NEW ZEALAND GEOTECHNICAL SOCIETY INC

DECEMBER 2020 issue 100

NZ GEOMECHANICS NEWS

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CELEBRATING 100 ISSUES

CLIMATE CHANGE AND GEOTECHNICS

SUMNER ROAD REBUILD

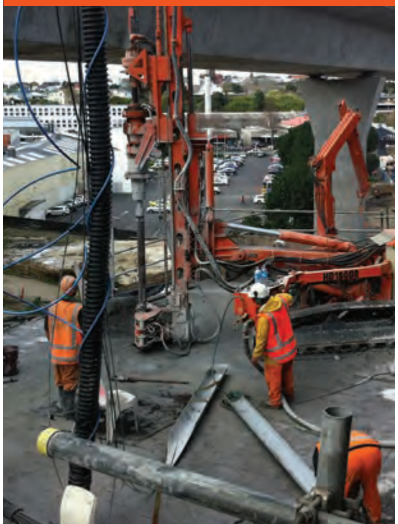
NSHM FRAMEWORK UPDATE

PHOTO COMPETITION WINNERS

RETAINING YOUR BUSINESS IS OUR BUSINESS

Design

Construct



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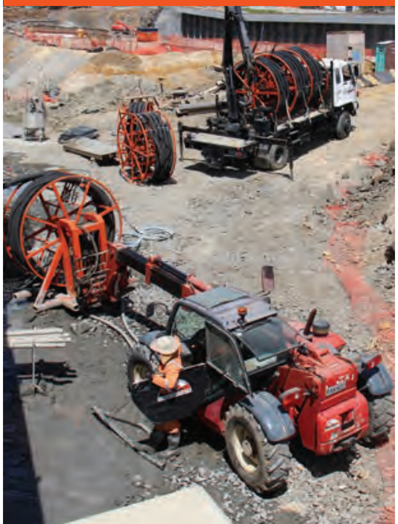

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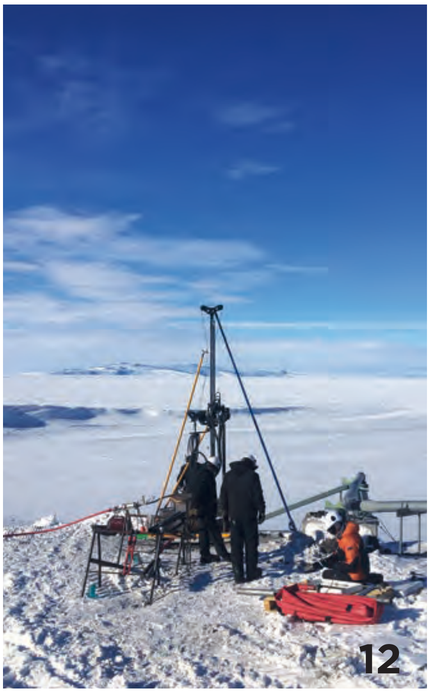

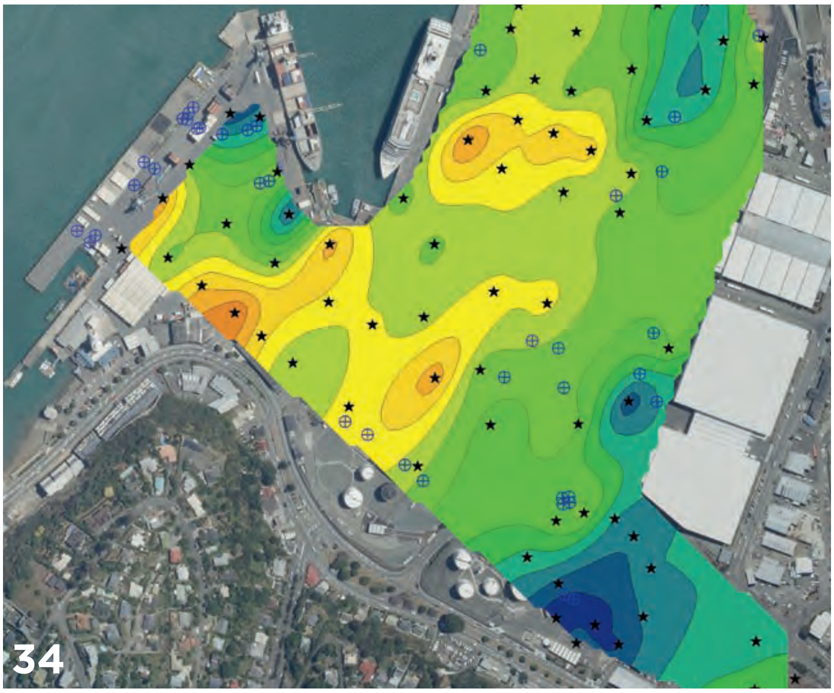

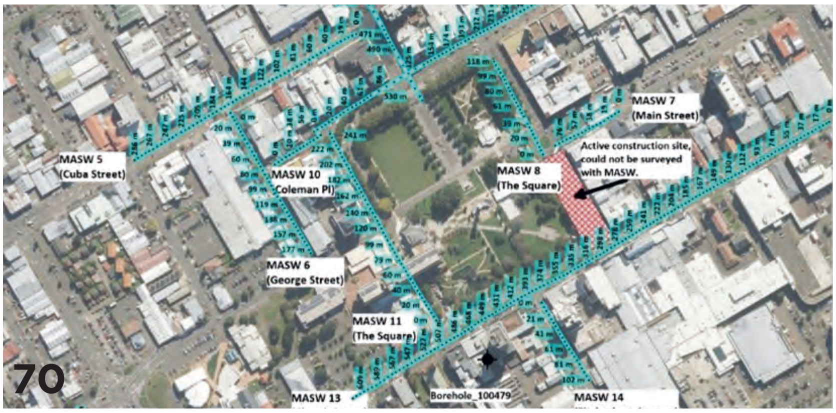

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COVER IMAGE: Lukas Janku, KiwiRail: Twins

From the Chair

Ross is Auckland Council's geotechnical and geological practice lead, which involves managing all aspects of geotechnical and geological risk. This ranges from emergency management and geohazard studies to geotechnical design, standards and policy.

Ross Roberts
Chair, Management Committee

FIGURE 1 – Gavin Alexander receiving his NZGS life membership, presented to honour the huge impact he had on the society

GAVIN ALEXANDER

I am very sorry to share with you that our former Chair and NZGS Life Member, Gavin Alexander, passed away on 19th November 2020 after a battle with cancer, surrounded by his family.

Gavin served on the NZGS management committee for a decade taking on the roles of Treasurer, Chair and more recently the ISSMGE Vice President for Australasia.

During Gavin's tenure as Chair the NZGS was awarded the Outstanding Member Society of the International Society for Soil Mechanics and Geotechnical Engineering in recognition of the impact that the Society was having under his stewardship.

He also took a leading role in managing the Earthquake Geotechnical Engineering Practice Modules, guiding their delivery in a way that has resulted in a very significant improvement in performance across the geotechnical discipline.

Gavin's calm, warm and collaborative approach made him a great leader who was always a pleasure to work with. I am particularly grateful for the advice and support he gave me, and no doubt also many of you, over the years.

21ST NZGS SYMPOSIUM

Planning for the 21st NZGS Symposium, which was to be held at the Dunedin Conference Centre in October 2020, has been severely disrupted by the travel limitations imposed across the country. The technical content of the symposium is very strong, and registrations have been good, so a decision was made to avoid cancellation and to instead postpone to March 2021, when there is a significantly improved chance of successfully hosting a face-to-face event that will truly celebrate the great work of the authors.

Helen Hendrickson

Miles Buob

Pedro Martins

Stuart Read

John Scott

We owe a debt of gratitude goes to the hard-working symposium committee who have put in a huge effort developing multiple parallel options for hosting the event (including online options):

- Eleni Gkeli (WSP, Convenor)
- Phil Robins (BECA, Technical Coordinator)
- Doug Mason (WSP, Finances, Technical Subcommittee)
- Ayoub Riman (ENGEO, Technical subcommittee, Soil Structure interaction plenary session)
- Chris Sandoval (Tonkin and Taylor, General Support, YGP and Public Lecture)
- Nathan Schumacher (Freelance, Sponsorship)
- Helen Hendrickson (WSP, General support, Sponsorship)
- Stuart Read (GNS), Don Macfarlane (AECOM), and David Stewart (WSP) - workshop and field trip subcommittee

Further information about the Symposium is available on our website and on the Symposium site https://www.confer.nz/nzgs2020/

CLIMATE CHANGE IN GEOTECHNICAL ENGINEERING

The Management Committee is currently considering running a special one-day mini conference focused on the implications of climate change to the geotechnical sector. We are in the process of developing the scope for this, and anticipate an event in May, June or July 2021. If you would like to be involved in this event, please contact committee member Jen Smith (JSmith@tonkintaylor.co.nz).

If you have any thoughts about how the geotechnical professions can better adapt to or mitigate the effects of climate, start preparing your paper now!

CHANGES ON THE MANAGEMENT COMMITTEE

Our ever-enthusiastic Young Geotechnical Professional Representative, Áine McCarthy, has stepped down from her role to take maternity leave. She has been extremely effective in her time on the committee, among other things developing a series of very successful YGP mini-symposia around the country. These events have been extremely well received, and we hope to continue them in the future.

After a very competitive selection process our new YGP rep has been selected. Helen Hendrickson will make a worthy successor to Áine, and the handover has already taken place. Because of the very impressive selection of candidates who put their names forward for these roles, we have also created some additional roles. The first of these YGP-specific roles (outside of the management committee) will be focused on development of YGP-centric training material. Miles Buob's enthusiasm for developing training meant that he was a natural candidate for this role.

We will also be creating three further YGP roles (also outside of the management committee), supporting our international society Vice Presidents, and we would like to encourage the unsuccessful candidates to apply for these roles. These roles are designed to link our YGPs more closely with the international societies, and to encourage more YGP activity in each of these groups.

This move is particularly important now because Pedro Martins has indicated his desire to step back from his role representing Australasia in the IAEG Young Engineering Geologists Committee. Pedro has been a fantastic link to our international society, and the success he has brought to this role has encouraged the NZGS committee to seek to replicate his role for the sister societies (ISSMGE and ISRM). We will be calling for expressions of interest soon.

NEW LIFE MEMBERS

I'm delighted to announce that two new life members were elected at our AGM in September. Life membership is an honour bestowed on our most esteemed members for their service to the NZ Geotechnical Society. Stuart Read and John Scott have been long-standing supporters of the NZGS, and this award is extremely well deserved.

John Scott has given a lot of support to the national management committee over a period of many years. Despite not being a committee member John has consistently gone beyond the core requirements of his roles at MBIE and EQC by actively reaching out to the NZGS and involving us in his work. He is a regular guest at committee meetings, providing advice and insights that have made the work of the management committee significantly easier and more productive.

John has been the driving force behind the New Zealand Geotechnical Database since its inception. His advocacy and tactful lobbying have been crucial to the database taking form and being accepted. His legacy is now a world-leading national database of huge value to all our members.

Stuart Read has given many years of exemplary service to the society over a period of four decades. He first joined the NZGS committee in 1979, and quickly became editor of Geomechanics News from 1980 to 1982. He was secretary of the society from 1982 until 1985, and went on to become one of the lead authors of the original 1988 field guide for the description of soil and rock.

For the past four years Stuart has represented New Zealand and Australia as the Australasian Vice President for the International Society for Rock Mechanics and Rock Engineering (ISRM). His support and advice to the NZGS committee has been very valuable, and his work developing and coordinating many training courses, particularly for engineering geologists, has benefitted many of our members. He has been involved in organising many of our conferences and symposia, including the current 21st NZGS Symposium.

As one of the original team involved in developing the Professional Engineering Geologist (PEngGeol) qualification he has made a material difference to the recognition of engineering geology as a profession in New Zealand.

ADVOCACY

The NZGS has continued to advocate for our members interests through our links to Engineering New Zealand and the Building Systems Performance Team in MBIE.

Of note since our last Geomechanics News is the new round of consultation started by Engineering New Zealand on professional registration. We will keep working with Engineering NZ for the best outcome possible for our profession and members, in concert with our sister societies (SESOC and NZSEE). Our feedback to date can broadly be summarised as supportive of the Engineering NZ proposals, with a desire to see the timeframes shortened.

A little further into the future I expect that there will be a significant volume of work related to the Building Code updates required for the new National Seismic Hazard Model and the "Building for Climate Change" programme. The new National Seismic Hazard Model is currently being developed by GNS Science and should be finished in 2022. We anticipate this will result in some changes to loadings in parts of the country, as well as a change of approach to site classification. The Building for Climate Change programme has the potential to significantly change construction in New Zealand and will require us to change our design processes to consider the emissions throughout the design life of a structure.

I represent NZGS on the MBIE Code Advisory Panel, so please contact me if you would like to know more about these.

CONCLUSION

Despite a most challenging year, most of the activities of the NZGS have continued successfully. I am particularly looking forward to our symposium in March and to seeing you all there.

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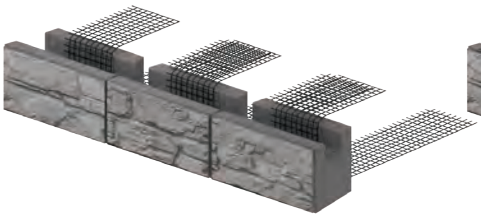

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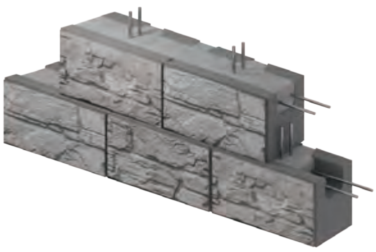

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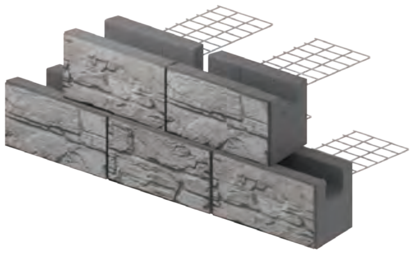

STEEL GRID

The MagnumStone steel grid solution provides designers with a great option.

SOIL NAILED

Soil nailing could be one of the fastest growing solutions for retaining wall structures.

Feed the world

IN OUR LAST issue we featured an article outlining current research into sea level rise and its likely effects in New Zealand and another offering ideas for reuse of old unwanted tyres. While our interest as engineers is focussed on the fun jobs associated with engineering works for the design and construction of coastal erosion/protection and landfills, the companion elephant in the room is population growth and all it associated implications for feeding, watering and sheltering increasing numbers of people and handling their waste. While New Zealand seems to be relatively isolated from these issues in the foreseeable future, this situation will not last forever. So what do we as engineers do about it???

Over the next decades mankind will demand more food from fewer land and water resources. Pressure on land and water supply will come from not only population growth, but also from technological change, forestry and agricultural demands and economic development. Population growth will increase food demand and, therefore, the demand for agricultural land, while increased agricultural intensity due to population growth will increase land degradation over time. And we will still be mining minerals and aggregates (and coal and oil). At the same time, along the coasts of many countries, productive and residential/industrial land will be lost to the oceans. Plus we will continue to generate increasingly large volumes of waste to dispose of – where??? Meanwhile, man will continue to overfish the seas and fill them with plastic. That wonderful stuff has only been around since the 1950's and look at the adverse effects it is now having!

But its not all bad. Society and world leaders (with a few prominent exceptions) now recognise that warming caused by greenhouse gasses is occurring across the globe and are facing up to its implications and effects. In this issue we continue to share the thoughts of key players working here in New Zealand. Our Feature Paper by Ross Roberts describes climate change effects where they have overlaps with geotechnical design and hazard assessment (with particular reference to Auckland as an example), discusses the impact that these changes are expected to have on geotechnical engineering practice in the coming years and decades, and presents a framework for managing these in the design processes.

Your responsibility as geotechnical professionals is to actively involve yourself in, and contribute positively to, the changes that are here and those that are on the way. Good for it!

Don and Gabriele

Don Macfarlane has worked as an applied engineering geologist for nearly 40 years and has accumulated some knowledge, a fair bit of wisdom and a few brickbats along the way.

His real interest is dams and associated issues (seismic hazard, slope instability) but any good geohazard affecting an engineering structure will do. These days he is a Technical Director with AECOM in Christchurch.

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

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

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


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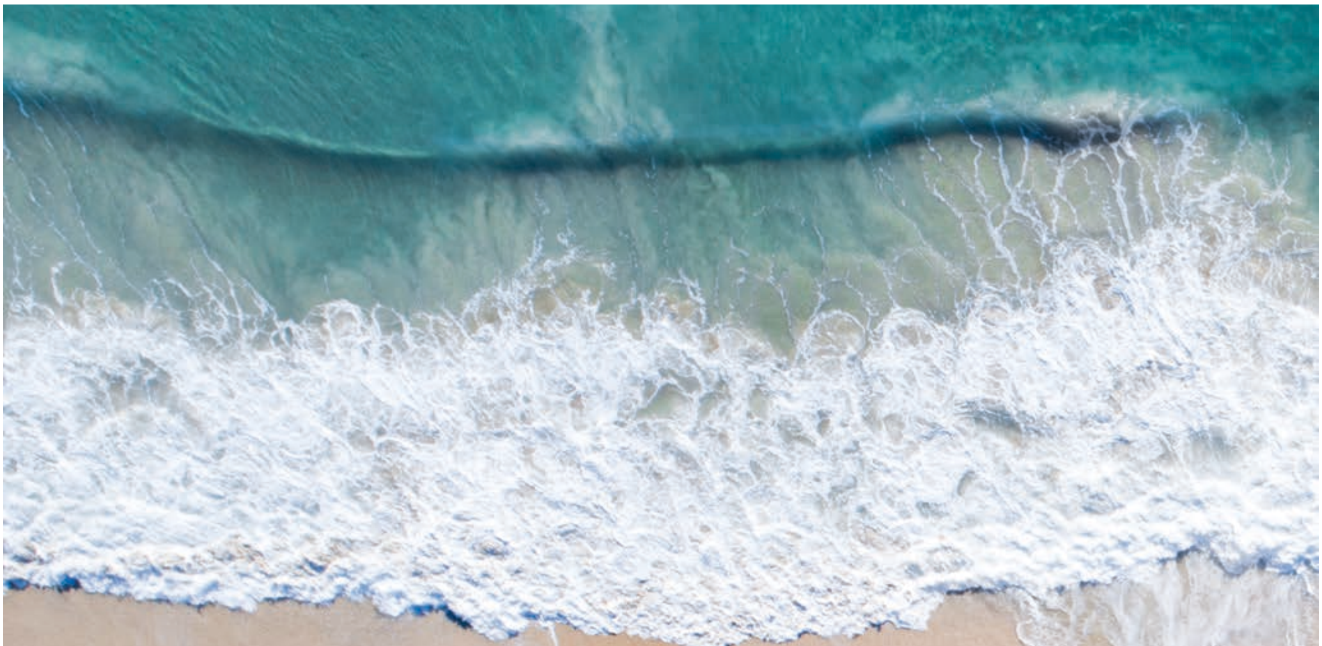
NEWS – IN BRIEF



75% of the World's sandy beaches are stable or growing

Analysis of satellite derived shoreline data indicates that 24% of the world's sandy beaches are eroding at rates exceeding 0.5 m/yr, while 28% are accreting and 48% are stable.

Coastal zones constitute one of the most heavily populated and developed land zones in the world. Despite the utility and economic benefits that coasts provide, there is no reliable global-scale assessment of historical shoreline change trends. By using freely available optical satellite images captured since 1984, in conjunction with sophisticated image interrogation and analysis methods, a global-scale assessment of the occurrence of sandy beaches and rates of shoreline change was undertaken. This found that 31% of the world's ice-free shoreline beaches are sandy. Analysis of satellite derived shoreline data for the 33-year period 1984–2016 indicated that 24% of the world's sandy beaches are eroding at rates exceeding 0.5 m/yr, while 28% are accreting and 48% are stable. The majority of the sandy shorelines in marine protected areas are eroding, raising cause for serious concern.

https://www.nature.com/articles/s41598-018-24630-6

Menard Oceania boosts its New Zealand 'front row'

In August 2020, international design and construction geotechnical specialist Menard Oceania announced that it is reinforcing its New Zealand 'front row' with the appointment of Doug Close as Bid and Project Manager.

Before joining Menard Oceania, Doug's six years in New Zealand have included the role of Operations Manager – Geotechnical, at March Construction and more recently he was Piling Manager at Fulton Hogan. Originally from Glasgow, Scotland, Doug spent the earlier part of his career working in the UK ground improvement sector.

Menard Oceania is a part of Soletanche Freyssinet, a group of world leaders in soil and structural engineering. This relationship further strengthens Menard's ability to manage complex integrated problems and continue as a leader and pioneer of specialist geotechnical contracting.





