested.
- Calibration of methods with real data and actual performance is important.
- New knowledge needs to be interpreted, e.g. by translating into design charts or computer programmes. The research institutions themselves need to take the process this step further.

Anand Puppala, University of Texas (Arlington), USA

- Work on big data.
- Industry funding is difficult.

4.3 Discussion Closure and Thanks

Valerie Bernhardt (Terrasol) closed the discussion session and thanked all the participants. In her closing remarks, she mentioned:

- The survey serves as a point of reference for next steps.
- The survey makes people aware of the real issues.
- The communication problem is from both sides. Everyone needs to make an effort; it is not a one-sided problem.
- Interact with other Board level committees.
- Survey for academic/consultant members.
- Open access is an important initiative to be maintained and expanded.

5. NEXT STEPS

In concluding the joint TOC and CAPG workshop session, Sukumar Pathmanandavel (Aurecon, Chair of CAPG) set out what he sees as the next steps in the process.

The CAPG and TOC, with help from TCs, plan to disseminate these findings among the profession. (This paper helps fulfil this intention.)

Specialised sessions are planned for the five ISSMGE mid-term regional conferences in 2019 to discuss, debate, and promote issues relating to geotechnical engineering that have, or are perceived to have, a significant impact on the commercial sector of the ISSMGE. CAPG will interact with the local organising committees to develop topics and invited participants relevant to the needs of each region.

The possibility of a further survey before the 20th ICSMGE in Sydney in 2021 has been raised. Inclusion of a request for topics that industry would like researched has also been mentioned. This will be considered by the CAPG/TOC.

Suggestions on this work can be submitted to Sukumar Pathmanandavel (S.Pathmanandavel@aurecongroup.com) and Peter Day (day@jaws.co.za).

6. ACKNOWLEDGEMENTS

The following representatives of the Corporate Associates were actively involved in the development and launch of the global survey:

- Sukumar Pathmanandavel (CAPG Chair), Aurecon
- Chaido Doulala-Rigby (Yuli), Tensar
- Kim Chan and Sam Mackenzie, GHD
- Karel Allaert, Jan de Nul

- Gabriele Zapf, formerly with Siemens
- Mandy Korf, Deltares
- Ian Hosking, AECOM
- Valérie Bernhardt, Terrasol/ Setec group.

Special thanks are due to:

- Roger Frank, ISSMGE Immediate Past President, for his leadership and his interest and involvement in all CAPG's activities
- Pierre Delage, Chair of the TOC, for invaluable assistance both in the development of the global survey, and for being the focal point for communication with the TCs
- Sam Mackenzie of GHD for his excellent work of developing and deploying the survey tool
- Jennifer Nicks, Chair of the Young Member Presidential Group for support in reducing complexity of the survey data for use by the technical committees
- all the ISSMGE Technical Committees for participating, framing of survey questions and analysis of results
- and, finally, to the ISSMGE members who participated in the survey.

7. REFERENCES

Day, P.W. (2017). Terzaghi Oration: Challenges and shortcomings in geotechnical engineering practice in the context of a developing country. In Proc. 19th Int. Conf. on SMGE, Seoul, pp 11-34.

Delage P. (2017). Report of the Technical Oversight Committee on the term 2013 – 2017. In Proc. 19th Int. Conf. on SMGE, Seoul, Korea

TC205/TC304 (2017). Joint TC205/TC304 Working Group on 'Discussion of statistical/reliability methods for Eurocodes, Final Report'. Available at: http://bit.ly/2EA65Mr.

Name the Geo's Competition Results

ENTRIES FOR THE "Name the CPP geo's" competition were a bit sparse. This is probably not too surprising as you really need to be in the picture to have much idea of who we all were back then – there are a few cosmetic changes with time!

David Stewart, now WSP-Opus scored 100%. Good one David.

Bruce Riddolls was next best. He didn't do too well with the summer students (Richard Justice, Virginia Cunningham, Mark McKenzie)

We won't shame the also-rans.

Back Row: Richard Justice, Richard DeLuca, David Stewart, Jeff Bryant, Royden Thomson, David Barrell, Graeme Halliday, Peter Brooks, Bruce Riddolls, Dean Fergusson, Virginia Cunningham
Front: Tim Coote, Glen Coates, Neil Crampton, Guy Grocott, Dick Beetham, Don Macfarlane, Gary Smith, Peter Wood, Mark McKenzie, Charlie Watts

NZGS Awards Calendar

NZGS SCHOLARSHIPS (ANNUAL)

Important dates are:

23 Nov 2018	Closing date for applications
Feb 2019	Successful applicants announced (if any)

To date, 3 applications have been received

NZGS STUDENT AWARDS POSTER COMPETITION (ANNUAL)

Important dates are:

Mid Dec 2018	Closing date for applications
End Jan 2019	Closing date for receipt of poster
Feb 2019	Judging and announcement of winners at local NZGS evening event

NZGS GEOMECHANICS AWARD

Christopher R. McGann, University of Canterbury
Brendon A. Bradley, University of Canterbury
Merrick L. Taylor, Arup
Liam M. Wotherspoon, University of Auckland
Misko Cubrinovski, University of Canterbury for their paper "Development of an empirical correlation for predicting shear wave velocity of Christchurch soils from cone penetration test data," (published in Soil Dynamics & Earthquake Engineering, 75, 66-75, 2015).

NZGS-7ICEGE SCHOLARSHIPS

Important dates are:

25 Nov 2018 Closing date for submission of full papers
Feb 2019 Successful applicants announced

Eight (8) applications were received.