PHOTO COMPETITION WINNERS

WINNER

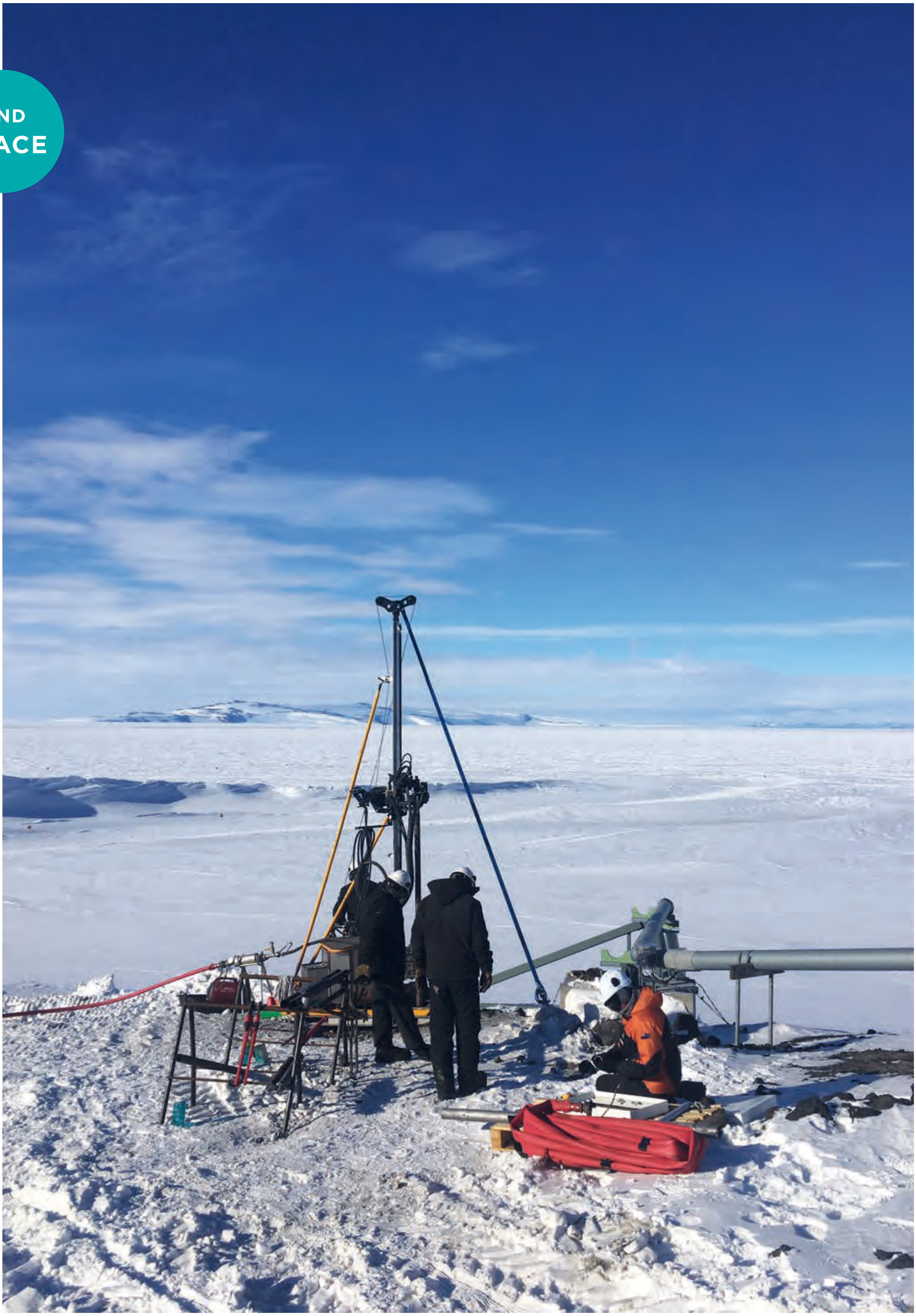
ONCE AGAIN WE had a great range of photos submitted, and a record number of entries for which we thank you very much. It's great to see the range of places we work, the things we see and the weird things that appeal to some of you.

Not surprisingly, with the large number of entries the judges had trouble agreeing the winners and in the end we doubled the size of the judging panel to try to get agreement. Then we added a scoring system... Then we finally came to a decision.

The winners of the 2020 Photo Competition are:

1ST PLACE

Lukas Janku, KiwiRail: **Twins**



2ND PLACE

2^{ND} PLACE **Tim McMorran, Golders: On Ice**
Geotechnical drilling investigations for the Scott Base Redevelopment Project. Photograph taken October 2019 by Matt Jordan, Antarctica New Zealand.

3^{RD} place **Christoph Kraus, Beca: A Road to Nowhere**
"A road to nowhere...at the north end of the Te Ore Ore landslide on State Highway 4"

4^{TH} Place **Michelle Willis, Earthtec: Cairns**
"Man-made mountains (alpine cairns in front of Mount Cook)"

5^{TH} Place **Lukas Janku, KiwiRail: Deep Reflections**

MBIE Bi-Annual E-Newsletter

BACKGROUND:

MBIE amended Acceptable Solution (B1/AS1) in November 2019 to acknowledge liquefaction prone-ground is not exclusive to Canterbury region and the requirements should therefore be extended to all of New Zealand. The amendment facilitates the design of house foundations to

- Ensure new buildings (especially residential buildings) are designed and built with the right level of resilience to appropriately manage liquefaction related risk with cost effective solutions.
- Reduce the likelihood of extensive and catastrophic foundation failures where liquefaction risk is high across New Zealand
- Provides clarity to both Territorial Authorities (TA's) and engineers ensuring new building foundations are being built safely and resilient enough to withstand risk on liquefaction-prone ground.

The change has a two year transition period starting from 29 November 2019 until 28 November 2021. The reason for this transition period is to allow TA's time to complete liquefaction vulnerability mapping[1.]

Due to COVID-19, TA's around New Zealand may have experienced delays and disruption preparing to or in the process of completing mapping of liquefaction-prone ground, while this also delayed communications from MBIE.

IMPACT OF GOOD GROUND DEFINITION CHANGE:

In essence, the change revokes the use of a 'deemed to comply' pathway for foundations unless the ground has been assessed and or categorised as not being liquefaction-prone – i.e. good ground. While there is an established framework for carrying out liquefaction mapping issued by MBIE, MfE[2] (2017), there is no consistent approach (nationally) to demonstrate compliance with the Building Code.

In addition, BCA's/TA's have indicated the need for substantial technical and regulatory guidance supported by information and education to support them map their region.

WHO IS AFFECTED?

These changes will affect several different stakeholders such as TA's, Building Consent Authorities (BCA's), designers, engineers and other end users. Feedback from the public consultation period[3] indicated the following:

- A small number of councils expressed concern in completing liquefaction mapping within the proposed transition time.
- Information (technical and regulatory) and further education was requested to target TA's, BCA's and other stakeholders to make the transition as smooth as possible.

BENEFITS FROM THIS CHANGE:

The change promotes safer and more resilient foundations for all of New Zealand and mapping liquefaction-prone throughout New Zealand, will help the industry avoid disruption and building owners incurring extra costs.

The cascading effect of natural hazards is difficult to measure/ quantify. However, dependant on the level of detail the completed mapping to in a particular region, understanding the risk and severity associated with the ground condition alongside a good foundation design could help reduce/ mitigate the impact of cascading hazards.

WHAT IS MBIE DOING?

MBIE understands that mapping for natural hazards (especially liquefaction) varies around the country with some TA's or regions well ahead of the requirement while other TA's require more support. So it is very important that MBIE takes a comprehensive approach to promoting compliance when the changes come live post 29 November 2021.

The framework for implementation of "Good Ground" definition change will involve a two phased approach:

Phase 1: Creating flowcharts and methodology statement for regional liquefaction assessments

Phase 2: National technical information that includes foundations and performance objectives

MBIE is currently finalising the publication of Phase 1 information. For the latest information, please visit building.govt.nz (https://www.building.govt.nz/building-code-compliance/geotechnical-education/ensuring-new-buildings-can-withstand-liquefaction-risks).

What to expect before the transition period finishes?

Other than the information expected later this year, It is expected the work associated with the transition would continue to ramp up from now until November 2021 with the intent to increase collaboration with the sector and relevant stakeholders. MBIE intends to carry out this in collaboration with other agencies (like EQC & MfE) and convey messages through Engineering New Zealand digital channels.

1 Regional and local councils have a requirement under the Resource Management Act (RMA) to manage hazards in their local jurisdiction and to ensure building work complies with the Building Code

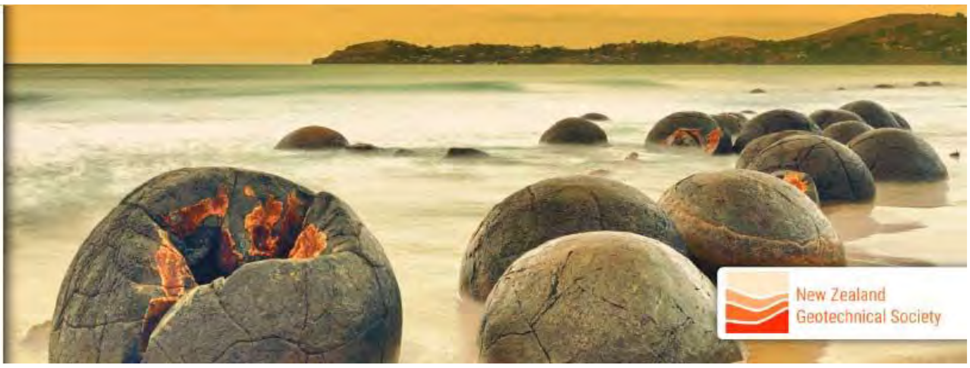
2 MBIE/MfE guidance 2017 - A risk-based process to address liquefaction in land use planning and development

3 Public consultation carried out in Aug – Sep 2019 as part of Building Code Update.

NZGS SYMPOSIUM 2021

Good grounds for the future

24-26 March 2021
Dunedin • New Zealand



Dear members

It is our pleasure to announce that planning for the NZGS 21st Symposium in Dunedin continues, now with a new date from the **24th to the 26th of March 2021**. We have been honoured and humbled by the ongoing support and commitment of our members and the industry during the year. We are looking forward to meeting you all in person for a rewarding event next year.

Our technical programme remains practically unaltered and includes:

- Online talks by international keynote speakers
- 65 talks and 25 posters
- A panel discussion on Soil / Structure Interaction & Collaboration between Geotechnical and Structural Engineers
- An update on the revision and finalisation of the MBIE/NZGS Geotechnical Modules
- A public talk themed around natural hazard, risk and opportunities to become more resilient as a community - delivered by Hugh Cowan with thanks to EQC
- Two parallel sessions devoted to the awarded NCTIR work

Don’t forget the pre-Symposium event!

23 March: Queenstown Workshop

"Engineering value from the monitoring of slopes – current practice & the future"

Join us for this afternoon workshop at Copthorne Hotel and Resort Queenstown. Hear from experts including Joseph Wartman, Michael Olsen, Mark Vessely, Peter Amos & Chris Massey.

24 March: Field Study through Cromwell Gorge

The field study will take guests by bus from Queenstown to Dunedin to arrive in time for the Symposium Welcome Reception. Your choice of visiting either Clyde Dam or Macraes Mine during the trip. Fly into Queenstown and depart from Dunedin.

NZGS SYMPOSIUM 2021

An overview of the technical program is uploaded on the website:

www.confer.nz/nzgs2021/

Thoughts for young Geotech engineers

Peter Millar

The editors of Geotechnics News have initiated a regular feature seeking career reflections from geotechnical practitioners that may assist undergraduate and graduates in managing their professional development and identify their long-term goals.

Tonkin + Taylor's Peter Millar is a geotechnical professional with over 40 years' experience. Peter's career includes periods of research, design and construction of land development, buildings and major infrastructure projects in New Zealand and SE Asia, governance boards of Alliances and senior management.

I ATTENDED A retirement function for a CEO recently where many of the senior management of constructors, consultants and major infrastructure clients were present. The collective experience in the room was massive, being responsible for much of the construction of recent buildings, transport systems and infrastructure projects in NZ. These engineers had steered the industry through some tough times and periods of extraordinary development and they were immensely enthusiastic about having had the opportunity to be part of NZ's development.

Why did they choose engineering and elect to spend a major part of their career in NZ? Why have majority of them worked well beyond the average retirement age and continue to give services to the engineering sector?

It brings to mind the old saying that "geologists never retire they just become old fossils".

Most of us would respond that their love for engineering is much more than a job - it is a journey, a service, a life choice and mostly a pleasure. We look forward to each day with a purpose, seeking to make a difference. This does not mean at the expense of all else, particularly family and recreation, but it is a strong driver. The engineering community in which we operate is also a major factor. The profession is mostly populated by like-minded people who seek to deliver good work, make a positive contribution to society, and provide opportunities for others.

Reflecting back, I would have to admit many of the defining moments in my career happened in spite of me, or through the actions of others. But I would like to think that I was selected on more than chance. There were occasions when I needed to make choices where I had to back myself to succeed and I would like to share a few.

Like many others at this time I joined the Ministry of Works and on completion of my masters' degree, I made the obligatory transfer to head office in Wellington. I was immersed, together with a large cohort of young engineers, in design roles for projects throughout NZ.

While initially located in the research and development team, I took opportunities to meet and get exposure to the work being undertaken by other divisions. It was mandatory to make at least two training transfers occurring every 18 months for the graduates, so it was useful to identify and promote yourself to seniors that could recommend your next shift. The transfer system ensured engineers got a range of experience to broaden their skills rather than being siloed early. They were then able to identify their career paths and could ultimately either take responsibility for management of diverse teams or dedicate themselves to a technical specialist role.

This 18-month transfer requirement was most useful in encouraging you to regularly review your progress, reflect on performances, and take personal responsibility for planning your own future.

So my early journey progressed from a Rock Mechanics postgraduate degree, into concrete technology and the early condition assessment of dams throughout NZ, investigations and design of the tunnels and caverns for the Rangipo Underground Power Station, followed by a transfer to Turangi to help build this amazing project.

This next formative lesson was having a role in building structures you have designed; my elegant thin arch lining and rock anchors system for supporting the powerhouse cavern had been skilfully designed using the state-of-the-art finite element software with stresses defined to the third decimal point. However, I had never heard of overbreak and the rock structure simply refused to comply with the 3D geological model we had carefully produced. It was in colour, so it should have been true.

Then there were the grizzly tunnellers who were experienced who could "listen" to the ground and observe the performance of the steel sets and lathing to gauge the adequacy of support. It was a big step for this young engineer fresh from the "bullshit" office (Wellington) to attempt to convince them to trust this new approach that challenged their conventional methods. I quickly had to broaden my vision, learn from their concerns, and progressively gain their confidence.

I also had to consider sensitivity to changes in ground conditions, appreciate the impacts of sequencing, constructability and safety in design. Working closely with the constructors ensured overall best results for project outcomes and built some great relationships. It certainly changed my appreciation of working in teams.

I had greatly benefited from the opportunity to be involved throughout the investigation, design and build stages of such a challenging project. It enabled me to understand the perspective of other disciplines, but it also allowed me to identify that my primary interest was probably in design.

I applied for the lead role for the Geotechnical Section at Central Laboratories and over the next eight years I was able to apply both my practical skills and design experience to identify opportunities to add value to projects. As well as technical expertise, I also needed to develop marketing and commercial skills to convince proposed budget holders to invest in the testing, instrumentation and design services we could provide. The economy was growing and the group expanded rapidly.

We worked throughout NZ with projects ranging from large scale rock shear testing for the foundations at Clyde Dam, trial load tests for long wall mining at Huntly, seismic refraction testing for oil exploration in Taranaki, development of downhole pressuremeter and geophysical testing equipment, dynamic compaction trials for Ohaaki power station, together with research on soft rocks, basecourse pavement materials and investigating vibration effects of construction on historic structures.

The critical differences between pure research and these operational research projects was that you generally only get one chance to get a result and it needs to be completed on time. It really focuses the approach.

Partial commercialisation of the Ministry of Works and Development encouraged me to consider a change into the private consulting industry and join the geotechnical team at Tonkin + Taylor. My timing may not have been optimal as it was just before 1987 financial crash, but I was fortunate as I had specialist skills that were needed for several projects and I was able to apply my marketing experience to help secure some good baseline projects.

This experience strengthened my resolve to always retain and continue development of the specialist skill that I could offer and I always encourage others to do the same. This has created many opportunities to differentiate ourselves including a number of major projects in Southeast Asia where we have successfully competed with international consultancies.

I recall one project where we invested in transporting and demonstrating our newly developed pavement testing technology to the client. I managed to sort out an electronics issue with minutes to spare, complete the trial and was greeted on return to our office with the signed contract. There have been many similar experiences.

I have since had positions of responsibility in the company including periods as Geotechnical Group Manager and Managing Director of the Tonkin + Taylor Group, but ensured I retained an element of technical projects and have subsequently returned to the operations team.

I focus on a mix of Governance roles on Alliance projects and provide technical direction for both land development and foundations of multistorey structures, while also assisting in business development and externally undertaking dispute resolution roles. A particular recent highlight has been multiple roles throughout the life of the Western Ring Route (Waterview tunnel) project where I initially managed the T+T team working on the specimen design, provided expert witness to the Board of Inquiry, then was geotechnical design lead for the ipAA phase before serving as a Board member for the project construction phase then transitioning to the Maintenance Alliance Board. Currently I am on the Boards of the CRL (Central Rail Loop), Wynyard Edge (America's Cup), Mt Messenger and Piritahi housing alliance projects. I feel privileged to be part of these enthusiastic teams who are keen to make a difference to our society, and where I can also provide advice and offer review.

The diverse range of projects I have worked on has required me to develop strong relationships with clients, developers, government agencies, architects, other design engineers and constructors. These relationships are dependent on trust, fairness and respect for what each can bring.

From my experience, the likelihood of successful outcomes is high when contracts are fair to all parties, do not seek to assign risk to parties that cannot reasonably manage them, and promote good relationships.

Geotechnical engineers are mostly involved during the early stages of projects. Our projects often present high risks, we have limited ability to test and have limited data. The materials which we operate with are variable and often perverse. Our clients are highly reliant us applying good judgement, supported by modelling and analysis. There will also be pressures to limit costs. We will not always get it right so when issues occur it is critically important that they are resolved quickly and risks to subsequent phases are also addressed. A first response to focus on blame and cost allocation is not constructive and inevitably leads to escalation, so wherever possible rapid response to mitigate and remediate is advised. This is where investment in good relations and trust pays dividends.

A FEW FINAL REFLECTIONS

Would I have chosen a different career No

Would I have been wealthier if I had selected a different career Probably

Would I have worked less hours Yes

Do I think I have made a positive difference to my community Yes

Do I believe you, the next generation of engineers will outperform us Of course

30 years ago some enlightened people took a deep breath and decided to give the current group of leaders a chance to demonstrate their potential; make sure you take your opportunities.

AGS ICSMGE 2021 Grant Application

The Australian Geomechanics Society (AGS) will offer a number of grants to assist individuals who might require additional resources to attend the International Conference on Soil Mechanics and Geotechnical Engineering in Sydney in 2021.

Application Requirements

- Applicants receiving a grant from the AGS Fund will not be eligible to receive a grant from the ISSMGE Foundation and vice-versa.
- Applications are due by **1 December 2020**.
- The Conference Local Organising Committee (LOC) and the AGS will determine the successful applicants by the end of February 2021.
- The LOC has determined that the AGS National Funding will be used in the following areas:
  - As a waiver or reduction of registration fees.
  - As a waiver or partial waiver of local accommodation booking (in a designated ICSMGE 2021 hotel determined by the LOC).
- All applicants must demonstrate a willingness to fund their own travel if not residing in Sydney.

Successful Candidates

Successful candidates must agree to provide a summary report of approximately 500 words including images (maximum two) for possible inclusion on the ICSMGE 2021 and AGS website and for publicity purposes. This report must be delivered within four weeks of attending the Conference.

Priority will be given to

- Applicants who have submitted an abstract (either a poster or oral presentation) for the Conference.
- Applicants who can demonstrate that their economic status makes self-funding challenging or impossible without such support.

To apply for the AGS ICSMGE 2021 Grant visit www.icsmge2021.org

Climate change, sustainable development and geotechnical engineering:
A New Zealand framework for improvement

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ABSTRACT
Climactic warming caused by the emissions of anthropogenic greenhouse gasses is occurring across the globe. These changes will increase the exposure of the built environment to hazards such as sea level rise and coastal inundation and exacerbate existing hazards such as expansive soils and landslides. In New Zealand, the built environment and its construction is responsible for about 20% of the emissions that are the primary cause of climate change. The way our built environment is designed must change to adapt to these future increases in hazard and must also mitigate emissions where possible to limit future increases in hazard to manageable levels. This paper describes climate change effects where they have overlaps with geotechnical design and hazard assessment (with particular reference to Auckland as an example), discusses the impact that these changes are expected to have on geotechnical practice in the coming years and decades, and presents a framework for managing these in the design processes.

Keywords: climate change, geotechnical engineering, sustainable development, adaptable design

1 INTRODUCTION

1.1 CLIMATE CHANGE

Since the last ice age, which ended approximately 12,000 years ago, Earth's climate has been relatively stable. This stability in temperature and sea level was crucial for the development of modern civilization. However, since about 1900 average temperatures around the globe have been increasing. This warming trend is expected to have a drastic impact on modern life which is tailored to the stable climate in which it developed.

Increasing temperatures have been strongly linked to emissions of anthropogenic greenhouse gasses (fig 1), such as CO2. Since the 1950s many of the observed changes are unprecedented. including environmental changes such as increased atmospheric and oceanic temperatures, decreased mass volume of snow and ice worldwide, and a significant rise in global mean sea level (IPCC 2014a).

The decisions we make today need to consider the

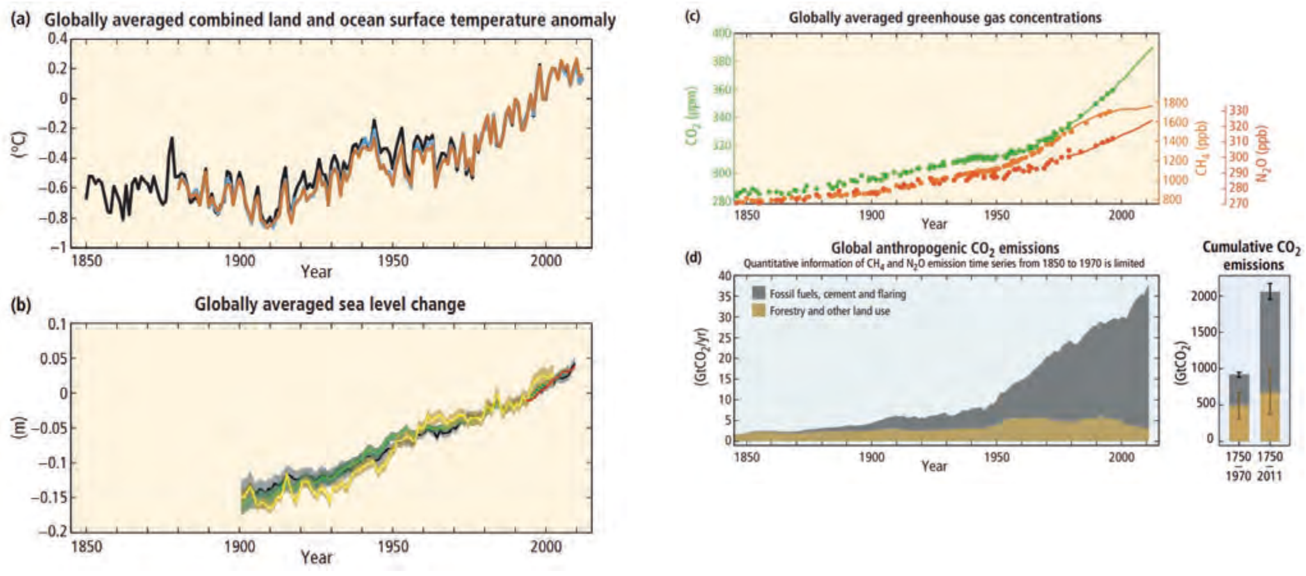

FIGURE 1. The relationship between observations (panels a, b and c with yellow background) and emissions (panel d, light blue background). After IPCC 2014a Figure SPM.1.

changing environment and the potential disruptions it will cause. A typical structure constructed today will still be in use in 2070 and beyond. However, the climate it encounters will be significantly different. Planning decisions and investment in major infrastructure made today will have consequences over even longer timeframes (for example, once a town is built its existence is near-permanent).