Rolando Orense

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

r.orense@auckland.ac.nz

NZGS 2018
STUDENT
PRESENTATION
AWARDS
POSTER
COMPETITION

INVITATION TO PARTICIPATE

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

The 2018 Student Presentation Awards will be a Poster Competition and is open to all students.

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

14 DECEMBER 2018

International Association for Engineering Geology and the Environment

Mark Eggers

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

Doug Johnson

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

OUTCOMES OF IAEG EXECUTIVE & COUNCIL MEETINGS 2018

The Executive Committee meetings were held on 14 and 15 September 2018 and the annual Council Meeting on 16 September in conjunction with the 13th IAEG Congress in San Francisco. Key points to note for the Committee and Council meetings are summarised below.

• **Membership** As of 24 August 2018, membership details reported to the Secretary-General for 2018 from 41 of the 61 National Groups totalled 4105 members with China (591), Germany (509), New Zealand (422) and Australia (296) the largest of the national groups. New Zealand has been consistently the third largest National Group for at least the last four years with Australasia the third largest VP region with about 18% of the global membership. Not a bad effort considering we only comprise two National Groups.

• **Financials -** The IAEG continues to operate on sound footing with essentially a neutral budget for 2017, 2018 and 2019. The shift to digital publication of the bulletin reduces the cost of publication and this is reflected in reduced fees to members with the bulletin. As a reminder for members of IAEG, through NZGS the 2019 membership will be €32 with bulletin and €20 without.

• **Strategic plan** – Further work has been developed on the vision and mission statements together with the long term objectives for development of a programme of work to incorporate the specific objectives associated with the technical commissions, management committees and other activities of the Association. These are to be addressed by the new Executive.

• **New National Groups** - Both Croatia and Iceland have reactivated their National Groups in 2018 which were accepted by Council in San Francisco. Bhutan and Myanmar are exploring the formation of new National Groups and support is being provided to Iran.

• **Awards** – the following awards were voted by the Executive in Paris for announcement at the Congress in San Francisco:

- Hans Cloos Medal – Prof Runqiu Huang from China
- Marcel Arnould Medal – Louis Primel from France
- Honorary Membership – Cristian Marunteanu from Romania

• **Honorary President** – Prof Ricardo Oliveira from Portugal was nominated by Scott Burns (as current President) as Honorary President. This was voted and passed by Council in San Francisco. Ricardo is only the second person in the 54 year history of the Association to be made Honorary President, along with Prof Marcel Arnould. This appointment is reserved for people of distinction in recognition of extraordinary and long term merit resulting in prosperity and development of the Association. This honour is restricted to very rare and outstanding cases.

• **Richard Wolters Prize** – four candidates competed for the Richard Wolters Prize in San Francisco. This award specifically recognises meritorious scientific achievement by a younger member of the engineering geology profession. It is awarded based on the candidate's CV, best three published papers and a talk given at the Congress. The winner of the award for 2018 is Wei-An Chao from Chinese Taipei as voted by a judging panel chaired by Prof

Ricardo Oliveira. Sarah Bastin, from Canterbury University New Zealand, second.

ELECTION OF OFFICERS FOR 2019-2022 -

The following people were voted in at the Council meeting in San Francisco:
President: Rafig Azzam (Germany)
Secretary-General: Faquan Wu (China)
Treasurer: Jean-Alain Fleurisson (France)
Vice-President for Africa: Tamunoene Kingdom Simeon Abam (Nigeria)
Vice-President for Asia (2 positions): Tand Huiming (China) and Bo-An Jang (Korea)
Vice-President for Australasia: Doug Johnson (New Zealand)
Vice-President for Europe (2 positions): Eugene Voznesensky (Russia) and Vassilis Marinos (Greece)
Vice-President for South America: Norberto Jorge Bejerman (Argentina)
Vice-President for North America: Jean Hutchinson (Canada).

Prior to the election Council voted for the addition of a second Vice-President for Asia in recognition of the substantial growth of the National Groups in this region over the last 5-10 years. Hence both Europe and Asia are now represented by two VP's in the Executive. Comments on the Presidential election are made separately below.

• **Staggered VP terms** – the current Executive had agreed on the need to have staggered terms for the VP's so that a majority of the Executive is not 'turned over' every four years at the same time. However, this change was too late to be implemented for the election in San Francisco and will be taken up by the new Executive in 2019.

• **Digitisation of IAEG proceedings and publications** – Martin Culshaw (current Editor-in-Chief of the Bulletin) is investigating the cost and process of converting all paper copies of historic IAEG publications and making these available to members via the website. Some of this will be digitised in the UK and some in India. This project is ongoing.

• **Bylaws and Statutes** – a sub-committee of the Executive has been investigating updates and changes to the Bylaws and Statutes. The recommendations will be taken up by the new Executive in 2019.

• **Bulletin of Engineering Geology -** Martin Culshaw steps down as Editor-in-Chief of the Bulletin at the end of 2018 after 6 years in the role. Louis Wong from Hong Kong has already started as a new Editor-in-Chief as from the start of 2018 and Resat Ulusay from Turkey will step in as the second Editor-in-Chief from January 2019. Bulletin has an editorial board of 78 members. Currently CY Chin and Ann Williams are the only board members listed as coming from New Zealand. **Please note from 2019 the Bulletin will be online only.** In 2019 it will comprise 8 parts rather than the usual 4 to clear a backlog of papers that has accumulated.

• **Next Congress** – Council ratified the Executive recommendation that the next Congress due in 2022 will be held in Chengdu, China.

13TH IAEG CONGRESS 2018

See separate report from Pedro.