This paper discusses the impact that these climactic changes are expected to have on geotechnical practice in the coming years and decades and presents a framework for managing these in the design process.

1.2 SUSTAINABLE DEVELOPMENT

Sustainable development is development that meets the needs of the present without compromising the capacity of future generations to meet their own needs (Brutland Report, United Nations World Commission on Environment and Development, 1987). It is the broader framework under which the management of climate change's impact on the built environment falls.

Published in 1987, the Brundtland Report clearly highlighted the threat posed by climate change:

> *"Little time is available for corrective action. In some cases we may already be close to transgressing critical thresholds. While scientists continue to research and debate causes and effects, in many cases we already know enough to warrant action. This is true locally and regionally in the cases of such threats as desertification, deforestation, toxic wastes, and acidification; it is true globally for such threats as climate change, ozone depletion, and species loss. The risks increase faster than do our abilities to manage them."*

Seventeen UN Sustainability Goals were adopted by all UN Member States in 2015 as part of the 2030 Agenda for Sustainable Development, which set out a 15-year plan to achieve the goals (United Nations, n.d.). Goal 13 requires member states to "Take urgent action to combat climate change and its impacts", and includes the following targets:

(i) 13.1 Strengthen resilience and adaptive capacity to climate-related hazards and natural disasters in all countries.
(ii) 13.2 Integrate climate change measures into national policies, strategies and planning

Partly because of this, there is a significant effort in member countries to achieve these targets in the coming decade.

2 CLIMATE CHANGE IMPACTS

2.3 GENERAL CLIMATE CHANGE IMPACTS

Goal 15 of the UN Sustainability Goals focuses on land degradation, the loss of life-supporting land resource due to processes such as soil erosion, desertification, deforestation, and acidification.

Land degradation is exacerbated by climate change which can cause increases in:

- Rainfall intensity and changing rainfall patterns.
- Flooding frequency and severity.
- Drought frequency and severity.
- Heat stress.
- Wind strength.
- Coastal erosion and more frequent and extensive inundation caused by sea-level rise and wave action.
- Permafrost thaw.

Land degradation and increasing instability adds significant land use pressure in regions where land suitable for building was already scarce (IPCC, 2019).

2.4 GEOTECHNICAL IMPACTS

Land degradation and changes to land stability have a significant impact on the built environment. Climate change impacts of particular relevance to geotechnical engineering in New Zealand include drought and its impact on expansive and settlement-prone soils, coastal erosion and its impact on coastal cliffs, and rainfall intensity and its impact on land stability.

These issues create financial and social problems as well as engineering problems. Properties in hazard zones may struggle to get insurance, exposing the owners to significant loss. Even if the building platform can be protected, the loss of adjacent land can significantly devalue a building.

2.4.1 DROUGHT

Forecasts of future total precipitation rates vary geographically with some areas expecting an increase, and others a reduction. In Auckland, total rainfall is expected to decrease only slightly. However, changes in seasonal patterns may result in wetter autumns and drier springs (fig 2). An increase in the number and severity of droughts is expected to result in a greater range of groundwater levels, with the seasonal changes expected to result in lower groundwater levels in summer. This effect will be exacerbated by higher rates of evapotranspiration and in some areas because of increasing groundwater abstraction for human use.

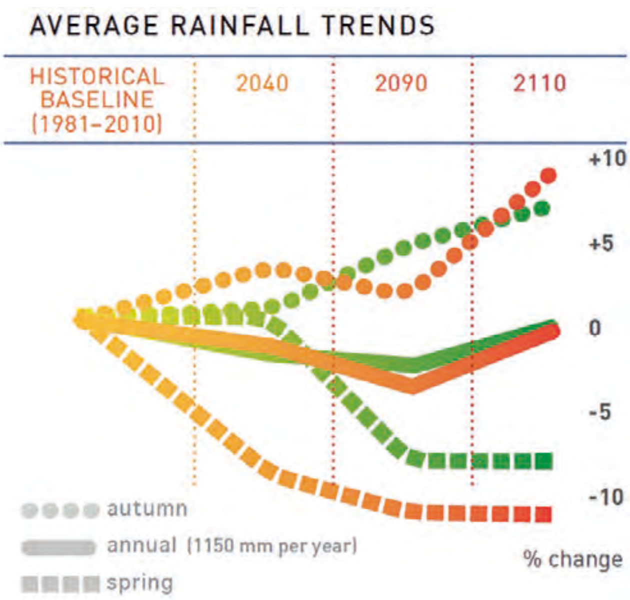

FIGURE 2. Average rainfall trends for Auckland to 2110 (Lorrey et al, 2018). RCP8.5 represents a scenario where emissions continue to rise at current rates, while RCP4.5 represents an emissions peak at 2040, followed by a slow decline likely to result in global temperatures from rising 2°C by 2100.

These more extreme summer lows are expected to increase shrinkage of expansive soils and increase (often irreversibly) settlement in normally consolidated clays and peat. This was highlighted by the Auckland's dry summer of 2020. Between 1 January and 21 May just 126 mm of rain was recorded at NIWA's Mangere weather station - an amount less than two-thirds of normal - making the drought one of the worst since the early 1900s, and resulting in a doubling of reports of building damage (pers. comm. Theo Hnat, Mainmark).

Slope stability is also likely to be affected by drought. Although (as noted below) drier conditions are often advantageous for slope stability, some drought effects can destabilise slopes, particularly:

- Loss of vegetation
- Desiccation cracking
- Reduction in soil suction

Extended droughts will kill some vegetation, particularly species that are adapted to a specific climatic zone. Over time, more drought-tolerant species will likely take over, but in the interim the loss of the stability and erosion control provided by root reinforcement and foliage cover may be significant.

Desiccation cracking due to soil drying is largely governed by the soil's plasticity, the temperature, and the number of volume change cycles experienced. These cracks form natural weaknesses and enhance vertical permeability allowing rainstorms to rapidly saturate specific horizons, particularly along existing weak planes that are common at weathering or lithological boundaries.

In partially saturated soils, reduction in soil suction causes a corresponding reduction in shear strength. This is largely governed by the pore size with fine grained soils retaining strength noticeably longer than soils with larger particles at similar saturation levels (Vahedifard et al, 2018). This loss of strength is challenging to predict and could result in unforeseen failures.

2.4.2 COASTAL EROSION AND SEA LEVEL RISE

Sea levels around Auckland are forecast to increase by between 0.4 m and 1.1 m by 2100 (fig 3).

An increase in coastal inundation and erosion rates is predicted for most areas. Cliff toe erosion will increase slope instability risks, and geotechnical structures will be more exposed to flooding and erosion.

As high-value land is eroded, demand for coastal protection measures is likely to increase. It is already common to install palisade walls around cliff-top properties to protect them against future erosion, and demand for sea walls is increasing.

2.4.3 RAINFALL INTENSITY AND STORMINESS

Across New Zealand, many areas are forecast to receive more intense rainfall, including Auckland (fig 4) where extreme rainfall intensity is forecast to increase by 15-30% by 2090 (Lorrey et al, 2018).

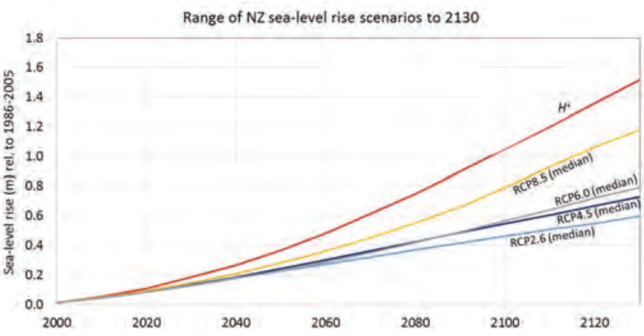

FIGURE 3. Range of sea level rise scenarios for NZ to 2130 (Stephens, 2017)

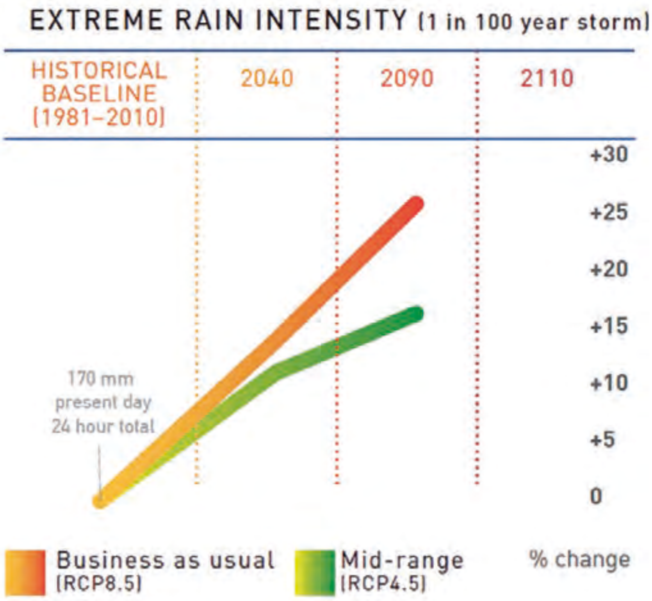

FIGURE 4. Extreme rain intensity scenarios for Auckland to 2090 (Lorrey et al, 2018).

In areas already forecast to receive an increase in total precipitation, higher groundwater levels and pore water pressures are expected and will contribute to a reduction of shear strength, a reduction in soil suction and cohesion, and an increase in the weight (wet density) of the slope materials, all decreasing slope stability. Therefore, less rain will be required to fall in each event to reach a critical level that can cause a slope to fail. With an increase in the rainfall intensity this is likely to cause a significant increase in landslide susceptibility.

Conversely, a reduction in total precipitation would normally result in more stable conditions. However, the forecast increase in rainfall intensity may cause higher infiltration into dry, cracked ground during storm events (where the soil and bedrock allow it) and an increase in subsurface drainage and throughflow. Together these contribute to the build-up of perched water tables, the reduction of effective normal stresses and the shear strength, again contributing to slope instability (Gariano and Guzzetti, 2016).

Increased rainfall intensity will also increase surface runoff (overland flow) and the related surface erosion processes. These may in turn facilitate debris flow initiation and enlargement, as well as contributing to increased erosion of the river banks, increasing bank instability. The instability of the river banks may propagate upslope or laterally, initiating new landslides or reactivating old, dormant ones.

Increases in the number and severity of storms is also forecast, which has implications for wind and wave loadings on structures, as well as increasing rates of coastal erosion. Stronger winds are likely to result in further destabilisation of slopes as instability can be triggered by trees blown over in storms.

2.4.4 COASTAL GROUNDWATER LEVEL RISE

As sea levels rise, groundwater in the vicinity of the coast is also likely to rise. In areas where the groundwater is already shallow, such as low-lying alluvial plains, flooding will likely be exacerbated by reduced stormwater infiltration. Shallower water will affect the strength of soils beneath foundations, potentially causing widespread structural damage, and more land will become susceptible to liquefaction during seismic shaking (Quilter et al, 2015).

2.4.5 MULTI-HAZARDS

Climate change is predicted to increase the risk posed by individual extreme events. However, multi-hazard scenarios where the occurrence of simultaneous or sequential events is considered to result in a much larger total risk.

For example, a period of intense drought (causing desiccation cracking and vegetation die-back) followed by an intense rainfall event (causing erosion of now unprotected surfaces, and rapid saturation of deep soils through desiccation cracks) would likely lead to much higher risk from landslides than either of these two events alone.

3 CLIMATE CHANGE STRATEGIES

In general responses to climate change responses are split into two categories: mitigation and adaptation.

Mitigation focuses on our ability to control emissions and is a human intervention to reduce the sources or enhance the sinks of greenhouse gases (IPCC, 2014b). The goal of mitigation is to stabilise greenhouse gas levels in a timeframe sufficient to:

- Allow ecosystems to adapt naturally to climate change.
- Ensure that food production is not threatened.
- Enable economic development to proceed in a sustainable manner.

Adaptation is the process of adjustment to actual or expected climate changes and their impacts. In human systems, adaptation seeks to moderate or avoid harm or exploit beneficial opportunities (IPCC 2014c).

Because of the quantity of greenhouse gasses added to the atmosphere in the past few centuries, we are already committed to some level of climate change and therefore to some level of adaptation. Without reducing our current rate of emissions, these changes will become much more severe. Therefore, it is generally accepted that both approaches are required in parallel. Adaptation is essential to respond to the changes that will happen, and mitigation is essential to limit these changes to manageable levels.

Adaptation and mitigation each have the potential to contribute to, or to impede, sustainable development. The best outcomes will occur when adaptation and mitigation work together to reduce risks of disruptions from climate change. Trade-offs between adaptation and mitigation and between economic goals and environmental goals may be required. In some cases adaptation may increase greenhouse gas emissions (e.g. increased air conditioning in response to higher temperatures). In other cases mitigation may impede adaptation (e.g. emissions trading may increase the cost of concrete, making it unaffordable for some countries to invest in relocating critical infrastructure away from hazard zones).

Climate change requires new approaches to sustainable development that consider the complex interactions between climatic, social and ecological systems. Climate-resilient pathways need to be defined that show how development can take place over time in a way that combines adaptation and mitigation to realize the goal of sustainable development. These pathways are iterative, continually evolving processes for managing change within complex systems (Denton et. al., 2014).

These approaches have secondary benefits. Strategies for climate change responses and strategies for sustainable development are often highly interactive, and in some cases, reducing the risk related to climate change enhances the capacity to manage other risks (Denton et. al., 2014).

4 ADAPTATION

In the context of geotechnical engineering, adaptation strategies for structures or other assets can be divided into three categories:

(i) Reducing vulnerability. For example, designing to a higher standard helps structures withstand changes in future loading.
(ii) Avoiding exposure. For example, this may be achieved by building in a less risky location.
(iii) Increasing adaptability. By accommodating future change in the design this approach helps avoid locking in investments that could make future adjustments difficult and costly.

4.1 REDUCING VULNERABILITY

When assessing the cost/benefit ratio of projects in the built environment, it is normal practice to project the investment life of the project for 30 to 100 years. It is usually assumed (often implicitly) that the distribution of past variations in the environment is a reasonably accurate guide to the range of likely future variations: e.g., historical records of the variations in future river flows, coastal tidal and storm event patterns (Mazmanian et al 2013).

However, future environmental conditions will be dramatically different because of the accelerating rate of climate change and so relying on the past patterns when evaluating development proposals is hazardous.

Loadings and other design criteria specified in national legislation and standards are commonly applied when designing structures. However, these criteria often lag well behind current scientific knowledge and professional best practice as they are usually based on historical conditions and the processes to change them can be quite lengthy. These standards should be seen as setting a minimum standard and should be critically assessed by designers and clients. Clients have a vested interest in ensuring the longevity of their assets and should not assume that compliance with local or national legislation will be enough to meet their long-term requirements in a changing environment.

4.2 REDUCING EXPOSURE

Reducing the exposure to future climate change risk requires an understanding of how the hazards described in Section 2.2 will affect specific sites proposed for development. This is more challenging for some hazards than others and can be more challenging on a site-specific basis than regionally.

Coastal inundation is complex, but less so than some other hazards. Forecasts of sea level rise are available, and models have been produced for many regions in the world indicating areas that are likely to be inundated under a range of climate change scenarios. These can then be considered in land-use planning and site selection. The models can be used directly in design (e.g. establishing appropriate setbacks from the inundation extent or minimum floor levels above flood levels) with an appropriate scenario selected to match local regulations and the client's risk appetite.

Other hazards are more challenging to model. For example, changes in slope stability are likely to be controlled by changes in total rainfall, changes in rainfall intensity, changes in vegetation and changes in temperature. While models are available for each of these input parameters, there are no readily available tools to assess what impact they will have on a particular slope. Collection of quality open geotechnical datasets will become increasingly important to support modelling of future impacts.

Hazards that are relatively consistent across large areas, such as settlement of expansive soils, can be readily addressed on a regional basis by identifying at risk areas and adapting building codes to be more resilient. However, codification can create unintended negative consequences such as increased construction cost and emissions.

4.3 INCREASING ADAPTABILITY AND DEALING WITH UNCERTAINTY IN DECISION MAKING

A series of decisions are made when zoning land use, developing land, making infrastructure investment, and designing. These decisions occur at many stages in the process starting with identifying the need for change through business case, concept design, site selection, detailed design, construction, operation, and decommissioning.

Models based on past patterns and events are often inadequate for characterising the effects of global warming and the complex dynamics that will result (Lempert et al, 2000). This places us in an analytical environment of deep uncertainty, which Lempert defines as "a condition where the parties to a decision do not know or do not agree upon the system model relating potential actions to outcomes, the prior probabilities for the value of key uncertain input parameters to the system model(s), and/or the value function that should be used to rank alternative outcomes". A key challenge when dealing with climate change is the lack of certainty about the future timing and rate of change – we cannot easily predict the conditions that our projects will encounter during their design life.

To cope with this deep uncertainty, a host of methods are being developed. These range from simulations to narratives and Delphi and Foresight exercises (in which individuals and groups participate in imagining plausible scenarios about future states, based on alternative projections of climate change), to "no-regrets" strategies and the dynamic adaptive pathways planning approach. All these approaches are intended to help policy makers better gauge what courses of action are preferable, in the light of the deep uncertainties they face.

Dynamic Adaptive Pathway Planning is the preferred approach in New Zealand, and is described further in MfE 2017. It is a decision-making process where multiple possible outcomes are prepared for. Trigger points are defined within the design process that will, at some point in the future, determine which outcome will be followed, or at what time the next stage will take place. In geotechnical terms, this is somewhat analogous to the observational method in geotechnical engineering described by Peck (1969), but with the addition of more formally defined trigger points and subsequent actions.

For example, a building may be designed on land that is currently stable, but which may have its stability reduced below acceptable criteria under certain climate change scenarios. The designer could consider options to create more resilient foundations from the start, or instead could design a dynamic approach where they set aside a suitable area of land for a future retaining wall to be built when groundwater hits a particular trigger level.

Adaptive management approaches, which enable actions or policies to proceed in the light of uncertainties, are not new. They have been used for resource management decision-making (e.g., water quality) and policy development both internationally and in New Zealand over the last few decades (Lawrence et al, 2020).

In response to rising sea levels around our shores, the New Zealand Coastal Policy Statement and supporting guidance (MfE, 2017) advocate the use of an adaptive planning approach to deal with the uncertainty and change around associated risks in the future. Policy 27 of the NZCPS outlines a strategy for managing the rising risk to existing coastal developments from climate-change effects, where a range of options for reducing coastal hazard risk should be assessed over "at least 100 years" and include "identifying and planning for transition mechanisms and time frames for moving to more sustainable approaches".

5 MITIGATION

5.1 REDUCTION TARGETS

Multiple lines of evidence indicate a strong, consistent, almost linear relationship between cumulative CO_2 emissions and projected global temperature change (IPCC 2014). To encourage mitigation New Zealand has several greenhouse gas emissions reductions targets. Our international targets are:

- Five per cent reduction below 1990 gross emissions for the period 2013-2020
- 30 per cent reduction below 2005 (or 11 per cent below 1990) gross emissions for the period 2021-2030 (MfE, n.d.).

In addition to these international targets, in 2019 the New Zealand Climate Change Response (Zero Carbon) Amendment Act set into law a new domestic target of net zero emissions of all greenhouse gases other than biogenic methane by 2050. This new target brings New Zealand in line with the global ambition set under the Paris Agreement.

5.2 CONSTRUCTION SECTOR IMPACT

The building and construction sector is a large contributor to greenhouse gas emissions from producing materials, constructing buildings and infrastructure, and the energy used in buildings. Globally, energy use in buildings contributed 19% of society's global carbon footprint in 2010, according to the Intergovernmental Panel on Climate Change.

Vickers et al. 2018 identified that New Zealand follows similar trends to international figures (fig 5). Considering the full life cycle (construction, use and end-of-life), the contribution of the built environment (i.e. buildings and infrastructure) is approximately 13% of New Zealand's gross carbon footprint. However, this value ignores emissions generated overseas during manufacture of construction materials, many of which are imported. Adjusting for the carbon footprint embodied in New Zealand exports and imports, the contribution to CO_2 equivalent from the built environment climbs to 20%.

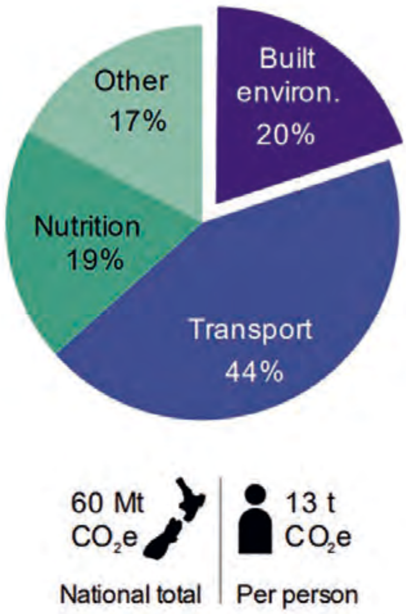

FIGURE 5. A breakdown of New Zealand's carbon footprint in 2015 from a life cycle consumption perspective including international trade (Vickers et al. 2018). CO2e is CO2 equivalent, a measure of greenhouse gas emissions.

Emissions from buildings and infrastructure are commonly divided into embodied emissions, operational emissions, and end-of-life emissions.