VICE-PRESIDENT FOR AUSTRALASIA AND AUSTRALIAN IAEG LIAISON

Doug Johnson of T+T is the new IAEG Vice-President for Australasia, replacing Mark Eggers whom we thank for his outstanding contribution. The AGS will nominate a Liaison to work beside Doug for the term 2019 to 2022. This appointment will be made at the AGS National Committee meeting to be held in Adelaide on 2 November 2018. Hence the successful candidate will be known by the time of the NZGS management committee meeting on 4 December 2018.

Doug Johnson
NZ IAEG Representative

International Society for Rock Mechanics and Rock Engineering

This report mainly covers ISRM-related information from the Board and Council meetings held in Singapore on 28 – 30 October 2018 in association with ARMS10 - the ISRM 10th Asian Rock Mechanics Symposium (http://www.arms10.org - 31 October – 03 November). 52 of the 61 ISRM National Groups were present or represented at the Council meeting. Outcomes, in particular those requiring a vote at the Council meeting, were:

8TH MULLER AWARD

Three nominations for the 8th Muller award, a recognition of distinguished contributions to the profession of rock mechanics and rock engineering and the most prestigious ISRM award, were received:

Erion Bukaci nominated by AGS, Albanian Geotechnical Society
Peter Kaiser nominated by CARMA, Canadian Rock Mechanics Association
Dick Stacey nominated by SANIRE, South African National Institute of Rock Engineering.

The Council vote resulted in Peter Kaiser receiving the award. The award lecture will be presented during the 15th ISRM Congress in Foz do Iguacu, Brazil in September 2019.

15TH ISRM INTERNATIONAL CONGRESS

Proposals to hold the 15th ISRM International Congress on Rock Mechanics and Rock Engineering in 2023 were received from:

Seoul, Korea
Salzburg, Austria

After formal presentations, including videos, to the Council meeting Salzburg was elected as the venue.

REVISION OF STATUTES AND BY-LAWS

Updates to the statutes and four by-laws (1, 2, 4 and 5) proposed by the Board, with an alternative update to By-law 2 proposed by the Australian National Group, were considered at the Council meeting. The Statutes, with a minor update to one of the clauses, and By-laws 1, 4 and 5 were approved during the meeting.

The update on By-Law 2, which resulted from the nomination process for the election of the 2019 – 2023 President at the 2018 meeting, required a vote on the two proposals with the Board update being approved. The issue related to the timing of nomination support and by whom with respect to announcement of all nominations by the Secretariat.

ROCHA MEDAL (2019 & 2020)

Twenty one theses for the 2019 award, recognising the most meritorious PhD thesis in rock mechanics, were received (0 from New Zealand, one from Australia). The award committee selected the following winner, announced during the Council meeting:

Lei Qinghua (China) with the thesis “Characterisation and modelling of natural fracture networks: geometry, geomechanics and fluid flow” (Imperial College, UK).

The award consists of the Rocha medal, a diploma and a cash prize. The award lecture will be given during the 14th ISRM Congress in Brazil. There was one runner up award for:

Wu Bangbiao (China) with the thesis “Dynamic tensile failure of rocks subjected to simulated in situ stresses” (Univ. of Toronto, Canada).

Nominations for the 2020 award are open and need to be with the ISRM Secretary General by 31 December 2018 (for evaluation in 2019). Further details are on the ISRM website, noting that a 10,000 word summary of the thesis needs to be prepared and ISRM membership demonstrated.

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

COMMISSIONS

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

Commissions run on a voluntary basis and several are very active, with associated publications (e.g. blue and orange books for testing methods), some less so, with one having stopped in the last months with another currently suspending activities. The Technical Oversight Committee (TOC) - Doug Stead, chair, Stuart Read and Norikazu Shimizu reported to the Board and Council meetings giving an overview on activities over the last year concluding that most are performing creditably. The report will be included in the next ISRM News Journal (due out early 2019).

FINANCES

Two financial items were approved at the Council meeting:

2017 year: Profit of Euro (€)15,000 from income of Euro127,000 and expenditure Euro112,000. Pattern similar to 2016 year (income mainly from subscriptions, ISRM sponsored conference fees, with growth in the use of OnePetro, the digital online library, expenditure mainly €55,000 for Secretariat, €20,000 newsletters with an additional item of contribution to the Education Fund Euro €7,000).

2019 budget: Similar to normal operation with Euro134,000 income (€90,000 from fees) and Euro119,000 expenditure including Education Fund (€10,000).

MEMBERSHIP:

The ISRM currently has an all-time record of 8215 individual members, belonging to 61 National Groups. This represents an increase of 5% in the number of individual members over the last year, mainly from China and Russia. Europe and Asia have the greatest individual membership (>35%), with Asia growing steadily. The other regions including Australasia having ~6%. – currently 325 in Australia and 175 from New Zealand. There are 155 Corporate memberships, with four from Australia and none from New Zealand.

YOUNG PROFESSIONALS:

Fostering of younger members (under 35) is a recognised need, and being more actively promoted by some national groups, as is the case in New Zealand and Australia (e.g. YGP conference in Hobart), than others. It is also receiving increasing attention, including for students, in association with mainstream conferences, in particular at regional ISRM regional conferences and congresses (e.g. ARMS10 in 2018 in Singapore there was a student night, Rockbowl quiz and informal social quizzes).

Another recent initiative has been the Early Career Forum at regional ISRM conferences. Funded in part by the Education Fund, partly from the ISRM budget it gives six to ten young professionals from the regional opportunity to present papers in a mainstream session. The first occurrence was at AfriRock in Capetown in October 2017, followed by Eurock in St Petersburg in May 2018 and most recently at ARMS10 in Singapore. It is programmed at the Congress in Foz do Iguacu.

NEW GUIDELINES FOR AWARDS AND CONFERENCE PROCEEDINGS

The Board approved the guidelines for two new awards: the "Best Paper Award" at ISRM International and Regional Symposia and the "ISRM Technical Excellence Award" to be given every two years. Guidelines for "ISRM Science and Technology Awards" are being finalised and will be approved soon.

The Board also approved "Guidelines for the Proceedings of ISRM Sponsored Conferences". This is partly regarding paper formatting requirements for papers on the OnePetro database, and partly for indexing by Scopus and other refereed paper databases.

14TH ISRM INTERNATIONAL CONGRESS ON ROCK MECHANICS AND ROCK ENGINEERING

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

The call for abstracts, which closed in early November, has resulted in submission of over 700 abstracts (four from New Zealand, thirty from Australia). Paper allocation depends on a formula depending on factors such as past Congress papers, membership (NZ 1%, Australia 3.5% of papers). Abstracts are about to be evaluated through National Groups (myself for New Zealand, Sevda coordinating for Australia) with papers based on accepted abstracts due for review in early 2019.

Keynote speakers, which are based on geographic area representation, have been invited (Dr John Read on the geotechnical engineer in open pit slopes is the selection made by the Congress on behalf of Australasia – Chris Massey presented at the 2015 Congress). Further information on the Congress, which will also include a strong focus on student and young professional activities is available at www.isrm2019.com.

ISRM 2019 – 2023 TERM

My term as Australian Vice President will finish at the Foz do Iguacu Congress, and notice has been given by the Secretariat (14th Sep) that Vice Presidents for the 2019 – 2023 term will be elected at the Council meeting associated with the 2019 Congress. Nominations are due with the Secretariat by 15th March 2019 accompanied by National Group nomination, one page CV and 5 minute video.

Under the understanding between AGS and NZGS the Australasian Vice-President for the term will be selected by Australia, and New Zealand would support the Australian nominee.

ISRM ON-LINE LECTURES

One on-line lecture has been given over the last months:

23rd by Prof Maurice Dusseault (Canada) on 18th September on "Subsurface Geomechanics - challenges in naturally fractured rock masses".

The lectures are available on the ISRM website (ISRM online lectures - e.g. https://www.isrm.net/gca/index.php?id=1343 for Prof Stille).

The next lecturer is Dr Claudio Olallo (Spain).

COMMUNICATION

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

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

- ISRM Newsletter, which has been published quarterly since March 2008.
  Last issue No 43 in September 2018
- ISRM News Journal, now under the editorship of Dr José Muralha (Portugal)
  Last issue No 20 in Dec 2017

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

FedIGS (Federation of International Geo-Engineering Societies www.geoengineeringfederation.org/):

FedIGS is a grouping under which the three international societies that NZ Geotechnical Society affiliates to operates. Prof Xia-Ting Feng (China), president of ISRM 2011- 2015 was elected Chairman of FedIGS for the 2018 – 2022 term. The first meeting of the board was held during the IAEG Congress in San Francisco between 17 and 22 September 2018.

Three Technical Commissions operate under the FedIGS umbrella –

- JCT1 (Natural slopes and landslides) co-ordinated under IAEG
- JCT2 (representation of geo-engineering data) co-ordinated under ISRM
- JTC3 (Education and training) co-ordinated under ISSMGE

The JCT2 committee is in a changeover period, with election of a new chair and members. Dr Ian Brown, as a New Zealand representative, has joined the committee. The 3rd International on Information Technology in GeoEngineering on 29 Sep – 02 Oct 2019 in Guimaraes, Portugal is a JCT2 activity (like long runout landslides conference in Hong Kong in December as JCT1 activity).

Stuart Read

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

DECEMBER 2018

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

1. FIRST YEAR OF PROGRESS FOR THE BOARD

It's been just over a year since the current President and Board were elected, and significant progress has been made towards implementing the President's plan for his four year term. I encourage you to read Professor Ng's summary of progress on the ISSMGE website. https://www.issmge.org/filemanager/article/580/Message-From-The-President-Report-On-First-Year.pdf

2. TECHNICAL COMMITTEES

The state of the current technical committees covers a wide range, from very active (like TC203, Earthquake) to essentially defunct (like TC210, Dams). A new TC309, Machine Learning, is being established.

Following a proposal from the Chinese geotechnical society, it was decided at the Skopje board meeting to reactivate TC210 Dams and Embankments. This committee will be renamed Embankment Dams, and will be chaired by Professor Limin Zhang. A new TC on tailings dams is proposed, with the lead on that being taken by the Vice President for South America, Alejo Sfriso.

A review of the activity and effectiveness of the TCs is due, and will be reported at the next Council meeting (September 2019). It is timely that representation on the TC's is refreshed. To this end, progress is being made by each of our Societies to revitalise our involvement in the Technical Committees by opening up opportunities for new nominees and corresponding members. Graham Scholey and I are leading this, and ISSMGE members will have recently seen a call for new TC members. We appreciate the interest that has been shown in this valuable opportunity. NZGS nominations have been finalised, and I expect the AGS to follow in due course.