Embodied emissions are the emissions generated during the manufacture of the building products and materials used in construction, maintenance and renovation. They occur upstream of the building itself, are one-off or irregular, are largely invisible to the architect or builder, and are often locked in before the first occupier even steps into the building for the first time. Given that these emissions cannot be changed later, they gain in importance over time as the energy mix used to operate the building decarbonises (reducing the relevance of the operational phase).

In New Zealand, embodied carbon for residential buildings is dominated by emissions from the production of steel and concrete (Figure 6).

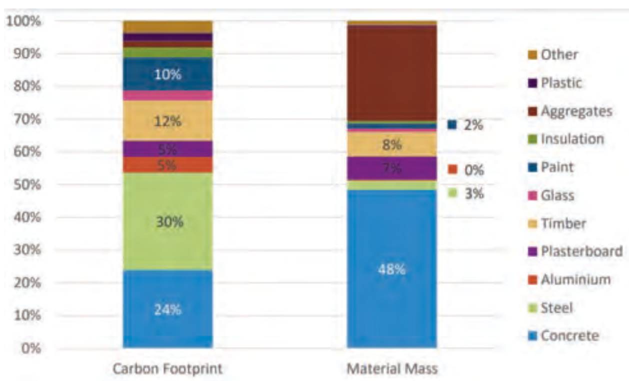

FIGURE 6. Carbon footprint and material mass breakdown for residential buildings in NZ over their full life (ThinkStep, 2019).

Operational emissions are the emissions produced by running a building (e.g. through heating and cooling). They are very visible as there is an ongoing cost associated with them (utility bills, maintenance bills, etc.), which creates a financial incentive for reduction. They can be improved through retrofits (e.g. replacing electric radiators with high-efficiency heat pumps) and higher-specification newbuilds (e.g. better insulation and air-tightness); however, there are cases where 'lock-in' occurs (e.g. under-slab insulation and building orientation).

End-of-life emissions are those generated in the demolition of a structure and the disposal of the resulting material.

Vickers divided the emissions into these groups and identified that approximately half of all emissions were embodied in building materials (used for both buildings and infrastructure), half were from operating our building stock (i.e., buildings only). Only a small proportion were from end-of-life. This was also supported by MBIE 2020b (fig 7).

However, for the unoccupied built environment (e.g. roads and other infrastructure), embodied emissions account for over 90% of the life cycle emissions (Huang et al, 2018), and therefore management of embodied emissions in infrastructure is likely to become a sigificant focus.

Chandrakumar et al (2019) attempted to calculate the acceptable whole-of-life carbon emissions for a residential building in New Zealand to be compatible with a global warming limit of 2°C. They found that the climate target of a detached New Zealand house over a 90-year lifetime is 71 tonnes CO2 equivalent, and reported that this would be equivalent to a reduction of 80% relative to current practice. Even in the best-case scenario where operational emissions were eliminated entirely, construction would still need to eliminate 60% of embodied carbon to meet national objectives. This is a very significant challenge, and one we have no current clear pathway to achieve.

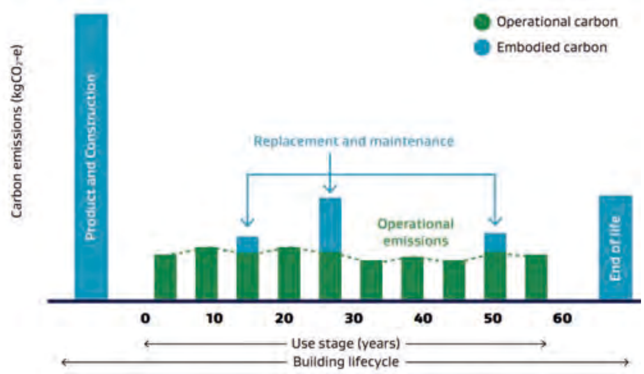

FIGURE 7. Operational and embodied carbon emissions over the life cycle of a building (MBIE 2020b).

5.3 MITIGATION IN GEOTECHNICAL ENGINEERING

Significant carbon equivalent reductions are required from our construction practices in order to meet our climate change goals. However, advice on achieving these in the geotechnical realm is rare, and it has been reported that emissions from the construction industry in New Zealand have increased by 66 percent in the decade from 2007 to 2017 (NZGBC, n.d.).

Concrete and steel are among the most widely used resources in geotechnical engineering, but efficient use is not always a significant consideration in design. The global production of cement has grown very rapidly in recent years, and after fossil fuels and land-use change, it is the third-largest source of anthropogenic emissions of carbon dioxide (Andrew, 2018).

Despite this, because of the longevity of these materials the whole-of-life carbon cost can be lower than other, initially less carbon emitting alternatives.

There are few tools to allow a robust comparison between different design alternatives in the geotechnical sphere. The BRANZ "Whole-building whole-of-life framework" provides tools, data and information to support decision making for sustainable building design. It assists calculation of the climate change impacts and other environmental impacts of our buildings (BRANZ, u.d.). This tool does allow a comparison between concrete and other building materials using data valid for New Zealand, but it focusses on structural elements, does not directly consider foundations and has no data for subgrade improvement, earthworks or other geotechnical structures.

6 NZ GOVERNMENT RESPONSE

Partly in response to our governments' commitment to UN Sustainability Goal 13, the New Zealand Ministry of Business, Innovation & Employment is developing a programme of work ("Building for Climate Change") designed to reduce emissions from buildings during their construction and operation, while also preparing buildings to withstand changes in the climate (MBIE, n.d.). Their stated goals for 2050 are:

- New Zealand's buildings are using as little energy and water as possible. They are warmer, drier and better ventilated, and provide a healthier place for us all to work and live.
- The wellbeing of New Zealanders has improved, they're leading healthier lives, and respiratory illnesses from cold and damp houses is uncommon. People also have more money in their pockets due to lower energy bills.
- Our infrastructure finds it easier to respond to demand for water, due to our lower use. This means we cope better with water shortages than we ever have before.
- The efficiencies from the Sector have made it easier for the grid to become more renewable meaning less emissions for the energy we do use.
- Energy Efficiency and carbon cost are core considerations for the Sector and designs now meet an emissions budget as well as other regulatory requirements.
- Reusing buildings and recycling materials is an established part of a Sector that is well on the way to having a fully-fledged circular economy well supported by local supply chains.

In July 2020 MBIE released a consultation document describing a proposed framework for change within the building and construction sector (MBIE, 2020a). These changes, currently focused on new buildings, involve defining operational efficiency levels for buildings to be able to get building consent. Of particular relevance to geotechnical practice, they also include plans to reduce whole of life embodied carbon. The framework will set mandatory reporting requirements, and embodied carbon caps that will need to be achieved to get consent.

7 A FRAMEWORK FOR GEOTECHNICAL PRACTITIONERS

Decisions made by geotechnical practitioners, normally in consultation with their clients, colleagues, and supply chain, can have a significant impact on the ability to integrate mitigation and adaptation into a design.

Geotechnical practitioners are likely to have a requirement to change how we decide on sites for development and on design solutions in response to government policy (e.g. the MBIE Building for Climate Change programme).

The following proposals are made to assist with this decision-making process.

7.1 ASSESS FUTURE CONDITIONS

Without understanding how conditions affecting our project sites may change in the future, we cannot take them into account in the design. Geotechnical practitioners should take a lead in considering how climate change could affect the ground, and what this means for design and site suitability.

Consider current conditions and a range of plausible future environmental conditions that may exist during the functional life of the structure. Identify how each of these may change:

- The geological model
- Geotechnical parameters
- Vegetation cover, type and density

7.2 QUESTION THE SCOPE

Once geotechnical professionals get involved in a project it will often have been scoped, at least at a concept phase, and the geotechnical role is limited to delivering on that scope. However, geotechnical professionals have an understanding of hazards and climate change that might not have been considered in the decisions made.

It is imperative that an understanding of the client's drivers is shared by the whole project team, and that the project scope is regularly re-visited to re-confirm that it is still the best way to solve the client's problem.

7.3 PLAN FOR CHANGES IN CLIMATE AND SOCIETY

Consideration of how the long-term needs and requirements of clients will change due to the impacts of climate change is rapidly becoming a necessity. If a societal or climate change means that the functionality is partly lost, is the design flexible enough to allow easy adaptation for other likely scenarios?

Bringing climate knowledge into early planning discussions with clients is essential to identify if a more flexible approach may be advantageous for them.

7.4 CONSIDER FUNCTIONAL LIFE

Traditional design focuses on providing assurance that the structure will be safe over a specified design life, with little consideration given to what happens after that point. Most structures significantly outlast their design life and so will be subject to changes that did not occur within the design life of the structure. In addition to designing for the specified design life, geotechnical engineers should consider the full functional life of a structure. If a fully functional structure has to be demolished after its design life because the designer did not consider long-term effects, this is a significant waste.

Where possible, we will need to provide better techniques to preserve existing building stock as so as to avoid the negative carbon footprint of building new buildings and demolishing the old.

7.5 USE SCENARIO-BASED DESIGN

Dynamic Adaptive Pathway Planning can be used to give an insight into how future changes may affect a structure. A similar approach may be used to identify the range of plausible future scenarios (using the assessment of future conditions recommended in Section 7.1).

In the case of slope stability assessment, it is already normal practice to consider a range of load cases and model each independently, often with differing requirements for a factor of safety depending on the likelihood of the load case occurring during the design life or the uncertainty of the assumptions made.

This proposal takes the same approach and extends it to considering future climate scenarios which may alter soil properties, groundwater levels, vegetation reinforcement etc.

Scenarios that are only likely after the design life of the building could, with agreement of the client and regulator, accept a lower factor of safety than scenarios likely to occur within the design life.

7.6 BALANCE LONGEVITY AND ADAPTABILITY

We must consider both mitigation and adaptation in our designs. Often these requirements will be in conflict. For example, a very resilient foundation design may require more concrete and therefore have higher embodied emissions.

As geotechnical practitioners, we have the knowledge to offer designs that move beyond the basic ‘single design’ approach to a more adaptive approach that considers the longer term with a series of staged dynamic changes to ensure that the structure remains useful, resilient, and responsive to its environment.

It is recommended that a suite of options is developed, supported by clear advice to your client about the relative costs and benefits of using more robust (and potentially more carbon intensive and initially expensive) design relative to more adaptable designs which may have lower up-front costs but higher costs over their lifecycle. Consideration must be given to how to avoid the moral hazard of selecting the lower-cost option up front and leaving higher mitigation costs to future owners, particularly where the client is not likely to be a long-term owner of the building.

7.7 OPTIMISE MATERIAL USE WITH CARBON ACCOUNTING TOOLS

Although development of widely available tools for carbon accounting is still in the relatively early phases, and those that do exist are generally quite limited in the geotechnical sphere, early adoption of these techniques will drive further progress and give a more robust basis for decision making on projects. The whole-building whole-of-life framework may provide a useful starting point (BRANZ u.d.).

There are two primary options to reduce embodied carbon; increase building material efficiency, and reduce carbon intensity.

Increasing building material efficiency means using less material in new buildings, including reducing waste and minimising replacement over the building’s life cycle.

Reducing the carbon intensity of the materials used in new buildings is achieved by either by making design choices to use low-carbon materials over high-carbon alternatives, and/or reducing the embodied carbon of the construction materials (MBIE 2020b).

Both should be considered as part of the design process.

7.8 ASSESS SUSTAINABILITY THROUGHOUT DESIGN

Safety in design is now becoming common practice. By considering safety from project conception through to decommissioning, safety is being much more deeply considered in design.

The same approach should be standard practice for sustainable development and climate change response. Although geotechnical practitioners are often only involved in a design process after some key decisions have been made, their role in site selection and hazard assessment gives them a rare opportunity to influence the whole design process.

Consider how the materials used in the structure could be re-used or re-purposed at the end of the structures life, and where appropriate adjust the design to make this process easier. For example, consider how piles might be reused by future buildings, or how a slab footing could be crushed and what effect the reinforcement may have on this. Document these assumptions and make them available in the design documentation so that they can be used at the point of demolition.

8 ROLES AND RESPONSIBILITIES

8.1 ROLE OF CENTRAL GOVERNMENT

In New Zealand the government has a significant unbudgeted exposure to liability for natural hazard impacts, which will increase over time with growth in the values at stake and the anticipated weather effects of climate warming and sea level rise. The mean projections suggest that the Crown's annual contingent liability for natural hazards would grow from $0.7 billion in 2020 to $3.3 billion in 2050. That liability could be effectively reduced by investing in natural hazard risk mitigation that reduces risks and hence liability (NZIER, 2020). While insurance and reinsurance can only cover some of the risks of losses caused by hazards, in other cases, government faces an undefined liability for reinstating infrastructure damaged by natural disaster and for providing disaster relief. For the largest events, the impacts are potentially destabilising for government finances. Governments therefore have a huge incentive, and a huge responsibility, to manage these risks.

Governments can influence the energy optimization of private buildings, transportation and land use through their legal decisions and regulatory instruments. Central government bodies generally set minimum building standards. In New Zealand the central government role is particularly important in driving reductions in embodied carbon.

Reducing operational emissions is of direct benefit to the owner as it can result in reduced whole-of-life costs and is therefore relatively easy to incentivise. However, reducing embodied carbon in structures rarely results in lower whole-of-life costs, and is more likely to require government regulation to drive change.

8.2 ROLE OF LOCAL GOVERNMENT

Because local government entities are closer to their citizens than central governments, they can play a significant role in launching initiatives and bringing policies into action. However, local governments are limited by different levels of influence and control when they try to tackle global warming issues.

In New Zealand, local governments primary tool is in land-use planning to encourage adaptation measures.

Local governments can also motivate and educate their communities and stakeholders to take action to reduce carbon emissions. In comparison to national governments, local governments leverage proximity to their citizens to maximize motivation and education in this area.

Local governments have a lot of authority to reduce municipal carbon emissions in their own facilities. Although municipal buildings contribute little to overall urban emissions, the public building sector is one of the highest emissions sectors under direct municipal control. Local governments can act as role models for the private and commercial building sector by attempting to meet global climate targets.

8.3 ROLE OF CLIENTS

Clients must start to demand buildings which are fit for the coming century and beyond. Requiring sustainability and climate change to be considered throughout the design process by consultants and contractors (in the same manner as safety in design) would make a significant difference. When assessing tenders, consider what suppliers are doing to ensure the structure will not be exposed to future hazards, and will continue to perform as intended throughout its life. Consider specifying design for an agreed emissions scenario (for example, a 2°C global temperature rise) as a minimum with an assessment required to consider the implications of a higher (e.g. 4°C) rise, and require suppliers to describe how this will change the design parameters and decisions.

8.4 ROLE OF CONSULTANTS AND DESIGNERS

Designers and consultants need to move beyond national standards and guidelines and start considering a range of possible climates that may exist over the real life of a structure rather than only over its design life.

Designers should discuss with their clients the risks and encourage them to embed climate change mitigation and adaptation in their brief.

9 CONCLUSIONS

Our climate is changing, and this change poses many challenges for owners and designers of structures that are usually expected to last for many decades or even centuries.

Geotechnical professionals have an understanding of how climate changes the ground on which these structures stand and can provide insight into how changes in climate may change the natural hazards to which these structures will be exposed.

Geotechnical professionals have the skills to help their clients adapt to, and to some extent to mitigate, the effects of climate change.

A framework is presented that proposes a simple approach to integrate the appropriate thinking process into design.

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REFERENCES

Andrew, R. M., (2018). "Global CO2 emissions from cement production". Earth Syst. Sci. Data, 10, 195–217, 2018. https://doi.org/10.5194/essd-10-195-2018

BRANZ (u.d.). "Whole-building whole-of-life framework". Recovered on 30 August 2020 from https://www.branz.co.nz/environment-zero-carbon-research/framework/

Chandrakumar, C., McLaren, S. J., Dowdell, D., and Jaques, R. (2019). "A top-down approach for setting climate targets for buildings: the case of a New Zealand detached house". In: Proceedings of the Sustainable Built Environment Conference 2019. IOP Conf. Series: Earth and Environmental Science 323 (2019) 012183. doi:10.1088/1755-1315/323/1/012183.

Denton, F., T.J.Wilbanks, A.C. Abeysinghe, I. Burton, Q. Gao, M.C. Lemos, T. Masui, K.L. O'Brien, and K.Warner, 2014: Climate-resilient pathways: adaptation, mitigation, and sustainable development. In: Climate Change 2014: Impacts, Adaptation, and Vulnerability. Part A: Global and Sectoral Aspects. Contribution of Working Group II to the Fifth Assessment Report of the Intergovernmental Panel on Climate Change [Field, C.B., V.R. Barros, D.J. Dokken, K.J. Mach, M.D. Mastrandrea, T.E. Bilir, M. Chatterjee, K.L. Ebi, Y.O. Estrada, R.C. Genova, B. Girma, E.S. Kissel, A.N. Levy, S. MacCracken, P.R. Mastrandrea, and L.L.White (eds.)]. Cambridge University Press, Cambridge, United Kingdom and New York, NY, USA, pp. 1101-1131.

Gariano, S. L., and Guzzetti, F. (2016). "Landslides in a changing climate". Earth-science reviews 162 (2016) 227-252.

Huang, L., Krigsvoll, G., Johansen, F., Liu, Y, Zhang, X., 2018. "Carbon emission of global construction sector". Renewable and Sustainable Energy Reviews 81 (2018) 1906-1916.

IPPC (2014a). "Climate Change 2014: Synthesis Report. Contribution of Working Groups I, II and III to the Fifth Assessment Report of the Intergovernmental Panel on Climate Change" [Core Writing Team, R.K. Pachauri and L.A. Meyer (eds.)]. IPCC, Geneva, Switzerland, 151 pp.

IPCC (2014b). "Summary for Policymakers". In: Climate Change 2014: Mitigation of Climate Change. Contribution of Working Group III to the Fifth Assessment Report of the Intergovernmental Panel on Climate Change [Edenhofer, O., R. Pichs-Madruga, Y. Sokona, E. Farahani, S. Kadner, K. Seyboth, A. Adler, I. Baum, S. Brunner, P. Eickemeier, B. Kriemann, J. Savolainen, S. Schlömer, C. von Stechow, T. Zwickel and J.C. Minx (eds.)]. Cambridge University Press, Cambridge, United Kingdom and New York, NY, USA.

IPCC (2014c). "Summary for policymakers". In: Climate Change 2014: Impacts, Adaptation, and Vulnerability. Part A: Global and Sectoral Aspects. Contribution of Working Group II to the Fifth Assessment Report of the Intergovernmental Panel on Climate Change [Field, C.B., V.R. Barros, D.J. Dokken, K.J. Mach, M.D. Mastrandrea, T.E. Bilir, M. Chatterjee, K.L. Ebi, Y.O. Estrada, R.C. Genova, B. Girma, E.S. Kissel, A.N. Levy, S. MacCracken, P.R. Mastrandrea, and L.L.White (eds.)]. Cambridge University Press, Cambridge, United Kingdom and New York, NY, USA, pp. 1-32.

IPCC (2019). "Summary for Policymakers". In: Climate Change and Land: an IPCC special report on climate change, desertification, land degradation, sustainable land management, food security, and greenhouse gas fluxes in terrestrial ecosystems [P.R. Shukla, J. Skea, E. Calvo Buendia, V. Masson-Delmotte, H.- O. Pörtner, D. C. Roberts, P. Zhai, R. Slade, S. Connors, R. van Diemen, M. Ferrat, E. Haughey, S. Luz, S. Neogi, M. Pathak, J. Petzold, J. Portugal Pereira, P. Vyas, E. Huntley, K. Kissick, M. Belkacemi, J. Malley, (eds.)]. In press.

Lawrence, J., Bell, R., Blackett, P., Stephens, S., Collins, D., Cradock-Henry, N., and Hardcastle, M., (2020). "Supporting decision making through adaptive tools in a changing climate Practice guidance on signals and triggers". Wellington: Deep South Challenge.

Lempert, R., Scheffran, J. and Sprinz. D. F. (2009). "Methods for long-term environmental policy challenges". Global Environmental Politics 9:106-133. http://dx.doi.org/10.1162/glep.2009.9.3.106

Lorrey, A.M., Pearce, P.R., Barkus, C., Anderson, S.J., and Clement-Jones, A. (2018). "Auckland Region climate change projections and impacts: Summary report. Revised January 2018". Prepared by the National Institute of Water and Atmospheric Research (NIWA) for Auckland Council. Auckland Council Technical Report, TR2017/031-2.

Mazmanian, D. A., Jurewitz, J. and Nelson, H. T. (2013). "A governing framework for climate change adaptation in the built environment". Ecology and Society 18(4): 56. http://dx.doi.org/10.5751/ES-05976-180456

Ministry for the Environment (2017). "Preparing for coastal change. A summary of coastal hazards and climate change guidance for local government". PO Box 10-362, Wellington, New Zealand. ISBN: 978-1-98-852526-6. Publication number: ME 1335.

Ministry for the Environment (2020). "New Zealand's Greenhouse Gas Inventory, 1990-2018". Ministry for the Environment Manatū Mō Te Taiao, PO Box 10-362, Wellington, New Zealand. ISSN: 1179-223X (electronic). Volume 1, chapters 1–15. Publication number: ME 1496.

Ministry for the Environment (n.d.). "About New Zealand's emissions reduction targets. Retrieved 12 August 2020 from https://www.mfe.govt.nz/climate-change/climate-change-and-government/emissions-reduction-targets/about-our-emissions

Ministry of Business, Innovation & Employment, 2020a. "Building for Climate Change: Transforming the building and construction sector to reduce emissions and improve climate resilience". July 2020. ISBN: (online) 978-1-99-001934-0

Ministry of Business, Innovation & Employment, 2020b. "Whole-of-life embodied carbon emissions reduction framework (Building for climate change programme)". August 2020. ISBN: (online) 978-1-99-001947-0

Ministry of Business, Innovation & Employment (n.d.). "Building for climate change". Retrieved 19 August 2020 from https://www.mbie.govt.nz/building-and-energy/building/building-for-climate-change/

New Zealand Green Building Council, n.d. "Climate change and building pollution". Recovered on 30 August 2020 from https://www.nzgbc.org.nz/Category?Action=View&Category_id=389

New Zealand Institute of Economic Research, 2020. "Investment in natural hazards mitigation – forecasts and findings about mitigation investment. Wellington, New Zealand. August 2020.