The NZGS nominations are as follows:

- TC101 Laboratory Testing - Gabriele Chiaro, University of Canterbury
- TC103 Numerical Methods - Ioannis Antonopolous, Coffey, Dr CY Chin, Beca
- TC104 Physical Modelling - Phil Robins, Beca
- TC105 Geo-mechanics - Associate Professor Rolando Orense, University of Auckland
- TC203 Earthquake Geotechnical Engineering - Professor Misko Cubrinovski, University of Canterbury, Dr Sjoerd Van Ballegooy, Tonkin and Taylor. Ioannis Antonopolous, Coffey, Associate Professor Rolando Orense and Professor Michael Pender, University of Auckland, join this committee as Corresponding Members
- TC207 Soil Structure Interaction - Ioannis Antonopolous, Coffey
- TC208 Slope Stability - Ross Roberts, Auckland City
- TC210 Embankment Dams - Dr James Burr, Beca
- TC211 Nidhal Al-Alusi, Mott Macdonald (Corresponding Member)

Gavin Alexander

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

- TC211 Ground Improvement - Phil Clayton, Beca, Dr Martin Larisch, Miyamoto International
- TC212 Deep Foundations - Dr Martin Larisch, Miyamoto International
- TC213 Scour and Erosion - Professor Bruce Melville, University of Auckland
- TC217 Land Reclamation - Tony Fairclough, Tonkin and Taylor
- TC309 Machine Learning - Vick Kumaran, Hiway Geotechnical

It is pleasing to have increased TC representation from the NZGS from five to nineteen Nominated or Corresponding Members.

I would like to thank those of you who have represented us on these committees in the past and who are now stepping down. I would also like to thank our new nominees for your enthusiasm, and hope that you will get great value from your involvement in these specialist committees. Please do remember to provide updates and feedback to your home society, so we can all share in and benefit from the work that the TCs undertake.

3. CONFERENCES

Paper reviews for the next ANZ conference, to be held in Perth in early April 2019, will be complete by the time you read this. This is the first of the next round of ISSMGE Regional Conferences, which are traditionally held between the four yearly international conferences. I look forward to meeting many of you in Perth.

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

4. FORTHCOMING BOARD MEETINGS

The next Board meetings are scheduled for 18 November (face to face, Mexico City) and 10 March 2019 (face to face, Singapore).

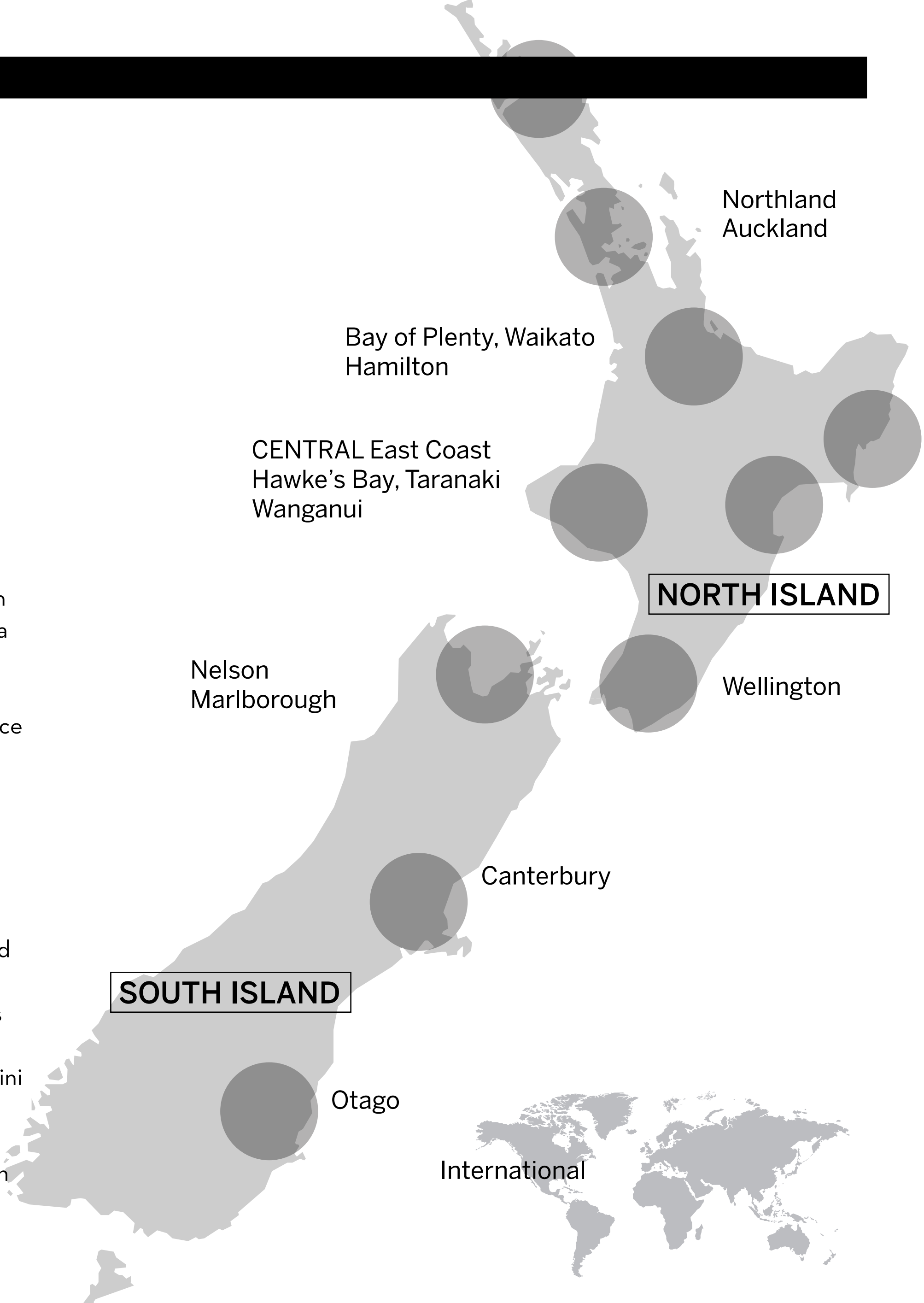
For more information about the ISSMGE, please visit www.issmge.org

Finally, we'd like to wish you and your families all the best for Christmas and the holiday season. We hope you manage to take a well-earned and enjoyable break.