Quilter, P. W., Ballegooy, S. van., Russ, M., 2015. "The effect of sea level rise on liquefaction vulnerability. Proceedings of the 6th International Conference on Earthquake Geotechnical Engineering, Christchurch, New Zealand.

Peck, R. B. (1969). "Advantages and limitations of the observational method in applied soil mechanics". Geotechnique 19(2)

Stephens, S. (2017). Tauranga Harbour extreme sea level analysis. NIWA, Hamilton

ThinkStep, 2019. "Under construction: Hidden emissions and untapped potential of buildings for New Zealand's 2050 zero carbon goal". Report version 1.3, 16 August 2019.

United Nations (n.d.). "Sustainable Development Goals". Retrieved on 21 August 2020 from https://www.un.org/sustainabledevelopment/

United Nations World Commission on Environment and Development (1987). "Our Common Future". Annex to document A/42/427 - Development and International Cooperation:Environment

Vahedifard, F., Williams, J. M., and AghaKouchak, A. (2018). "Geotechnical Engineering in the Face of Climate Change: Role of Multi-Physics Processes in Partially Saturated Soils". Proceedings of IFCEE 2018 353-364. 10.1061/9780784481585.035.

Vickers, J., Fisher, B. and Nebel, B. (2018). "The carbon footprint of New Zealand's built environment: Hotspot or not?" Wellington: thinkstep. Available: https://www.thinkstep.com/content/carbon-footprint-new-zealands-built-environmenthotspot-or-not

Why use surface geophysical methods in geotechnical investigations?

An opinion of a geophysicist.

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Dr Eva Sutter

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1 INTRODUCTION

Over the last two decades geophysical methods have become increasingly popular for many applications in modern engineering practices, especially in the disciplines of geotechnical and earthquake engineering. 'Making the invisible visible' is more and more where engineering standards, codes of practices, and professionals are heading to. We now live in an era where sites need characterising, where having a realistic 3D model of a site is often regarded superior to scattered 1D information, and where a decreasing number of clients and their insurances are willing to accept risks originating from unknown or insufficiently known ground conditions. This is where the use of surface geophysical methods (i.e. where geophysical signals are recorded from sensors placed on the surface) provides the geo-professionals with a few major advantages; they are non-invasive, cover large areas in relatively short times, and generate continuous 2D and 3D data sets of the underground physical properties. All of which contribute to gaining a better overall understanding of the ground that is developed on.

While it is one goal of this article to highlight the advantages of applying surface geophysical methods to geotechnical engineering questions, it is also an attempt at clearing some of the experienced misconceptions that seem to exist between the disciplines. For the geophysical methods to unfold their full potential it is necessary that their advantages and limitations are well communicated and understood. Early geophysical contractor involvement helps with this and is crucial to ensure the geophysical data is collected in a way suitable to address the tasks appropriately. While there is knowledge about certain geophysical methods in the engineering community, when it comes to geophysics, there is often no 'one size fits all' approach possible and many different factors determine whether an investigation will be successful or not.

The following sections will give a brief overview of some of the 'most wanted' geophysical outputs and demonstrate, with two different examples, the usefulness of integrating a surface geophysical investigation into a geotechnical program early on.

2 SHEAR WAVE VELOCITY AND SURFACE GEOPHYSICAL METHODS TYPICALLY USED IN ENGINEERING APPLICATIONS

The increasingly created need through modern regulations to assess the seismic hazard at a site, have contributed to the increased use of the seismic methods in the engineering industry. One physical ground parameter that is of particular interest to geotechnical and earthquake engineers is the shear wave velocity (Vs). The NEHRP Site Classification, Eurocode 8 and NZS1170.5:2004, to name only a few building standards, all require the use of the shear wave velocity over the first 30 m of the underground (i.e. Vs30) to classify a site. NZS1170.5:2004 ranks the surface geophysical methods of obtaining Vs30 second in a list of seven options. Only the use of invasive geophysical methods that do not require the velocity to be obtained by inversion of the geophysical measurements (i.e. a mathematical process of matching the geophysical data to a ground model) are considered a better choice. However, the latter methods are invasive and only point specific.

There are a few surface geophysical methods available to measure shear wave velocity, the most prominent of them probably is the Multichannel Analysis of Surface Waves (MASW). MASW is often used to produce 2D images along straight profile lines. It is important to understand though that the test is a 1D measurement that produces a shear wave velocity versus depth profile at the midpoint of the total active geophone array. The 2D sections are then obtained by interpolating between several closely spaced MASW tests along a profile line. While there are standards for measuring Vs in situ with invasive geophysical methods or in the laboratory, no standard exists to date that is setting out how to measure this property by means of using surface geophysical methods. This leaves the geotechnical professional with little control over the geophysical data quality. Some guidelines (e.g. NZ Ground Investigation Specification, 2017) set out a framework for what shall be included in a surface geophysical report. However, these guidelines completely ignore any aspects of good data acquisition or processing practices. Two very crucial steps to generate a meaningful data set. Choosing to work with a qualified and experienced geophysicist is therefore highly important. A good source of information for experts and non-experts alike is the guideline for surface wave analysis (Foti *et al*., 2017) developed by an international panel of expert users.

Another seismic method often used in engineering applications is seismic refraction. This recovers the P-wave velocity (Vp) profile of the ground by extracting the refracted P-wave information of a seismogram (i.e. first signal arrivals). Several different ways of analysing this data exist, with the more sophisticated technique of seismic refraction tomography (SRT) being able to recover a detailed 2D Vp model of the underground. The model data can be used directly as P-wave velocity in engineering calculations, as a priori information in MASW processing, or can be interpreted in terms of geological units. An ASTM standard is available for seismic refraction testing (ASTM D5777-00).

Probably one of the most used non-seismic surface geophysical method (together with Ground Penetrating Radar (GPR) which is not discussed here) is the electrical resistivity tomography (ERT). While of less interest from an earthquake or liquefaction point of view, the method is a lot more economic to apply (as compared to seismic) and produces 2D profiles or even 3D volumes of underground resistivity distribution in a relatively short time. This information can be used to extrapolate seismic testing to other areas of a site and is often translated to geological interpretations. The measured electrical resistivity as a physical ground property is also used in earthing design calculations. ASTM D6431-18 provides a standard for this method.

3 EXAMPLES OF INTEGRATING SURFACE-BASED GEOPHYSICS IN GEOTECHNICAL INVESTIGATIONS

For larger scale projects, and especially when budgets for ground investigations are tight and only allow for limited geotechnical testing, surface geophysical investigations can make a tremendous difference in the volume of acquired knowledge about the ground. However, often the scope for geophysics is unclear and regarded as a simple add-on to the invasive testing program. This then becomes quickly too expensive and is discarded all together. In contrast, if a geophysical scope is integrated as part of the geotechnical program and used as a tool to connect fewer dots and fill in the data gaps in a smart way, budgets can be satisfied while gaining a more comprehensive image of the ground. Two different examples are used to illustrate this principle. Both examples are taken from real projects, though, due to respecting client confidentiality no specific details can be given.

EXAMPLE 1: SEISMIC RESILIENCE AND GEOLOGICAL STRATIGRAPHY

Owners (and their insurers) of infrastructure, structures and buildings with high importance levels (e.g. IL3+) increasingly consider undertaking seismic resilience assessments for their assets to enable them to manage the risks and protect the structures by improving the ground, where necessary and possible, before a potential disaster occurs. In this example, an approximate area of 30 hectares needed to be assessed by the geotechnical engineers. Existing boreholes and CPT data were available in certain areas of the site and needed to be complemented by a new site investigation filling in the information gaps. The site is located on reclaimed land spanning several decades of reclamation stages.

The strategy of the geotechnical lead was to use surface geophysical testing (MASW) not only as a means of obtaining in situ shear wave velocity information, which was then used to determine the site class across the area, but also as a reconnaissance tool for positioning the new invasive borehole and CPT testing. This approach has the advantage that areas of concern can specifically

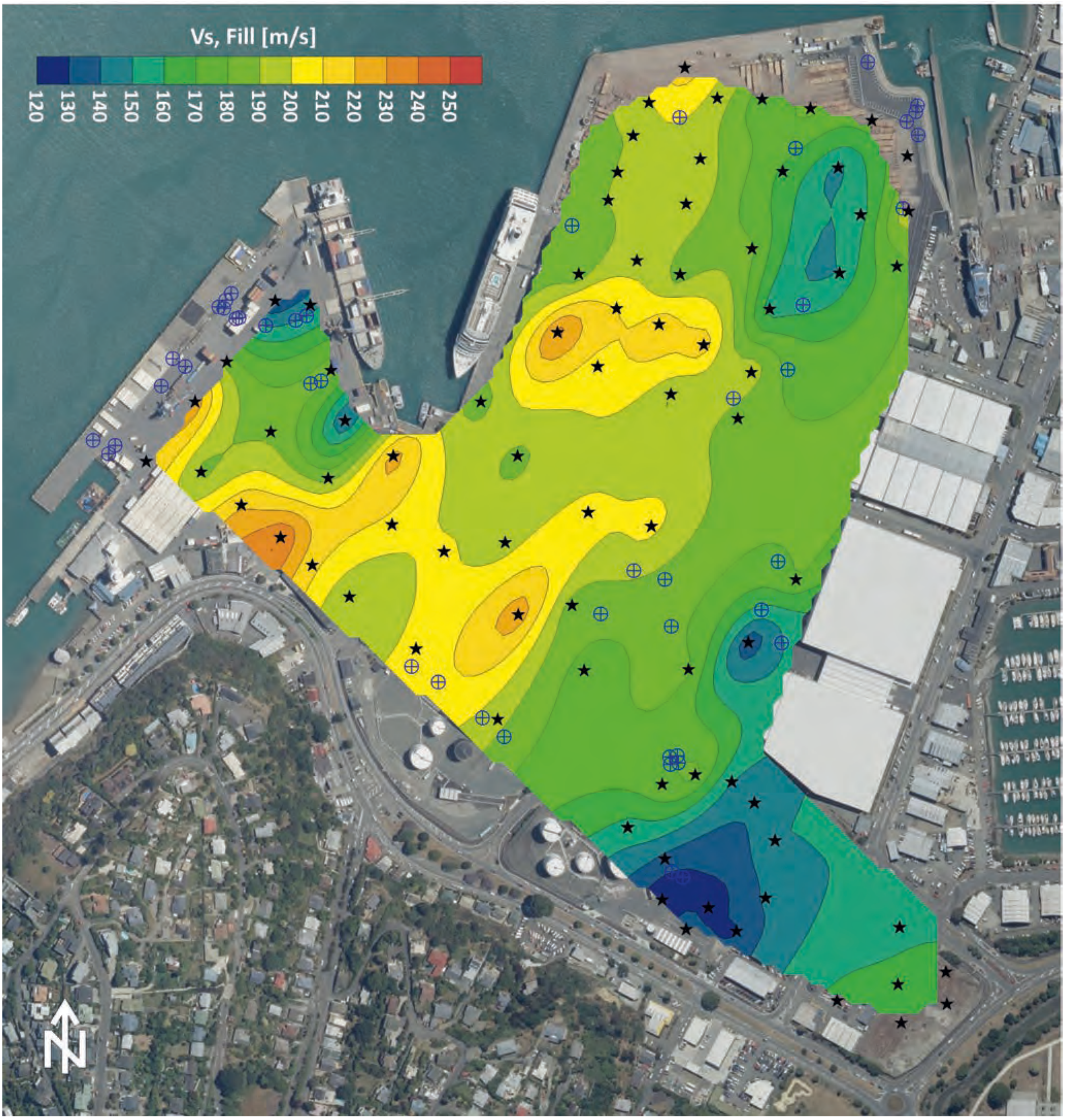

FIGURE 1: Contour map of the average shear wave velocity (Vs) of the reclamation fill produced from 79 MASW Vs measurements (black stars mark the test locations) using Kriging interpolation. Invasive CPT and borehole data were available at the locations marked by blue crossed circles (i.e. pre-existing data).

be targeted with the more expensive (by volume of information gained) invasive tests rather than randomly positioning these within the boundary of the site.

For covering large, laterally inhomogeneous areas, a few 2D profiles may not give a complete image of geological variations, especially in a situation where land was reclaimed over several stages and with different fill material. Therefore, the MASW testing was spread out across the site to cover a larger part and provide a pseudo-3D understanding of lateral Vs and geological stratigraphy changes across it. Figure 1 shows an example of an interpolated contour map depicting the average shear wave velocity of the reclamation fill (first ~4-5 m bgl). Areas of potentially softer or denser material are easily identifiable this way.

The MASW shear wave velocity models were also interpreted for geological units with the aid of the already existing invasive data. Then the identified layer boundary depths were used to produce contour maps similar to the one shown in Figure 1. Although, the geophysical data were not collected along specific profile lines, using these interpolated maps of depth

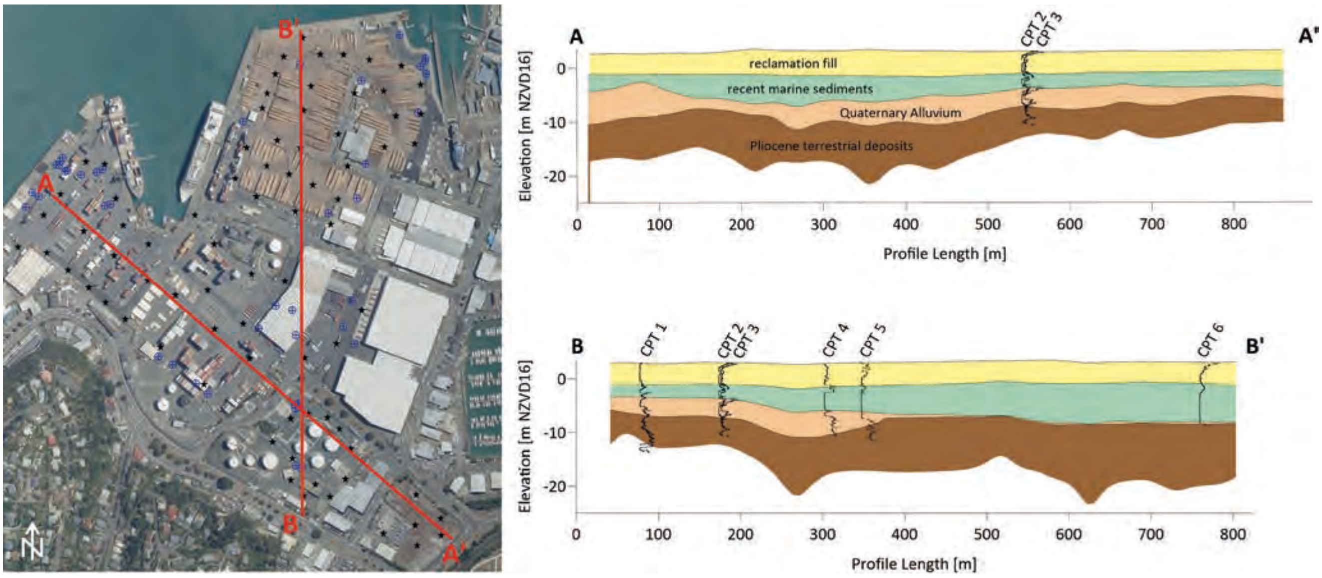

FIGURE 2: Two example cross-sections drawn from the MASW models and interpolated depths to geological layers. Different colours mark different geological units. The boundary to white below the deepest layer marks the maximum depth of the MASW data. Existing CPT logs were superimposed where the cross-sections crossed these or were located close by. Cross-sections were drawn through areas with a higher density of MASW data available to make them less dependent on interpolation.

to specific geological boundaries also allowed to draw cross-sections (see Figure 2) which provided a better understanding of vertical and lateral boundary changes in the vertical plane view. Furthermore, seismic refraction data (Vp) collected along with the MASW data provided an estimate of the depth to groundwater. This information and existing invasive data were used during the inversion of the MASW models to combat some of the non-uniqueness of the inversion procedure and hence making the models more robust.

Close collaboration between the geophysicist and the geotechnical engineer ensured that the geophysical data set was used to its full potential. As is indicated in Figure 2, the investigation depth achieved with the surface based MASW method did not everywhere provide the attempted 30 m below ground level (averaged around 20-25 m bgl) due to site and source restrictions. The geotechnical engineer then obtained Vs30 by extending the MASW Vs models to 30 m using the Boore *et al.* (2011) correlation.

New borehole, CPT, and seismic CPT (sCPTu) test locations were decided on after release of the geophysical draft report. The latter were used to compare to the MASW obtained shear wave velocities. These matched greatly and gave further confidence in the comprehensive data set. One of the main advantages of the surface-based geophysical method was that the achieved depths were significantly larger as compared to sCPTu. This was due to the CPT refusing as shallow as 4 m bgl (max. depth reached at the site was 17 m). In addition, MASW data was collected in areas where the CPT or borehole drill rigs were not permitted. The flexibility to operate in a large variety of terrain and environments is a very useful characteristic of the surface geophysical methods in general.

EXAMPLE 2: FEASIBILITY & RESOURCE CONSENT INVESTIGATION FOR LARGE-SCALE DEVELOPMENT

When medium to large size developments are in the feasibility or consenting stage, often a site-wide understanding of the site conditions, geology and groundwater level is needed. Obtaining a thorough image of the underground and being able to characterize a site early on can help to plan for unsuitable or problematic ground conditions, price a project correctly (e.g. earthworks costs), and avoid under- or over-designing the planned structures. In this example, obtaining a reliable indication of the depth to basalt was crucial for the client to determine if developing the desired site was feasible due to large excavation volumes needed in some of the areas. As a secondary task the geophysical information was used to assist with site classification and the identification of the distribution of clays, silts, and sands. Invasive and non-invasive tests were arranged in a grid-like way to get a relatively dense coverage of the 60-hectare site.

Because the seismic methods are generally the more expensive of the geophysical tests, the bulk of the geophysical investigation was done using ERT. This allowed to cover a lot of ground with real 2D profiles in a relatively shorter time. The information was used to distinguish between clayey, silty, and sandy soils as well as identifying the boundary to basalt. The SRT method was then used to get a separate model of the groundwater depth (Vp ~ 1500 $^{m}/_{s}$) and the depth to basalt (here, Vp > 2500 $^{m}/_{s}$), and to constrain the ERT model interpretation along several profile sections across the site. At each of the SRT profile locations, MASW data were collected in addition. The Vs information further helped constraining

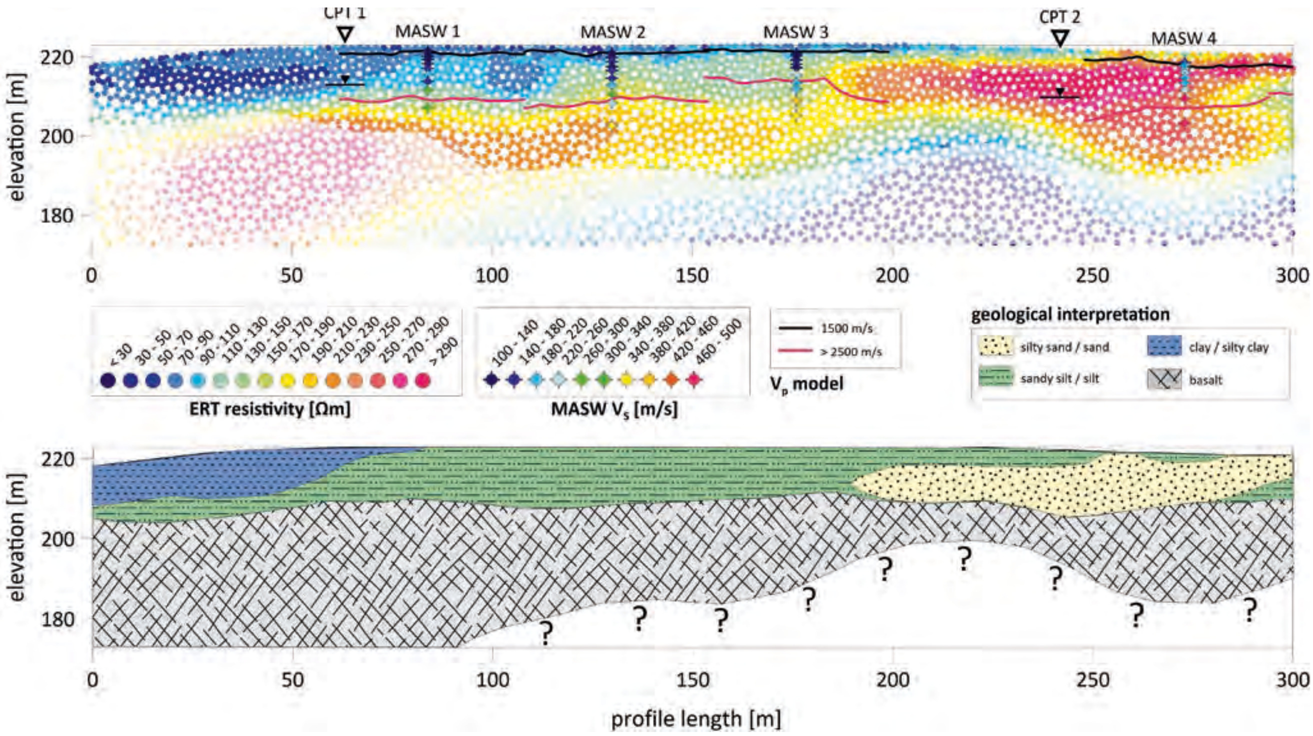

FIGURE 3: The top image shows the geophysical models of ERT (background colours), SRT (black and pink lines), and MASW (coloured cross dots). Where CPT (and borehole) data were available, these were superimposed on the geophysical data for aiding the interpretation of ground material types. Using all available information across the site geological models were drawn as shown in the bottom image.

the ERT and SRT model interpretations and provided a secondary indication of the liquefaction potential across the site (in addition to CPT).

In Figure 3 an example of a 300 m section of one of the geophysical profiles is given. The background colours of the top image are representing the ERT model (i.e. resistivity). The SRT Vp models are superimposed with black and pink lines for interpreted groundwater and basalt boundaries, respectively. MASW models are imaged with coloured cross-dots (vertical lines). CPT and borehole information were superimposed on the geophysical models where available. All information was then used to 'translate' the geophysical models to geological units as shown in the example bottom section in Figure 3. ERT helped to understand the large-scale geology with depths of up to 40 m bgl obtained. The seismic methods were focused on shallower depths of around 10-20 m bgl.