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

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

Branch reports



AUCKLAND

Quality talks have kept Auckland engaged over the second half of 2018. In July Mike Dobbie kindly presented on Load Bearing Bridge Abutments Supported on Polymer Geogrid Reinforced Fill. At the start of September the branch hosted the NZGS AGM after a presentation from Richard Kelly on the the Ballina Bypass Prediction Exercise Sydney. Later in the month a large crowd of around 200 people was in attendance at the 2017 Rankine Lecture titled "Triggering & Motion of Landslides" given by Professor Eduardo Alonso. In October Professor R. Kerry Rowe gave his very valuable 2017 Terzaghi Lecture on Protecting the Environment with Geosynthetics and Dr Andrew Lees enlightened us on FE modelling of polymetric geogrids for stabilising soils. At the end of November we are trialing a Xmas Mini Symposium for local practitioners to present on local topics and to sit down for dinner. A final presentation will be held in December on the Transient Surface Deformations Caused by the Gotthard Base Tunnel presented by Simon Löw. The Auckland branch coordinators hope everyone has enjoyed the year of presentations and thank you all for your attendance. We greatly appreciate the support of Geofabrics, Ground Investigation Limited, Geotechnics and Beca who have supported the talks and the society.

WAIKATO

On 26th June Phil Clayton - Principal and Technical Director in Geotechnical Engineering with Beca Limited presented on Some approaches to liquefaction assessment in Waikato soils in a near capacity University of Waikato lecture theatre. In this talk, Phil drew from his broad experience in seismic geotechnical engineering including input to the original NZGS Module 1 (now module 3), Module 5a and 6, guidance on the seismic design of retaining structures for residential sites in Greater Christchurch as well as the NZSEE/MBIE guide to the seismic assessment of existing buildings.

The presentation introduced several related papers on liquefaction assessment undertaken for the Hamilton Section of the Waikato Expressway submitted to the 2017 NZGS conference. Of interest to many who work in the rhyolitic soils of the Waikato, this project utilised shear wave velocity based liquefaction triggering assessment methods. This approach was supported by detailed crosshole and downhole methods and a program of paleoliquefaction investigation. Also discussed is the application of unit specific Ic cutoff and fines correction parameters that could be extended for wider use in the region.

Thanks to Vicki Moon and Willem De Lange for organising the U of W facilities and thanks to HD Geo for sponsoring the event and the gourmet catering!

FUTURE WAIKATO BRANCH EVENTS

We have the managed to secure Prof Misko Cubrinovski and the New Zealand Geomechanics Lecture to come to Hamilton early in 2019 – date is to be confirmed.

Liquefaction of Waikato soils is still a hot topic and we are working on some more society events around this – watch this space...

WELLINGTON

The Wellington branch has had a busy couple of months with more to come. In September Dr. Richard Kelly presented on the Ballina Bypass Prediction Exercise in Sydney after teaching a short course with Nick Wharmby on Design and Construction on Soft Soil. Also in September we were honoured to host Prof. Eduardo Alonso for the 57th Rankine Lecture. Prof. Alonso presented to a group of 30 (approx.) on the Triggering and Motion of Landslides and gave some insightful ideas about creep mechanism and new ways of numerically modelling landslides.

In October we also hosted two presentations. First, Prof. Kerry Rowe presented the 53rd Terzaghi Lecture. Prof. Rowe presented on Protecting the Environment with Geosynthetics. He gave insight into key factors affecting the design life of geosynthetics and garnered some robust discussion from the crowd of approximately 30. We also hosted Dr. Andrew Lees from Tensar International who presented on Finite Element Modelling of Geosynthetics. Dr. Lees discussed how geosynthetic and soil components act as a single interlocked material in vertical load applications.

To close out 2018 we have planned a presentation by Dr. Martin Larisch of Miyamoto International NZ on Effective Risk Management for Piles and Deep Foundations (21 November), and a presentation by Prof. Simon Loew from the University of Zurich on Monitoring and Early Warning of Potentially Catastrophic Rockslides (4 December).

For 2019 we are planning a site visit to Transmission Gully and Marco Holtrigter has offered to present on the CETANZ CPT Industry Best Practice Guidelines."

HAWKES BAY

Members of the branch recently met with the local ACENZ group and Hawkes Bay Civil Defense Emergency Management to discuss initial emergency response. Further meetings between local professional groups has been flagged to network the limited number of local professionals with a common interest.

Sirini De Silva has taken up the mantle as co-coordinator of the Hawkes Bay Branch. She replaces Tom Bunny, to whom we owe thanks to for his contribution over the last 18 months.

As always, we're interested in any suggestions from members.

★ GEO-NEWS WEEKLY E-NEWSLETTER ★

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

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

NELSON

We are hoping to have a presentation on learnings from Kaikoura EQ sequence at our next meeting. As always, please let us know any suggestions of potential Presentations or site visits that will be of interest to our members.

CHRISTCHURCH

Over the last few months we have very busy and had 5 presentations with visiting international professors and another training session, turn out has been very good at these presentations (ranging from 25-60 people) and well received by members. Coming up we have quite a busy schedule of presentations to the end of the year.

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

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

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

NORTHLAND

Philip Cook

I am a Chartered Professional Engineer. I have an interest in risk assessment, landslides, Northland Allochthon geology, liquefaction, and seismic assessment for earthquake resistant foundations, foundation settlement.

Look forward to improving the geotechnical features of soils in Northland. Enjoy the coastal lifestyle of Northland

phil@coco.co.nz

AUCKLAND

Eric Torvelainen

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

ETorvelainen@tonkin.co.nz

James Johnson

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

James.Johnson@beca.com

Christopher Wright

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

cwright@riley.co.nz

WAIKATO

Kori Lentfer

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

koril@cmwgeosciences.com

Andrew Holland

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

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

Andrew@hdc.net.nz

BAY OF PLENTY

James Griffiths

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

James.Griffiths@beca.com

Kim de Graaf

Kim is a Geotechnical Engineer with Beca and is based in Tauranga.

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

kim.degraaf@beca.com

HAWKE'S BAY

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

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

WELLINGTON

Aimee Rhodes
Aimee is a graduate geotechnical engineer with Opus. She recently completed her Masters degree in Earthquake Engineering with the University of Canterbury. Aimee has experience with liquefaction analysis and soil characterisation having worked on modelling liquefaction in stratified soils for her Masters research.
aimee.rhodes@opus.co.nz

Shirley Wang
Shirley is a Geotechnical Engineer with 8 years of experience working at Tonkin & Taylor Wellington Office. She graduated from Canterbury University with a BE(Hons) in 2009. She has experience in seismic assessment, geotechnical and environmental investigation, slope stability, foundation design and construction monitoring.
SWang@tonkintaylor.co.nz

Jerry Spinks
Jerry is a chartered professional engineer who has worked in New Zealand and the UK. Returning to New Zealand in 2011, he has worked on a variety of building projects. Jerry recently joined Jacobs Engineering, where he has been undertaking a number of landslide assessments and is working on the Pinehaven Flood Protection Scheme.
Jerry.Spinks@jacobs.com

NELSON

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

CANTERBURY

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

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

CANTERBURY

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

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

OTAGO

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

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

NEW ZEALAND GEOTECHNICAL SOCIETY INC

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

The aims of the Society are:

a) To advance the education and application of soil mechanics, rock mechanics and engineering geology among engineers and scientists.
b) To advance the practice and application of these disciplines in engineering.
c) To implement the statutes of the respective international societies in so far as they are applicable in New Zealand.
d) To ensure that the learning achieved through the above objectives is passed on to the public as is appropriate.

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

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

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

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

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

Teresa Roetman

Since the last issue of NZ Geomechanics News we have a steady stream of new members joining. As members you are entitled to reduced registration fees of all NZGS courses. We also offer students free membership! A new membership form is available on our website www.nzgs.org As members, if you join the website you have full access to our library which is continually expanding as well as recorded Presentations which will soon be increasing for those outside of the main centres. As new ones become available we shall advertise these into our weekly emails and the Linkedin page under Management Secretary.

I wish you all a happy holiday season and a prosperous New Year!

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

Management Committee 2018-2019

POSITION	NAME	EMAIL
Chair	Tony Fairclough	chair@nzgs.org
Vice-Chair & Treasurer	Ross Roberts	treasurer@nzgs.org
Immediate Past Chair	Charlie Price	price.charlie@outlook.com
Elected Member	Kevin Anderson	Kevin.Anderson2@aecom.com
Elected Member	Camilla Gibbons	Camilla.Gibbons@aurecongroup.com
Elected Member	Eleni Gkeli	eleni.gkeli@wsp-opus.co.nz
Elected Member	Rolando Orense	r.orense@auckland.ac.nz
Co-opted NZ Geomechanics Editor	Don Macfarlane	editor@nzgs.org
NZ Geomechanics Editor Co-editor	Gabriele Chiaro	gabriele.chiaro@canterbury.ac.nz
Co-opted YGP Representative	Pip Mills	Pip.mills@outlook.co.nz
IAEG Australasian Vice President (until January 2019)	Mark Eggers	Mark.Eggers@psm.com.au
IAEG NZ Representative (Taking over Australasian VP after January 2019)	Doug Johnson	DJohnson@tonkintaylor.co.nz
ISSMGE Australasian Vice President	Gavin Alexander	Gavin.alexander@beca.com
ISRM Australasian Vice President	Stuart Read	S.Read@gns.cri.nz

EDITORIAL POLICY

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

Readers are encouraged to submit articles for future editions of NZ Geomechanics News. Contributions typically comprise any of the following:

- **technical papers which may, but need not necessarily be, of a standard which would be required by international journals and conferences**
- **technical notes of any length**
- **feedback on papers and articles published in NZ Geomechanics News**
- **news or technical descriptions of geotechnical projects**
- **letters to the NZ Geotechnical Society or the Editor**
- **reports of events and personalities**
- **industry news**
- **opinion pieces**

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

Articles and papers are not normally refereed, although constructive post-publication feedback is welcomed. Authors and other contributors must be responsible for the integrity of their material and for permission to publish. Letters to the Editor about articles and papers will be forwarded to the author for a right of reply. The editors reserve the right to amend or abridge articles as required.