To have all this information was extremely valuable because the ground turned out to be highly variable not only in the vertical, but especially in the lateral dimensions. This was evident from the invasive testing (boreholes and CPT's) and confirmed by the continuous geophysical data that connected these points (e.g. between CPT 1 and CPT 2 in Figure 3). Using the borehole information and the interpreted boundaries to basalt (and groundwater) along the geophysical profile lines, contoured maps of the depth to basalt (see Figure 4 for an example) were produced to give a quick overview of this information to the geotechnical and civil engineers. They were then able to overlay this information with the planned cut and fill contours in CAD to establish whether the excavations were likely to intersect basalt.

At this site using one geophysical method alone would not have been sufficient to discern between some of the ground materials (e.g. high resistive sand/gravel and weathered basalt in ERT), which highlights that depending on the task, or several tasks, of a geophysical investigation, more than one geophysical method may be necessary or beneficial. A geophysicist can design the required geophysical program according to targets and budgets and ideally should be consulted by the geotechnical engineers before finalisation of the geotechnical investigation program.

4 CONCLUDING REMARKS

With an expanding global population, the need arises more and more to develop land that may not be optimal for infrastructure to be built on (e.g. on swamps, near fault lines, on liquefiable soils, on or close by instable slopes, etc.). It will become increasingly important to ensure the risks associated with the underground architecture is well known and can be addressed in the design. The advantages of using surface geophysical methods as part of a geotechnical investigation for such assessments are versatile as demonstrated by the two examples presented in this article. In summary, these are:

- Non-invasiveness and little restrictions regarding accessibility in difficult terrain (e.g. steep slopes, dense vegetation, areas where invasive testing is not possible/allowed).

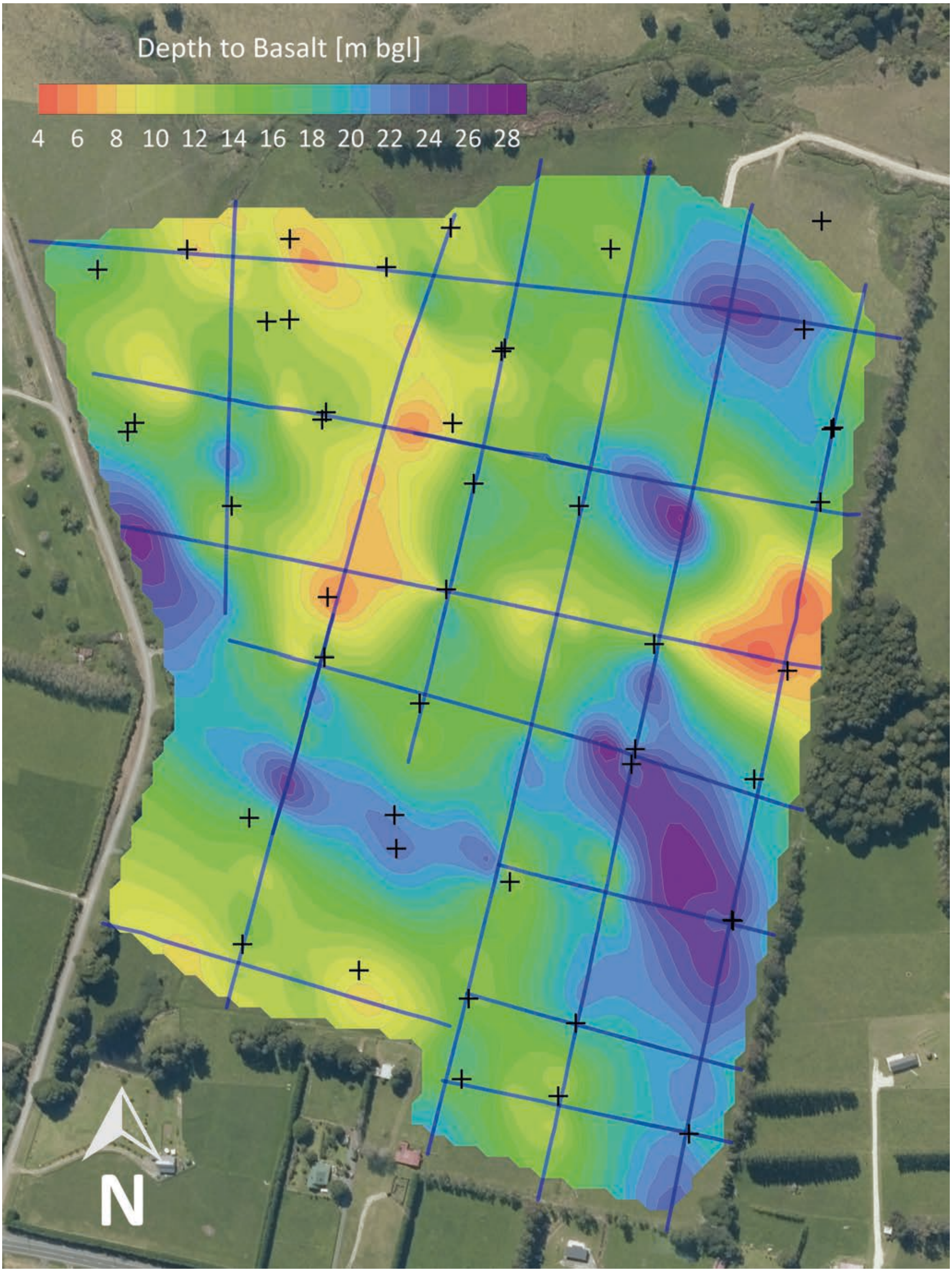

FIGURE 4: Interpreted depth to basalt across one part of the site. Interpretation was made base on geophysical and invasive data available and is interpolated in areas where no data was available. Geophysical data were collected along the blue lines. The longest profiles measured 700 m (for scale). Black crosses mark the locations of CPT and borehole testing.

- Depth coverage vs. associated cost (i.e. there is no correlation between cost and obtained depth, hence a profile to 40 m depth may cost the same as a profile to 10 m depth).
- Time spent vs. data volume obtained (e.g. a 200 m 2D ERT profile to 30 m depth may take around 4 h to collect and analyse, whereas a series of boreholes to cover the same volumes would take considerably longer (apart from possibly being completely unfeasible)).
- Power to identify vertical and lateral geological variations at the same time (i.e. 2D & 3D capabilities).

It is also important to highlight that there are, as with everything in life, some limitations to the surface geophysical methods. These are mainly related to the indirect nature of obtaining the ground models (i.e. the non-unique character of the inversion procedure) and mean that surface geophysical information may have a lower resolution than invasive methods. This is especially true for data obtained at larger depths, an increasing distance away from the geophysical signal source. However, if used by experienced professionals and regarded as a tool to optimize a geotechnical program, the advantages of using these methods outweigh the limitations by far. After all, only greater data volumes will generate a higher information density which in turn provides a better understanding of a site.

REFERENCES

ASTM Standard D5777-00, 2000: *Standard Guide for Using the Seismic Refraction Method for Subsurface Investigation.* ASTM International, West Conshohocken, PA, 2000

ASTM Standard D6431-18, 2018: *Standard Guide for Using the Direct Current Resistivity Method for Subsurface Site Characterisation.* ASTM International, West Conshohocken, PA, 2018

Auckland Council (and collaborators), 2017: *New Zealand Ground Investigation Specification. Volume 1: Master Specification.* Auckland Council, New Zealand.

Boore D., Thompson E., and Cadet H., 2011: *Regional Correlations of VS30 and Velocities Averaged Over Depths Less Than and Greater Than 30 Meters.* The Bulletin of the Seismological Society of America. 101. 3046-3059. 10.1785/0120110071.

CEN 2004: *European Standard EN 1998-1: 2004 Eurocode 8: Design of structures for earthquake resistance. Part 1: General rules, Seismic action and rules for buildings.* European Committee for Standardization, Brussels, Belgium.

Foti, S., Hollender F., Garofalo F., Albarello D., Asten M., Bard P.-Y., Comina C., Cornou C., Cox B., Di Giulio G., Forbriger T., Hayashi K., Lunedei E., Martin A., Mercerat D., Ohrnberger M., Poggi V., Renalier F., Sicilia D., and Socco V., 2018: *Guidelines for the good practice of surface wave analysis: a product of the InterPACIFIC project.* Bulletin of Earthquake Engineering, 16:2367-2420

Building Seismic Safety Council, 2003: *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (FEMA450).* 2003 Edition. National Institute of Building Sciences, Washington, D.C., USA.

New Zealand Standard NZS1170.5:2004, 2016: *Structural design actions. Part 5: Earthquake actions – New Zealand (Incorporating Amendment No. 1).* Standards New Zealand – Paerewa Aotearoa, Wellington, New Zealand.

Update on New Zealand National Seismic Hazard Model

M C Gerstenberger, GNS Science (on behalf of the team)

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1.0 INTRODUCTION

A revision project for the New Zealand National Seismic Hazard Model (NSHM) has recently commenced.
In this project, funded by the Ministry of Business, Innovation and Employment (MBIE) and the Earthquake Commission (EQC), we expect to deliver a revised NSHM by 31 August 2022.

The primary deliverables are: (1) a revised NSHM in an open-source computational library, (2) a web tool to make the NSHM and its results openly available and (3) peer-reviewed documentation.

The project design has been guided by the last decade or more of scientific research in seismic hazard modelling in New Zealand and internationally, by a 2017 international review of the NSHM and through interaction with key stakeholders.

This paper is a summary of GNS Science Report 2020/38 which can be downloaded at https://www.gns.cri.nz/NSHM/project-outputs. The full report includes comments and recommendations made by the Technical Advisory Group.

1.1 NSHM PROJECT STRUCTURE

There are three main constituents within the project structure:

- Science working groups
- Technical Advisory Group (TAG)
- Project Steering Group.

There are four main science working groups, each having sub-groups for specific tasks. The Core NSHM group leads the overall direction of the model, including the model integration. The Seismicity Rate Models (SRM) working group develops the models that forecast magnitudes and locations of earthquakes. The Ground Motion Characterisation Models (GMCM) working group develops the models and methods that estimate the shaking that an earthquake will produce, including any local site effects. The Service Delivery working group ensures that we have software and hardware solutions to the problems we need to solve.

The full science team (all working groups) comprises more than 50 scientists from across New Zealand, Australia, the USA, Canada and Europe.

The TAG consists of 16 members who are tasked with providing technical advice to the science teams. TAG membership is roughly evenly split between hazard scientists and technical endusers. The TAG's role is to ensure that the NSHM is based on best-available science and that it provides the most useful outputs for end-users; this includes input into the development of the framework laid out in this document. As part of this task, the TAG works as a participatory review panel and will provide ongoing peer review of the NSHM as it is developed. The TAG also provides the first port-of-call for end-user engagement, and TAG members are expected to engage with their wider communities on NSHM topics.

The Project Steering Group has membership from GNS Science, MBIE and EQC. This group provides governance for the NSHM Programme.

1.2 END-USER ENGAGEMENT

Interaction with the end-user community is required for several primary reasons. Firstly, we must ensure that the outputs of the model are useful and provided in appropriate formats for users. Secondly, we need to understand and consider any impact that science decisions made by the NSHM team may have on end-users. Finally, success of the NSHM requires continual dialogue so that the end-user communities understand the NSHM and the implications of the hazard estimates it provides.

The TAG is our primary contact for our end-user engagement; however, we intend to be in regular communication with New Zealand technical societies and to have direct engagement with end-users from across sectors. A series of end-user workshops are planned throughout the next two years to introduce the NSHM and to receive feedback on the proposed NSHM outputs.

1.3 PURPOSE OF THIS DOCUMENT

The purpose of this document is to describe the major components of work we intend to undertake prior to delivering a revised NSHM in August 2022. It details the overall philosophy of the model and the main components that we will explore and implement within the NSHM. The document is not intended to fully specify all details of the NSHM project but should provide enough insight for the reader to have an overview of the scientific components of the project and to tease out any further questions the reader may have.

Over the last five months, including with input from the TAG, we have identified the work packages we will pursue over the next two years. In many cases, the work is well underway. As part of the prioritisation work of the last months, we have identified work that needs further understanding (either scientifically or from the enduser community) before final priorities can be determined, or other work which may require redirection during the course of the NSHM development. In redirecting our work, we have developed alternative and aligned procedures. As such, this document reflects the view of the NSHM team at the time of writing and is subject to change based on further scientific understanding and community and stakeholder engagement. A primary purpose of this document is to present the plan for development of the NSHM in the next two years and to promote the engagement process.

2 OVERALL MODEL CONSIDERATIONS

2.1 EPISTEMIC UNCERTAINTY

To convey the full extent of our knowledge of hazard to end-users, it is necessary that epistemic uncertainty be considered, modelled and presented to end-users in a digestible form. However, within a two-year project (or any length project for that matter), it is impossible to completely explore and quantify the full range of epistemic uncertainties. Our goal over the next two years will be to evaluate and model what can be considered, with manageable effort, to be the largest and most critical uncertainties affecting hazard estimates. This exploration will not be exhaustive but will aim to provide indicative confidence bounds on the hazard estimates. The form in which the uncertainties will be provided to end-users is yet to be determined. We envision that fractiles will be considered, as well as less rigorous approaches. We also note that, while the need to provide useful and rigorous estimates of epistemic uncertainty in NSHMs is recognised internationally, limited efforts have been made to provide a thorough investigation and developing a feasible plan to address epistemic uncertainty in the 2022 revision requires additional consideration, so we expect the thoroughness of the uncertainty quantification to increase in future NSHM revisions.

2.2 SUBJECTIVITY AND EXPERT JUDGMENT

As with any scientific project, there is no escaping the need to use expert judgment in the development of the NSHM. Expert judgment in the NSHM ranges from minor un-influential decisions around parameter choices to highly influential decisions around, for example, logictree weights. It is not practical or pragmatic to require formal elicitation of expert judgment through all aspects of the NSHM development. Formal elicitation will be reserved for the weighting of logic trees and will follow guidelines from Christophersen and Gerstenberger (Forthcoming 2021), which is similar to what was applied in the development of the Canterbury Seismic Hazard Model (Gerstenberger et al. 2014, 2016) and other recent work. Subject-relevant experts from the NSHM team will provide weights via a structured expert elicitation process (Cooke 1991). For aspects in which we cannot apply structured elicitation, appropriate checks and balances to minimise bias (e.g. as described in Christophersen and Gerstenberger, forthcoming 2021) are required and are provided by the use of open team-based decision making and by participatory peer review by the TAG.

2.3 MODEL DELIVERY TIMELINE

We aim to have a draft of all SRM sub-models of the NSHM ready by July 1, 2021. In the eight months following, we will refine the draft models and endeavour to have a complete draft SRM by February 28, 2022. The GMCM framework is proposed to be complete by August 31, 2021, with a draft GMCM available by February 28, 2022. The final NSHM is due by August 31 2022, allowing six months for: (1) final review of the model and its implications, (2) further engagement with the technical end-user community and (3) final steps of the participatory review process by the TAG. NSHM outputs will be available online following completion of the project.

3 SEISMICITY RATE MODEL

3.1 TIME-DEPENDENCE

The degree to which we will explore time-dependence is not yet decided. However, we are not intending to explore the implementation of a short-term clustering model, which would require regular updates on, for example, yearly cycles. The NSHM will be based on a static earthquake rate forecast for the time-window of interest. The time-window will be adaptable to specific needs but will be based on the same forecast information, and the forecast will not be updated during the life of the 2022 NSHM (i.e. we are not considering a 'living' model).

Over the next 18 months, we will consider the application of three forms of explicit time-dependence.

1. We will consider the conditional probability of rupture on a small selection of well-studied faults, as we have done in some past versions of the NSHM. This involves using uncertainty distributions of slip-rate, single-event displacement, times of past fault rupture and time since last rupture to estimate a pseudo-Poisson rate for the time-period of the forecast. Our default starting point will be the work of Rhoades and Van Dissen (2003), which uses a combination of Log-normal, Weibull and Brownian Passage Time recurrence-time models to estimate the conditional probability of rupture on a fault. Our effort will build upon the Kaikōura Seismic Hazard Model, and we will consider approximately 12 major faults (or others as data, time and priorities allow).
   - In deciding if we should apply these rates, we will explore the quality of timing data; the consistency of the conditional probability assumptions, which assume semi-regular recurrence of earthquakes, with our overall NSHM assumptions; and whether a time-dependence model is ultimately desirable.
2. Past NSHMs (in New Zealand and internationally) have not considered the impact of aftershocks on hazard and, in fact, have explicitly removed their impact. This results in a systematic underestimation of the estimated hazard. More recently, the Boyd (2012) method allows for adding aftershock rates that are conditional on the occurrence of a main shock; in other words, it allows for accounting for potential aftershocks prior to their occurrence. This is not a model that requires updating and can be thought of as modelling potential aftershocks in a consistent manner with the modelling of main shocks.
   - Before applying the Boyd method to the final NSHM, we will consider its robustness, our ability to apply it in New Zealand and the impact on ground-motion model calculations (e.g. potentially ignored reductions in within-event uncertainty). Implications to the end-user community must also be considered.
3. We will consider the impact of medium-term clustering on expected earthquake rates over the time-period covered by the NSHM. Most major earthquakes are preceded in the medium term by an increase in the magnitude and rate of minor earthquakes in an area similar to that occupied by the eventual aftershocks. This increase takes place over a period ranging from months to decades, depending on magnitude. The 'Every Earthquake a Precursor According to Scale' (EEPAS) model (Rhoades and Evison 2004) estimates the increase in earthquake rate expected when an earthquake of any magnitude occurs, based on observed relations that every earthquake may be a precursor to a larger one to follow. In constructing the seismicity rate model, we will use the EEPAS model to estimate the net effect of medium-term clustering on the expected rates at any magnitude and location for the forecast period of the NSHM, not the time variation of rates within the period.

The NSHM will include other data sets and decisions that are impacted by the temporal variability of seismicity and other related observations (i.e. non-stationarity of earthquake occurrence and earth processes). This is true of any NSHM. We will aim for a consistent philosophy on how we apply data sets and assumptions and will document known and implied impacts of these assumptions. The final form of the model is yet to be determined, but we will aim to produce one or more of the following:

a. A model that attempts, as well as possible, to remove all explicit and implied timedependence (e.g. a hazard forecast for *any* 50/100 years).
b. A second model that includes known time-dependence (e.g. a hazard forecast for *the next* 50/100 years).
c. A model that acknowledges the non-stationarity of seismicity and explicitly allows for time-dependent (TD) or time-independent (TI) contributions and which quantifies for this uncertainty.

It is our current opinion that a true TI model (model A) will never be possible due to implications of, and inconsistencies in, datasets used to constrain seismic hazard. Because of this, a model that is transparent and acknowledges the uncertainty from TI or TD assumptions will provide a forecast that is more consistent with our understanding of earthquake occurrence.

3.2 CONSTRUCTING THE SEISMICITY RATE MODEL

The SRM will follow a traditional logic tree structure that combines fault-based models with distributed seismicity models. We will pursue multiple options within each of the two components of the SRM logic tree; these components will aim to capture different hypotheses of how to model the earthquake occurrence process (e.g. epistemic uncertainty). We will initially scope a broad range of models with an aim of reducing the broad set down to a representative and unbiased set.

The fault-based models will be developed using 'inversion' techniques, following the recipe and software developed for the United States Geological Survey Third California Earthquake Rupture Forecast (UCERF3) in their OpenSHA software engine. The main steps involved in developing the inversion models include: (1) development of rupture sets, (2) development of deformation models and (3) defining the constraints for the inversion. Informally, the inversion method has become known as the 'Grand Inversion'. Questions remain about the level of complexity we will be able to include in each of the three steps.

We envision that the biggest challenge to this 'inversion' approach will be the Hikurangi and Puysegur subduction zones; an inversion model has never been developed for a subduction interface before. Furthermore, the size and potential connectedness of the interface to crustal faults brings large complexity to the problem. If we are unable to develop an inversion model that includes Hikurangi and Puysegur, we will use a classical approach for the interfaces instead. Reducing the plausible ruptures on the interfaces, including joint ruptures with crustal faults, is a focus in the early part of the project. We will aim to allow for joint ruptures between each interface and its nearby crustal faults; however, this may prove too challenging in the next two years and may be reserved for future versions of the NSHM. Key research gaps include constraining the plausible joint ruptures and development (or applicability) of source-scaling relations. The Puysegur Subduction Zone will be treated with the same methods as the Hikurangi; however, with a much lower hazard and risk profile, it may be considered a lower priority and require simplification.

3.3 SEISMICITY RATE MODEL WORK PACKAGES

The primary components and work packages to develop the SRM include the following:

3.3.1 INVERSION MODELS

Community Fault Model

A version 1.0 of the Community Fault Model (CFM) is in development. This CFM builds upon Litchfield et al. (2014) and aims to bring in new and missing fault data in a series of community workshops in late 2020.

- Seismogenic depth across New Zealand will be re-evaluated and is expected to contain large changes from previous models.
- The NSHM fault model will be extracted from the CFM with simplification of complex geometries where necessary.
- Uncertainties on dip and also on seismogenic depth will be provided and hazard sensitivity to this will be explored. The impact of this uncertainty may prove to be as significant as rupture length.

Crustal Rupture Sets

The aim of the rupture sets is to define the plausible ruptures that will be considered in the inversion. We will start with the constraints as implemented in UCERF3, with some adaptations as necessary for New Zealand.

- **Plausibility filters:** The plausibility filters aim to reduce the rupture sets down to a number of ruptures that can be efficiently run using High Performance Computing. We aim to adopt UCERF3 plausibility filters as much as possible; however, NewZealand-specific adaptations will be necessary. It is currently unknown if we will pursue Coulomb-based filters (UCERF4 will use a new implementation of the Coulomb filters).
- **Splays:** UCERF3 was limited to only linear ruptures, which do not adequately represent known New Zealand ruptures. We will allow for possible splay ruptures and will work with UCERF4 on this.

Hikurangi Rupture Sets

Similar to crustal faults, we must define the starting sets along the entire interface rupture. This is something that UCERF3 was not required to do due to the tectonic setting of California.

- **Plausibility filters:** Instead of starting with a nominally comprehensive set of ruptures, as for crustal faults, we will build a limited set of ruptures using a limited range of aspect ratios and reduce these down to a representative set.
    - We will also explore the impact of uncertainty in constraint of up-dip and down-dip locking of the interface and uncertainty in the shape of the interface (e.g. depth beneath Wellington).