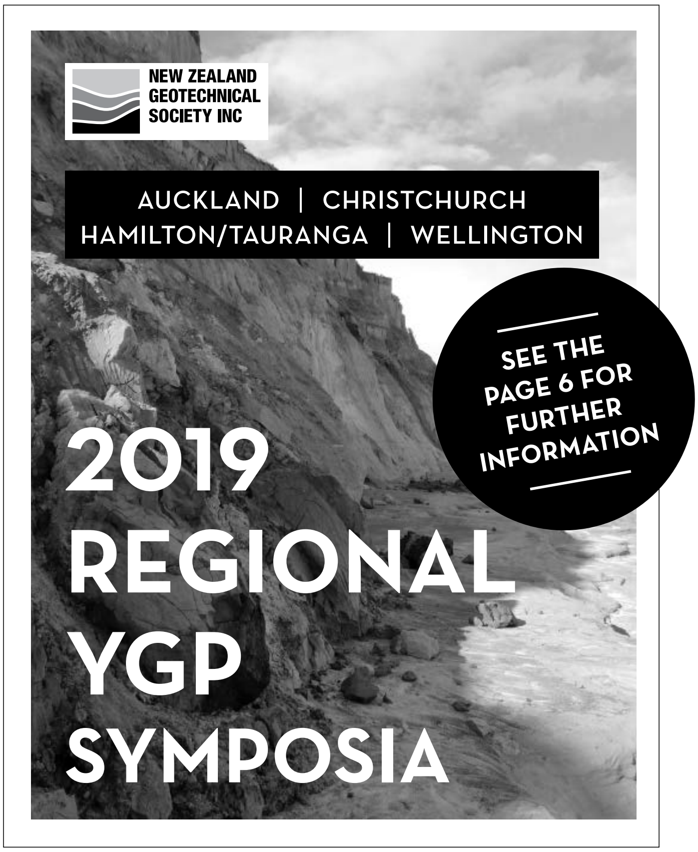
The statements made or opinions expressed do not necessarily reflect the views of the New Zealand Geotechnical Society Inc.

NEW ZEALAND GEOTECHNICAL SOCIETY INC

NZGS Membership SUBSCRIPTIONS

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

Advertisers' Directory



ADVERTISING

NZ Geomechanics News is published twice a year and distributed to the Society's 1000 plus members throughout New Zealand and overseas. The magazine is issued to society members who comprise professional geotechnical and civil engineers and engineering geologists from a wide range of consulting, contracting and university organisations, as well as those involved in laboratory and instrumentation services. NZGS aims to break even on publication, and is grateful for the support of advertisers in making the publication possible.

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

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

Notes
1. All rates given per issue and exclude GST
2. Space is subject to availability
3. A 3mm bleed is required on all ads that bleed off the page.
4. Advertiser to provide all flyers
5. Advertisers are responsible for ensuring they have all appropriate permissions to publish. This includes the text, images, logos etc. Use of the NZGS logo in advertising material is not allowed without pre-approval of the NZGS committee.

National and International Events

2019

22-24 January
Villars-su-Ollon, Switzerland
Winter School "Geomechanics for Energy and the Environment"

25-28 March
Hurghada, Tanta, Egypt
International Conference on Advances in Structural and Geotechnical Engineering

1-3 April
Perth, Australia
13th ANZ conference on Geomechanics (ANZ2019)

7-11 May
Okinawa, Japan
2019 Rock Dynamics Summit

11-13 April
Omis, Croatia
8th Conference of Croation Geotechnical Society with International Participation – Geotechnical Challenges in Karst.

3-5 June
Prague, Czech Republic
14th International Conference "Underground Construction Prague 2019"

17-20 June
Rome, Italy
7ICEGE International conference of Earthquake Geotechnical Engineering

10-13 June
Colorado, SA
7th International Conference on Debris Flow Hazards Mitigation

1 July
Rykjavic, Iceland
ECSMGE 2019 – XVII European Conference of Soil Mechanics and Geotechnical Engineering

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

29 Sept - 2 October
Guimaraes, Portugal
3rd International Conference on IT in Geotechnical Engineering (3rd ICITG2019)

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

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

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

17-20 November
Cancun, Mexico
XV1 Pan-American Conference on Soil Mechanics and Geotechnical Engineering

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

2020

1-3 April 2020
Perth, Australia
Slope Stability ANZ2020 Symposium

June 2020
Trondheim, Norway
EUROCK2020

29TH June-1ST July
Cambridge, England
TC204 Geotechnical Aspects of Underground Construction in Soft Ground

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

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

7 –11 September
Budapest, Hungary
6th International Conference of Geotechnical and Geophysical Site Characterization

13-16 October
Oxen Hill, US
45th Annual conference on Deep Foundations

2-6 November
Kyoto, Japan
Fifth World Landslide Forum

2021

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

GEOTECHNICS

Smarter and Safer - The Reliable Service Clearance Solution

Avoiding a service strike is a high priority for all of us in the field. The personal injuries, let alone potential loss of life, delays and expensive repairs are simply not worth the risk.

That's why here at Geotechnics, we utilise the latest Ground Penetrating Radar technology and Electromagnetic tools. This allows us to locate cables, pipes and all utilities; gathering subsurface data through images without destroying the surroundings.

Ground Penetrating Radar (GPR)

- A fast and non-destructive geophysical tool that sends electromagnetic pulses into the ground to obtain data about sub-surface features
- Able to locate concrete, plastic, metal and asbestos pipes, cables, voids, man-made objects, and other buried structures

Electromagnetic Tools

- Used in conjunction with a GPR, these tools are able to locate gas pipes, power cables, communication conduits, and steel water pipes by inducing a current onto the tracer wire or pipe

Hydro-exavation to Expose Services

- Where exact service depths are crucial, potholing and service-trenching by hydro-excavation is the safest method we can use

Because this comes second nature to us, our specialist, NULCA-accredited team can take all of the hassle out of determining what needs to be done to comprehensively locate utilities on your site.