Puysegur Rupture Sets

- We will follow similar methods as for Hikurangi; however, with fewer constraints and a lower risk profile for New Zealand, Puysegur is considered a lower priority than Hikurangi in the 2022 revision.

Joint Crustal-Hikurangi Ruptures

This work is given a lower priority than developing independent ruptures. Potential implementation paths include: no joint rupture, limited/selected key joint ruptures (e.g. Wairarapa + Hikurangi) or a more extensive and systematic suite. Which of the paths chosen will be based on available time once successful independent results can be confirmed. This will include developing an understanding of the sensitivity of hazard to joint ruptures.

Deformation Models

The deformation models provide the fundamental slip rate information for all faults in the fault model.

Earthquake Geology Deformation Model

Known slip rate data is being compiled with the CFM and will be applied to fault sections. Uncertainty estimates will be included in the form of a best estimate with upper and lower credible bounds. Ideally, the Earthquake Geology Deformation Model will be independent of the Geodetic Deformation model to maintain independence in the logic tree; however, for faults with unknown slip rate, at this point we have no known approach to use other than Geodetic estimates. We currently anticipate developing a single Earthquake Geology Deformation Model.

Geodetic Deformation Models

We will focus on using backslip-based geodetic deformation models to determine contemporary moment accumulation rates on the faults in the fault model (i.e. we will be determining slipdeficit rates). The backslip rates will be obtained for all faults in the source model by fitting to geodetic strain rate models using four different approaches to derive strain rates from the existing geodetic velocity field. A subset of the backslip models will be coupled with a traditional block modelling approach (constrained by the geodetic velocity field rather than strain rates), which will be used to estimate the backslip rates for the Hikurangi subduction zone and other major faults. We will pursue alternative approaches to partitioning the block model slip rates and backslip model slip rates to generate a suite of geodetically based slip deficit rate models. This will allow us to understand the impact of modeling methodologies and data uncertainties on the variability and uncertainties in the strain rates and slip deficit rates.

The output suite of deformation models will need to be reduced to a representative suite of models. We will investigate multiple methods to represent the model space in the least biased way (e.g. combining strain rate models, combining deformation models, developing a 'backbone' model and/or by weighting a suite of models). We will also explore utilising the residuals to the backslip models as input to the background seismicity model to characterise offfault deformation rates

3.3.2 DISTRIBUTED SEISMICITY MODELS

Distributed seismicity models allow for earthquakes on unknown faults. A recent study estimated that about 50% of major active faults in New Zealand may be unknown (Nicol et al. 2016). We will pursue two approaches: (i) hybrid gridded models and (ii) uniform area zones.

Earthquake Catalogue

We will develop a revised earthquake catalogue based on the GeoNet catalogue but with homogenised *Mw*. The aim is to develop a catalogue that is homogeneous in magnitude but with variable completeness, back to the beginning of the GeoNet catalogue (~1800). Pre1900 data may have to be used as is, and we will determine if the large uncertainty introduced in this data brings sufficient information gain.

Hybrid Model

We will use multiplicative hybrid modelling techniques to develop a gridded seismicity model that includes multiple data sources and other sources of uncertainty. The hybrid modelling technique allows for multiplicative or additive scaling of multiple models and data sets. The scaling is optimised in a learning period and tested in independent time periods.Testing will occur both forward and backward in time. How epistemic uncertainty from the hybrid will be expressed in the final SRM needs to be considered. Components of the final distributed hybrid model will likely include the following:

- Smoothed Seismicity (PPE; proximity to past earthquakes) – a similar approach to a density-based smoothed seismicity model.
- Uniform Poisson model – typically used as a base model of least information of the hybrid.
- Geodetic Strain Rate – four strain rate maps will be considered as input into the strain rate hybrid. The potential for double counting of strain between the Hybrid and the Geodetic Deformation Model will need to be understood and corrected.
- EEPAS (every earthquake a precursor according to scale) – a clustering model that has peak information over roughly a 15–20 year time-frame. After 20 years, the information in the model decays. This model adds significant forecasting skill over smoothed seismicity models. The rate will be applied to the model by calculating, for example, a mean annual rate for the forecast time of interest (similar to conditional time-dependent rates on faults). We will not update the EEPAS rates during the life of the 2022 NSHM.
- Other data inputs – proximity to mapped faults (PMF), proximity to plate interface (PPI).
- Catalogue options:
    - Declustering. There is no true declustering method, and the choice of method can have a significant impact on the final estimated hazard, particularly in regions dominated by the distributed seismicity mode. We will explore multiple declustering methods and assess their impact on hazard estimates. If the Boyd

(2012) method is used, we will need to ensure consistency with that method. Non-declustered catalogues will also be considered.

- Time-dependence. The choice of time-length of the catalogue used has an impact on the final hazard forecast. Any earthquake catalogue has insufficient data to ensure a random and robust sample and will be impacted by non-stationarity of earthquake rates. In other words, the longest time period may not be any more likely to represent long-term earthquake rates than a shorter time period. Time variability represents an uncertainty in the hazard estimates and will be explored. Additionally, magnitude uncertainty and threshold of completeness increases significantly as the age of the catalogue increases, particularly pre-1900. Increased magnitude uncertainty may bias the hazard upwards. The trade-off of this increased uncertainty with the additional information it provides needs to be considered.

Regionalised and Uniform Area Zone Model

Past New Zealand NSHMs have relied solely on smoothed seismicity models to estimate distributed seismicity rates. Recent work (Gerstenberger and Schorlemmer, in revision) has shown that, for low seismicity regions, zones with constant rate are likely to have more forecast skill than smoothed seismicity models. We will therefore develop an alternative model to the hybrid based on uniform area zones that contain a uniform earthquake rate.

- Gutenberg-Richter (GR) parameters: each zone will contain a single GR a-value and bvalue. The 2012 NSHM used area zones for b-value calculations.
- Seismotectonic Zones
- Other seismicity parameters will also be regionalised and will be applied to both the uniform area zones and hybrid seismicity models for the hazard calculations:
- Maximum magnitude within a zone.
- Preferential strike: we will consider if there is evidence for preferential strike within a zone. A magnitude dependent function will be considered.
- Faulting mechanisms.
- Hypocentral Depth distributions.

3.3.3 SLAB MODEL

Development of the slab models for Hikurangi and Puysegur have been a low priority to date. We currently consider two options, a 3D gridded model based on Stirling et al. (2012) and alternative options based on OpenQuake capabilities that require further understanding.

3.3.4 CONDITIONAL AFTERSHOCK MODEL

We will explore the appropriateness of the application of the Boyd (2012) conditional aftershock model, including any implications it may have for end-users. Note: Boyd (2012) is not a time-dependent clustering model and is not similar to what was done in the Canterbury Seismic Hazard Model (Gerstenberger et al. 2014, 2016). It does not require updating and does not model a specific aftershock sequence. This method simply acknowledges that the total NSHM rate of earthquakes is too low when aftershocks are not included.

3.3.5 TOTAL RATE / MOMENT BALANCE

The total rate of earthquakes expected by the NSHM can be considered an output of the model (e.g. as in Stirling et al. 2012) or the NSHM output can be constrained based on expected rates, which is the more common approach. In the UCERF3 model, this constraint contributed the largest uncertainties when loss was considered. When used as a constraint, it implicitly includes time-dependence, unless a truly long-term rate is used (however, this remains unknown). We will consider a range of catalogue, geodetic and geology-based constraints. Both moment- and rate-based constraints will be considered.

3.3.6 MAGNITUDE SCALING RELATIONS

Epistemic uncertainty in the scaling of magnitude as a function of source rupture area will be a feature of the new source model. Multiple relations will be assembled for crustal fault sources and also for interface sources. Weighting for the relations will be informed by their performance in residual analyses. A flatfile of historical large-to-great earthquakes from 1990 onwards is being assembled to provide the basis for the residual analyses.

4 GROUND MOTION CHARACTERISATION MODEL

4.1 2010 NATIONAL SEISMIC HAZARD MODEL

The GMCM largely defines what the details of the outputs of the NSHM look like. For example, the GMCM provides models for a suite of ground-motion intensity measure types, which are of interest to NSHM users. For the 2010 NSHM, the GMCM consisted of a single groundmotion model of McVerry et al. (2006). The McVerry et al. (2006) ground-motion model provided predictions for peak ground acceleration (PGA) and absolute acceleration response spectra at oscillator periods between 0.075 and 3 s.

Ground motion aleatory uncertainty was considered in the 2010 NSHM via the McVerry et al. (2006) standard deviation model but no epistemic uncertainty was considered. With no consideration of epistemic uncertainty, and also influenced by how the SRM was constructed, the outputs of the 2010 NSHM were considered 'best estimate' and not explicitly mean hazard.

4.2 GOALS FOR NATIONAL SEISMIC HAZARD MODEL REVISION

The NSHM revision will improve on the GMCM used for the 2010 NSHM. Particular areas for improvement are:

- to provide models for a wider range of intensity measures that are of interest to NSHM users

- to have a model that better utilises recorded New Zealand ground-motion data, and
- to provide a modern characterisation of epistemic uncertainty.

The latter point is critical for the NSHM to provide estimates of epistemic uncertainty in the hazard results.

The programme of work for the GMCM has yet to be defined beyond the six-month period between August 2020 and January 2021. In this time, the GMCM working group has two goals: to develop a 'minimum viable product' GMCM and to decide the work priorities for the following 18 months.

A 'minimum viable product' GMCM is a draft model that may not be technically satisfying but stitches the pieces of work together in an efficient manner and is able to produce hazard outputs. The benefits of a minimum viable product are that it allows us to determine the type of computational resources we need to run the model, define data format and storage solutions and design the architecture of the software for an efficiently maintained data product.

To inform the work priority decisions for the following 18 months, a suite of tests will be performed. These tests will centre around evaluating the performance of existing models against New Zealand data and the sensitivity of hazard results to various aspects of existing ground-motion models. By testing the models against data, we will understand if New Zealand data differs markedly from existing 'global' ground-motion models and whether adjustments of these models are necessary for reliable application in New Zealand. The hazard sensitivity tests will help us to understand which aspects of ground-motion models have the greatest influence on the final results, for example:

- How important is the large-magnitude scaling of Hikurangi subduction interface ground-motions to the seismic hazard in New Zealand?
- How important are distant Hikurangi and Kermadec interface motions to the long-period seismic hazard in Auckland?
- Do non-linear soil response models greatly influence hazard results in New Zealand's highest hazard regions?
- Do elaborate epistemic uncertainty models influence hazard greatly at probabilities of exceedance that are of interest to New Zealand's NSHM users, as opposed to simpler methods?
- Do path attenuation effects greatly influence hazard results in low-to-moderate seismicity regions of New Zealand?

Depending on how sensitive the hazard results are to these and other effects, we will prioritise our subsequent work accordingly.

4.3 MINIMUM GROUND MOTION CHARACTERISATION MODEL

To get to the minimum viable product GMCM, the working group has separated into three subgroups, each working on:

- the ground-motion database
- the ground-motion models, and
- the previously described hazard sensitivity studies.

New Zealand already has an existing ground-motion database (Van Houtte et al. 2017; Kaiser et al. 2017) that contains nearly all of the major events recorded in New Zealand (Mw approximately greater than 4.5). There is a general desire from the GMCM working group to expand the database to include smaller-magnitude data, thereby increasing the database size. Smaller magnitude data tends to better constrain certain aspects of the model, particularly the spatial variation of path attenuation, and site-specific amplification at seismic recording stations, although these data also come with modelling issues. Perhaps the primary benefit to updating the ground-motion database is to incorporate the large quantity of measured site information (e.g. the Vs30, Z1 and Z2.5 parameters) that has been collected in the past few years. This will allow the models to better partition uncertainty across source, path and site terms and possibly improve median model predictions.

Another improvement to the New Zealand ground-motion database will be a parallel database of simulated shaking estimates that can be used jointly or separately from the observational database. Many GMCMs around the world have simulations underpinning the ground-motion models, particularly the hanging wall and non-linear soil response models underpinning the NGA-West2 models. Unfortunately, these simulated data are not typically made available alongside the recorded ground-motion data, which precludes adjustments and updates of the GMCMs as simulated data methods improve. Any simulated data used in this project will be included in the ground-motion database. The six-month workplan includes setting up this workflow, but how simulated data will be used in the project will become clearer after the hazard sensitivity studies are completed.

The ground-motion models working group is initially focusing on collating all available models and incorporating them into the OpenQuake hazard software. With the recent publication of parts of the NGA-Subduction project, many new subduction interface models are now available, including some specifically fit to New Zealand data. The group will then compare available models (crustal, subduction interface and subducted slab models) to New Zealand data to ascertain whether adjustments are necessary. These adjustments can be facilitated by the Hassani and Atkinson (2017) approach, where adjustments for 'stress drop', path attenuation, site attenuation and crustal amplification are simple to apply. An initial challenge will be determining whether it is necessary to derive a reference rock profile for New Zealand and whether the reference rock profile needs to be constrained with independent datasets.

4.4 HAZARD OUTPUTS

The GMCM largely defines the type of outputs provided by the NSHM. The working group is currently working under the assumption of providing:

- Models for peak ground velocity (PGV), PGA and 5%-damped pseudo-acceleration spectra for 22 oscillator periods between 0.01 and 10 s. These models can be used for mean hazard, hazard disaggregation and mean magnitudes, at a minimum.
- These hazard metrics will be provided for average horizontal motions ('RotD50') and maximum horizontal motions ('RotD100') to take into account horizontal polarisation of ground-motion, as well as the 'larger of two as-recorded horizontal components', for consistency with NZS1170.5:2004.
- Hazard estimates as a function of Vs30. Such parameterisation of site effects is very common overseas but largely incompatible with the NZS1170.5:2004 site class. The NSHM Core Team is liaising with the TAG to best facilitate this issue.

Additional hazard outputs may be able to be provided, subject to budgetary constraints. These additional outputs will be prioritised according to the end-user group on the TAG and include conditional mean spectra (and other conditional intensity measures), inelastic response spectra and other values of damping.

4.5 WELLINGTON BASIN

Sedimentary basins around New Zealand are known to amplify and modify earthquake ground motions. In Wellington, amplification effects arising from the propagation of waves through the Wellington basin were identified as one factor likely to have exacerbated damage during the Kaikōura earthquake (Bradley et al. 2018; MBIE 2017). Basin amplification effects were observed in the 1–2 s spectral period range corresponding to typical fundamental resonant periods of mid-rise structures. The geometry and sediment fill of the Wellington basin is relatively well-characterised (Kaiser et al. 2019) and provides a good case study to investigate ways to model local site and basin amplification effects not always fully captured in NSHM. We aim to gain an understanding of how well new NSHM ground-motion modelling captures basin amplification effects and trial advanced approaches to model these specific effects in Wellington.

Based on the lessons learned, we will provide a suggested roadmap (white paper) to improve how we capture local basin amplification effects in urban seismic hazard nationally.

To develop an understanding of what the final roadmap might look like, we will initially focus on the following:

- Detailed Vs30 maps of Wellington basin, an updated velocity model and the definition of generic velocity profiles as needed for the GMWC modelling subgroup.
- Simulated 3D ground-motions from large events impacting the Wellington basin (e.g. Hikurangi interface, Wellington Fault, Kaikōura earthquake)
- Non-ergodic linear site effects terms at strong motion stations and assessment of how well site/basin effects are captured by modern and updated NSHM ground-motion models
- Assessment of spatially variable amplification by merging results of the above.

5 NATIONAL SEISMIC HAZARD MODEL TESTING

Useful testing of the forecast skill of seismic hazard models remains a difficult challenge. Statistical tests of model components will be included in both the SRM and GMCM model development (e.g. retrospective testing of geodesy and catalogue-based models). A later stage of the project will test the hazard estimates produced from the final 2022 NSHM against observations. A focus will be ground-motion-based testing at sites around New Zealand (e.g. Stirling and Petersen 2006; Stirling and Gerstenberger 2010). Sensitivity tests will also need to be conducted in both hazard and risk space.

6 SERVICE DELIVERY

We have five primary Service Delivery objectives (1–5 below) for the NSHM Revision; these are summarised below. In order to achieve the first four, we will be working closely with the SRM and GMCM working groups to ensure that their science is being incorporated, stored, documented, etc. in a way that makes sense to them and supports the aims above. The final objective, in particular, relies heavily on input from the TAG and wider end-user communities.

In addition to the development of a revised seismic hazard model and delivery of useful and usable results, we are working to support the following aims for the project:

- **Reliability** through implementation of best-practice development and systems, where possible. GitHub is our primary tool in ensuring our codebase is version controlled and tested.
- **Transparency** through collaboration, testing and documentation and open-source data, models and results (e.g. use of GitHub, Slack).
- **Accessibility** to data, models and results for both internal and external users and endusers.

OBJECTIVE 1: THE GRAND INVERSION AND OPENSHA

The development of inversion models by the SRM follows the same (or a similar) recipe as UCERF3 and uses software developed for UCERF3 and OpenSHA. The main steps to achieving that from this group include:

- developing tools to translate the New Zealand CFM into a format appropriate for OpenSHA
- ensuring that we are able to use visualisation tools such as SCEC-vdo in order to understand and analyse our results
- working with the OpenSHA team to understand the OpenSHA codebase (including new code to enable inversion modeling of a subduction zone) and computational requirements in order to implement New-Zealand-specific constraints and to set up sufficient infrastructure to test and run code to create rupture sets consistently
- producing rupture sets, and
- testing and benchmarking the inversion procedure to validate/confirm technical feasibility of the different options for SRM fault-based models (see Figure 3.2).

OBJECTIVE 2: HAZARD CALCULATION

We consider OpenQuake and OpenSHA to be our main options for seismic hazard calculation engines.

Currently, our preference is to use OpenQuake. The GMCM working group is using OpenQuake for the hazard sensitivity analysis to inform its work planning and is developing new NewZealand-relevant ground-motion models for this engine. OpenQuake's wide international user base, integration with risk and current uptake by New Zealand stakeholders are also compelling reasons for this preference. However, several concerns remain:

- Additional work is required to efficiently use inversion models in OpenQuake. Note: work has begun between USGS and the Global Earthquake Model Foundation / OpenQuake on this topic.
- Uncertainty surrounding the long-term plans for continued development of OpenQuake.
- As an alternative, we are confident that OpenSHA will be able to ingest the source models that the SRM working group is aiming to develop. If it is necessary to use only OpenSHA, our understanding is that the GMCM will be able to implement any new models into that engine; however, this needs to be better understood.

We still need to:

- Determine and set up a consistent environment in which we will develop on and run these engines.
- Evaluate and implement a storage system for inputs and results.

OBJECTIVE 3: TOOLS AND INFRASTRUCTURE

In order to achieve our objectives for this project, we are making use of a number of tools to support development and implementation of the NSHM and different aspects of its science. As key examples, we are currently:

- using GitHub for code storage, version control and documentation;
- implementing a continuous integration system around our versions of OpenSHA and OpenQuake in order to run tests and verify that changes we make to those codebases are working correctly;
- exploring higher-power computing options for producing rupture sets and eventually running hazard calculations (e.g. an internal GNS Science cluster, New Zealand super computing resources, cloud resources); and
- using Jupyter notebooks as a tool for developing repeatable tests and sharing codes that can be easily run by less technical users.

OBJECTIVE 4: STORAGE AND SHARING

Other important components that underpin our ability to achieve any of the outcomes of this project are how we store and enable sharing of data and models and how we manage that information and changes to it. These components need to be stored in a way that is flexible, reliable, secure and accessible by the intended users and must be clearly and reliably documented.

Currently, we are exploring cloud storage solutions such as Amazon S3, which would enable scalable, secure, accessible and reliable storage that we can use to develop appropriate solutions for sharing our data and models.

We are also evaluating our datasets, models and codebases in order to develop appropriate data management plans for them. These plans will centrally record our storage and access solutions (including GitHub), as well as evaluate maintenance and update expectations, risks and procedures around these components.

OBJECTIVE 5: RESULTS DELIVERY

A primary output for this project is the dissemination of useful and usable results to our endusers via a publicly available, web-based tool. In order to do this successfully, we need to:

- Determine the infrastructure needs of a web tool that can:
- provide necessary documentation and clearly communicate liability and any limitations/constraints of the results being provided
- accept results from any calculation engine we are required to use and appropriately access our storage solutions, and
- deliver static results and potentially dynamically calculate results with user input in a user-friendly way.

- Confirm with end-user communities that the outputs we believe we need to provide and the formats in which they are provided are appropriate. We are currently aiming to provide:
  - The NSHM sub-components, including SRM component models, GMCM component models (e.g. via OpenQuake) and the logic-tree specifications.
  - Hazard curves: annual probability of exceedance versus shaking (in terms of spectral acceleration with 5% damping) by location, site condition and period.
    - Hazard curves with some level of uncertainty to be determined.
  - Hazard spectra: shaking versus period by location, site condition and probability of exceedance.
    - Hazard spectra with some level of uncertainty to be determined.
  - Hazard maps showing shaking by probability of exceedance, period and for various site conditions (using Vs30, Z1 and Z2.5).
  - Sets of results that are appropriate for risk will also be considered and made available.
  - The above results to be provided in CSV and HDF5 format, ideally with appropriate plots, tables and maps.

Ideally, we would also like to include:

- Disaggregation:
  - Average magnitudes usable by geotechnical engineers.

Other items on the table but not necessarily in scope for this two-year project include:

- vertical spectra
- inelastic response spectra
- conditional mean spectra (and other conditional intensity measures)
- other values of damping, and
- measures of duration.

For the Reference list and the response by the TAG to the framework, including comments and recommendations, see the full report at https://www.gns.cri.nz/NSHM/project-outputs

A Study on Seismic Coefficient of the Mononobe-Okabe Equation and Dynamic Pressure Distribution for External Stability Analysis of a MSE Wall in Auckland Region

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ABSTRACT

This paper describes a study on seismic coefficient and dynamic earth pressure distribution of a geo-grid reinforced Mechanically Stabilized Earth (MSE) retaining wall. Dynamic finite element modeling is implemented on the wall to investigate its dynamic responses under selected historic ground motions for a typical geological condition of subsoil Class C within Auckland area. Results from the analysis are discussed and compared against estimations from conventional Mononobe-Okabe equation that is being applied for external stability assessment of the wall. Horizontal seismic coefficient kh is found to be ranging from 0.32 to 0.52 given that vertical seismic coefficient kv is ignored. It is also revealed from this study that conventionally adopted dynamic soil pressure distribution overestimates dynamic soil pressure within the upper half of the wall but underestimate dynamic soil pressure within the lower half of the wall. A modified profile is thus proposed based on observed increasing dynamic pressure rather than decreasing as assumed in the traditional assumptions, to improve accuracy of predicted dynamic soil pressure. Amplification of horizontal acceleration activated on the interface of wall mass and retained soil is also discussed in the context.

Key Words: seismic design, external stability, MSE retaining wall, Mononobe-Okabe equation, amplification

1 INTRODUCTION

Traditional limit equilibrium pseudo-static approach has been extensively applied for seismic design of earth retaining structures due to its theoretical simplicity and readily implementation. Within this framework, the Mononobe-Okabe theory (1926) is the most popular and typical approach for estimation of dynamic earth pressure under a ground motion defined by a horizontal seismic coefficient a_h and a vertical seismic coefficient a_v. To date, only limited research has been made aimed at improving its practical application for the design of geosynthetic reinforced retaining soil structures. Cai and Bathurst (1995) reported that conservative total dynamic forces were calculated using Mononobe-Okabe method compared to results yielded from finite element analysis. However, in their study, no quantitative assessment was made to describe this. Their study also explored small variations in the peak accelerations along the height of the wall selected for the analysis, which can be considered as an evidence of taking a single acceleration coefficient in Mononobe-Okabe method. Another comprehensive evaluation was published in the paper by Cai and Bathurst (1996) in which the theoretical critical acceleration coefficient k_c was derived from the factor of safety against both external sliding failure and internal sliding failure equations. Berg et al. (2009) recommended a maximum acceleration k_{max} for dynamic earth pressure calculation in Mononobe-Okabe expression and k_{max} can be determined taking peak ground acceleration by a site-specific factor F_{PGA}.

In New Zealand, how to better utilize the Mononobe-Okabe equation by appropriately adjusting its inputs incorporating specific geological conditions has attracted major attention from academic researchers and practicing engineers. Overseas design guidelines in combination with New Zealand experiences and research are well summarized in the Guidelines for Design & Construction of Geosynthetic-reinforced Soil Structures in New Zealand (Murashev, 2003). In this document, the horizontal seismic coefficient is taken as 60% of Peak Ground Acceleration (PGA) for calculation of dynamic soil pressure by Mononobe-Okabe method. Chin and Kayser (2013) investigated dynamic active earth pressure that is produced in a well-established 3m high cantilever wall and drew a conclusion that 65% of the free-free PGS up to 0.3g yields a reasonable match against Mononobe-Okabe derived forces.

In this paper, one typical Mechanically Stabilized Earth Retaining Structure (MSE Wall) is examined for the purpose of looking into the seismic coefficient which is a critical parameter in the Mononobe-Okabe equation to derive dynamic active earth pressure for external stability check. A representative sub-surface profile encountered in Auckland area, typical soil material properties and design criteria are considered in the analysis with an attempt to provide generalized findings and recommendations. Dynamic finite element modeling by using an interactive process of Sigma/W and Quake/W in GeoStudio 2020, is conducted to compare against theoretical seismic earth pressures given by the Mononobe-Okabe equation. The findings obtained in this study provides a useful reference to designers with regards to how to select a reasonably realistic seismic coefficient and also an insight into the pattern of dynamic pressure distribution in relation to a similar MSE wall in Auckland region. It shall be emphasized that this study is limited to insights into external stability of the wall and, thus, only seismic responses on the vertical face of back of the reinforced mass are discussed in the context.

2 ENGINEERING CASE

In New Zealand design practice, the Bridge Manual published by New Zealand Transport Agency (2018) is a prevailing guideline for the design of significant geotechnical works for projects not limited to transport infrastructural projects, for instance, embankment slopes, retaining structures and pile foundations etc. Among these engineering works, Mechanically Stabilized Earth Retaining Structures (MSE Wall) are almost always required to accommodate a grade separation for its incomparable advantages over other types of retaining structures including gravity wall, bored pile wall or others.

In response to such an extensive application of MSE walls in New Zealand, there is a necessity to pay continuous attention and interests in the study on its design and performance. In this study, a trial has been made with an intention of creating general knowledge on a MSE wall of typical scale with regards to selection of a proper design ground acceleration in the hypothesized pseudo-static analysis of MSE walls.

The model of interest and to be studied in this context is shown in the Figure 1. Assumed dimensions of the retaining wall and design criteria are presented in Table 1 and the wall in combination with the sub-surface conditions and its geometry is depicted in Figure 1. Note in this context no surcharge on the retaining wall is included.

Table 1. Parameters of Hypothesized MSE Wall

Design Condition	Assumed Parameters	Notes
Subsoil Class	C	As per NZS 1170.5 (2004)
Design Period	100 years	
Importance Level	3	
Design Free-field PGA (a_m)	0.28g	Ultimate Limit State
Annual Probability of Exceedance	1/2500	
Height of MSE Wall (*H*)	6.0m	
Width of Wall	4.5m	
Length of Extensible Geo-girds (*L*)	4.5m	0.6m interval vertically
Facing Structure	Keystone block	0.2m high facing unit

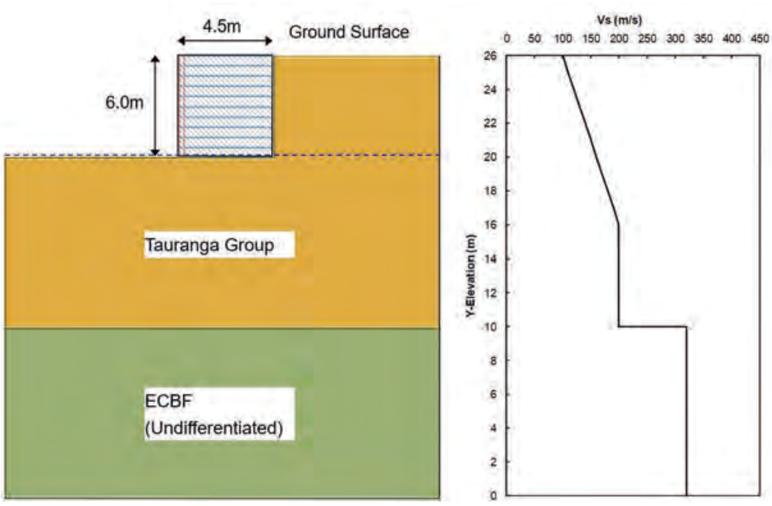


FIGURE 1. Typical MSE wall and assumed shear velocity with depth

TABLE 2. Material Parameters used in the Assessments

Parameters	MSE Fill*	Shallow Soil (TG)	Deep Soil (ECBF)
Unit weight γ (kN/m^3)	21	18	20
Effective friction angle ϕ' (°)	36	30	33
Effective cohesion c' (kPa)	0	5	5
Undrained Shear Stress S_u (kPa)	-	50	100
Poisson's ratio ν	0.25	0.33	0.25
At-rest earth pressure coefficient K_o	0.333	0.5 (1.0**)	0.455
Assumed shear velocity V_s (m/s)	200	Increasing with depth	320

* Coarse granular material for MSE Fill.
** Earth pressure coefficient K_o=1.0 in the stage of dynamic shaking under undrained case.

3 SUBSURFACE CONDITIONS AND PARAMETERS

Typical soil parameters are taken based on the author's knowledge and local experiences in Auckland area. A 16m-thick shallow cohesive soil layer and 10m-thick underlying hard stratum are defined in the subsurface profile, which are considered representative of Tauranga Group (TG) and ECBF (East Coast Bay Formation) extensively encountered in the Auckland region. The base of model is considered as unweathered (intact) rock having a shear velocity of 800m/s and it can be assumed to be rigid. Groundwater table is taken to be at the level of toe of the wall. Table 2 and Figure 2 give material parameters and variations of several critical parameters (dynamic shear modulus and damping ratio).

Shear modulus is estimated based on the classic correlation with shear velocity,

$$G_{max} = \rho V_s 2 \qquad (1)$$

Where, ρ is the unit density of soil, and V_s is the shear velocity.

Degrading pattern of dynamic shear modulus developed by Ishibashi and Zhang (1993) is taken into account in the study. It has been recognized that TGA falls among CI, CH and CV, exhibiting intermediate plasticity, high plasticity further to very high plasticity with a Plasticity Index PI value changing significantly (20 ≤ PI ≤ 60) (BRANZ, 2002). For the purpose of this investigation and also for simplicity, a figure of PI=35 is selected considering it can be deemed as representative of major part of TGA of CH. PI=35 is also used for estimation of damping ratio function proposed by Kramer (1996).

4 DYNAMIC FINITE ELEMENT ANALYSIS

4.1. Experimental Finite Element Model and Assumptions

To capture the dynamic earth pressure more accurately and its variations, dynamic analysis is made on the generic case study shown in Figure 1. Accordingly, a finite element model for a plan-strain case, is developed in Quake/W and is presented in Figure 2. Since the main interest of this study lies in the dynamic responses of the as-built retaining wall, the procedure in the analysis is simplified to be as follows:

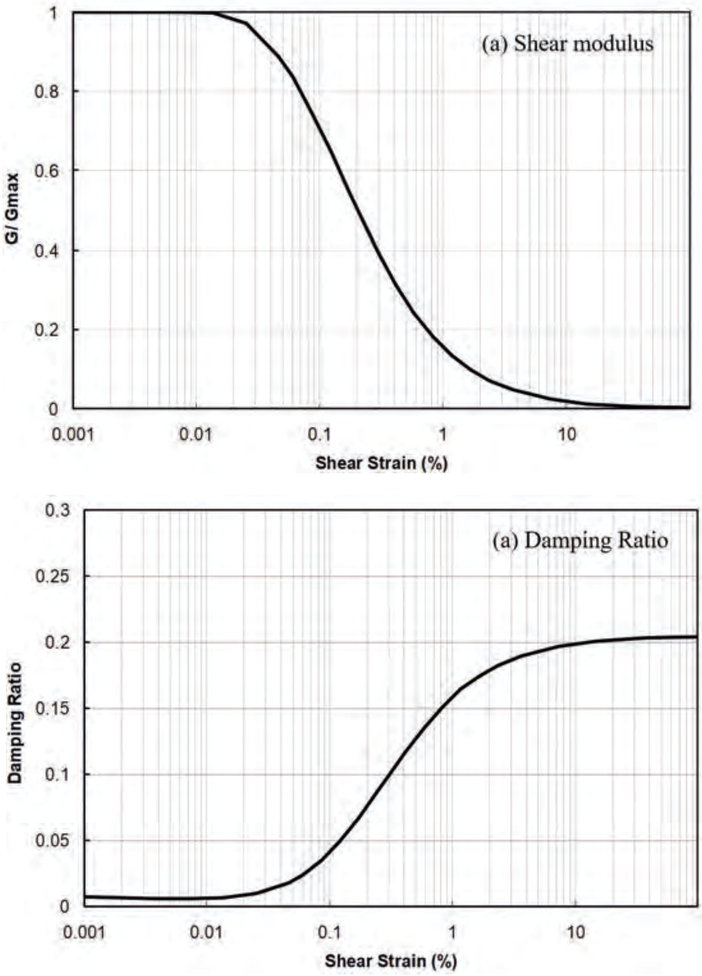


FIGURE 2. Variations of dynamic parameters (shear modulus and damping ratio)

- Stage 1: Initial stress to define the initial stress state in Sigma/W
- Stage 2: Excavate from +26.0m to +20.0m and build the MSE wall in Sigma/W
- Stage 3: Dynamic analysis in Quake/W

In total 1840 quadrilateral elements are created with minimum size of 0.5m in order to fit the model geometry. Vertical boundaries are defined to be 30m away from the face of wall. Both horizontal and vertical moments are restricted throughout the analysis at the bottom of model. On two vertical boundaries, only vertical movements are allowed in Stage 1 while only horizontal movements are permitted in Stage 3. In the Stage 3, cumulative movements from Stage 1 and Stage 2 are excluded.

In GeoStudio Sigma/W, geo-grids can be simulated by structural bar elements and their tensile stiffness are set to be EA=880kN/m. Keystone facing blocks are modeled as a chain for 10 continuous structural beams and they are connected by rigid joints. The bending stiffness of the beams is taken to be EI =1.13×10^4kNm^2 with a cross-section area of 0.3m. Both tension and compression are allowed in the analysis. Interface elements are set up at the interface between MSE gravel fill and its adjacent TG material. The intention of this is to reflect the impact of friction along the interface on the calculated earth pressure that is likely to be mobilized by movements of the wall when subject to a ground motion.

4.2. ACCELERATION TIME HISTORY

15 time history records are selected in the analysis. Basic features of them are summarized in the Table 3. To reduce the data storage, vibrating noise at the end of the recorded acceleration time history, which represents very little ground motion, has been ignored in the dynamic analysis. These seismic records are scaled in DEEPSOIL 7.0 (2016, 2020) for the first step to match the selected specific seismic case of interest with the design free field PGA a_m=0.28g.

4.3. DECONVOLUTION OF ACCELERATION-TIME RECORDS

Deconvolution of chosen recorded surface acceleration records was implemented using DEEPSOIL 7.0 (2016, 2020) by running an equivalent linear analysis on one-

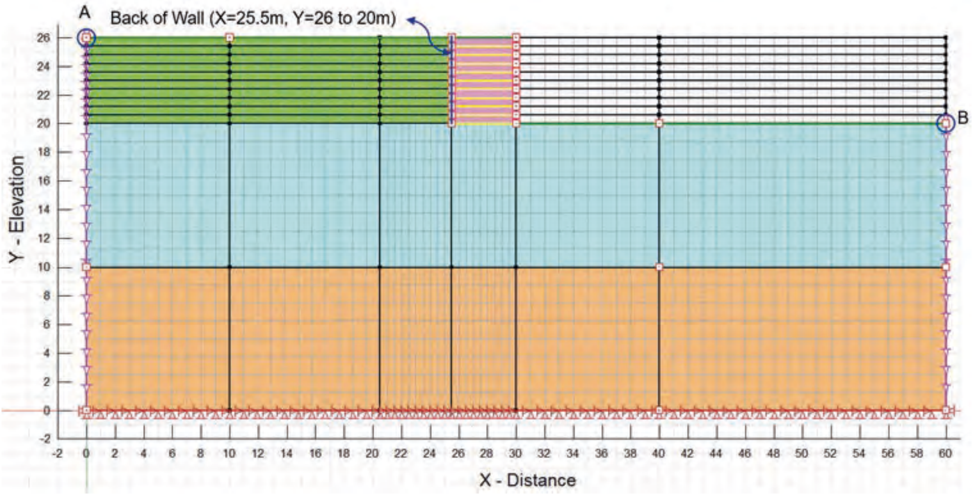


FIGURE 3. Finite element mesh for dynamic analysis

TABLE 3. Comparison of Peak Ground Accelerations

No	Seismic Record Name	PGA of Original Record (g)	Input PGA in Quake/W (g)	Calculated PGA at Surface Point A in Quake/W (g)	Calculated PGA at Surface Point B in Quake/W (g)
1	NewZealand 01	0.318	0.102	0.216	0.242
2	NewZealand 02	0.224	0.093	0.365	0.318
3	MammothLake	0.430	0.123	0.300	0.318
4	Northridge	0.217	0.100	0.345	0.274
5	Northridge 2	0.098	0.095	0.279	0.296
6	LomaGilroy	0.170	0.171	0.405	0.510
7	LomaGilroy 2	0.357	0.112	0.340	0.492
8	Chichi	0.183	0.168	0.296	0.271
9	Coyote	0.124	0.095	0.294	0.315
10	ImperiValley	0.169	0.121	0.294	0.437
11	Kobe	0.821	0.134	0.270	0.266
12	Kocali	0.219	0.063	0.403	0.453
13	Nahanni	0.148	0.122	0.422	0.505
14	Parkfield	0.357	0.083	0.333	0.261
15	WhittierNarrows	0.186	0.097	0.531	0.626

Notes: 1) Points A and B are at the ground surface and shown in Figure 3. 2) Input time-history into Quake/W is the same as output of DEEPSOIL at the rock.

dimensional soil column model. The model geometry and material parameters are kept the same as given in Figure 3 and Table 2. Deconvoluted time history records at the top of rock (the bottom of the numerical model) were directly input into Quake/W models to assess responses of the retaining wall. Calculated PGAs and spectral responses of dynamic motions at ground surface obtained from Quake/W modeling are compared with those derived from DEEPSOIL analysis. Table 3 gives a comparison of PGAs. As it shows, some discrepancies in PGAs and spectral responses are discovered and several seismic records (No. 6, 7, 12, 13 and 15 named in the table) yield much higher PGAs than designed PGA=0.28g. To ensure the feasibility and appropriateness of the Quake/W modeling, their corresponding time histories are eliminated in the analysis and accordingly their results from Quake/W are excluded in the following discussions. Figure 4 illustrate a typical comparative result for horizontal spectral acceleration for No.2 seismic record. Despite there are differences in cyclic peaks they generally appear comparable with each other.

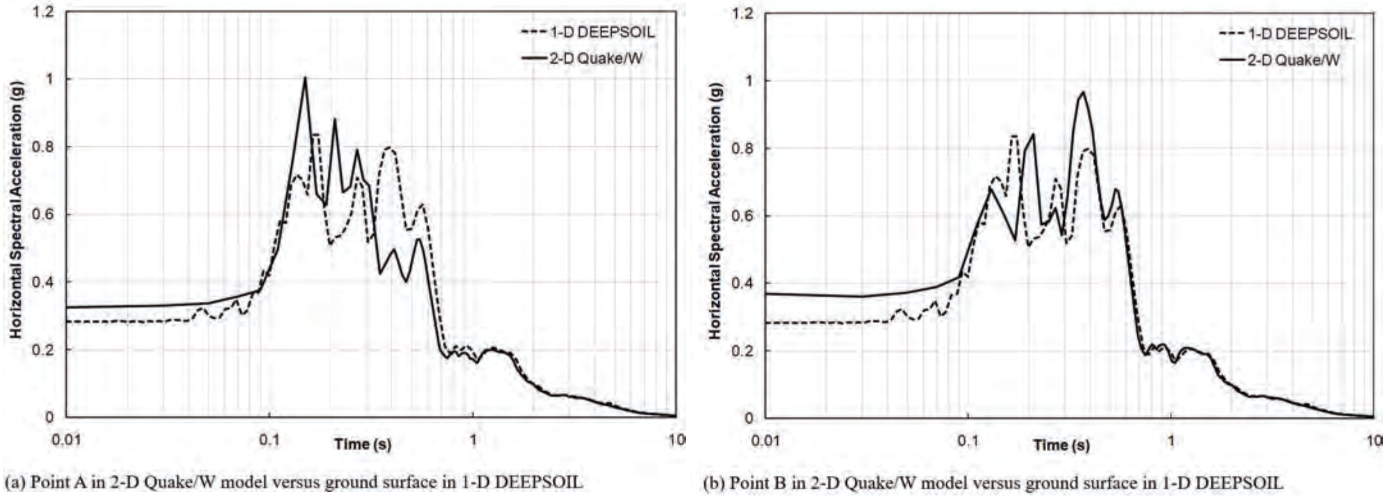


FIGURE 4. A typical comparison of spectral acceleration at the ground surface between DEEPSOIL and Quake/W for seismic record of New Zealand 02

4.4. CALIBRATION OF MODEL UNDER PRE-SHAKING STATIC STATE

The initial stress conditions under both K_o static and the K_A stress state are investigated to calibrate the finite element model. Here, K_o is the static earth pressure coefficient and can be written as $K_o=1-\sin\phi$. K_A is Coulomb's active earth pressure coefficient. Comparisons of calculated horizontal stresses from the model and theoretical estimations are plotted in Figure 5. As can be seen in the figure, there is basically a good agreement between theoretical derivations and Quake/W results under both stress conditions. This, therefore, well demonstrates the reliability and rationality of the model adopted in this study.

5 RESULTS OF ANALYSIS AND DISCUSSIONS

5.1. TRIGGERED SOIL-WALL SEISMIC MOTION

Amplification of horizontal acceleration throughout the height of wall has been previously revealed by a number of researchers (e.g. Segrestin and Bastick, 1988; Cai and Bathurst, 1995). However, the level of amplification relies on a variety of influencing factors, such as the peak ground acceleration, height of wall, stiffness of wall mass and retained soil and etc. Therefore, for this reason, it is recommended to carry out a specific analysis to assess amplification effect for the particular case of concern. Triggered horizontal peak accelerations on the interface of MSE wall mass and retained soil are extracted from the output of analysis, and are plotted in Figure 6. Data in this figure show that variations in the peak acceleration on the selected face of wall are moderately large. On average, it seems that the peak acceleration on the wall top is about 1.5 times that at the wall bottom for most of prescribed time history records. Results also imply that at the half of wall, the peak acceleration approximately equal to designed peak ground acceleration a_m=0.28g.

5.2. INDUCED VERTICAL PEAK ACCELERATION $K_V G$ AT BACK OF MSE MASS

Vertical accelerations can be taken from calculations and, therefore, this makes it possible to review vertical seismic

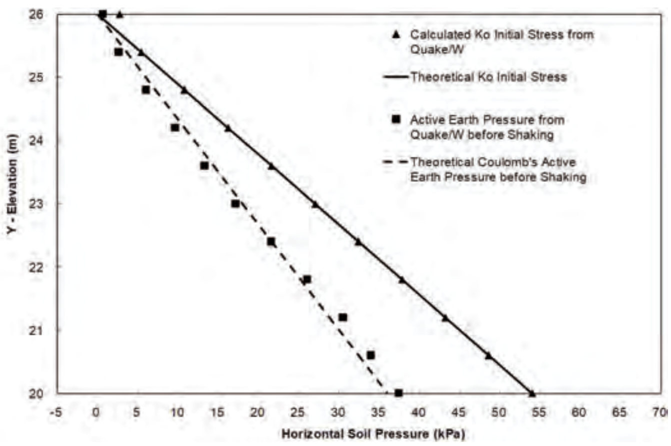


FIGURE 5. Calibration of static state prior to dynamic shaking

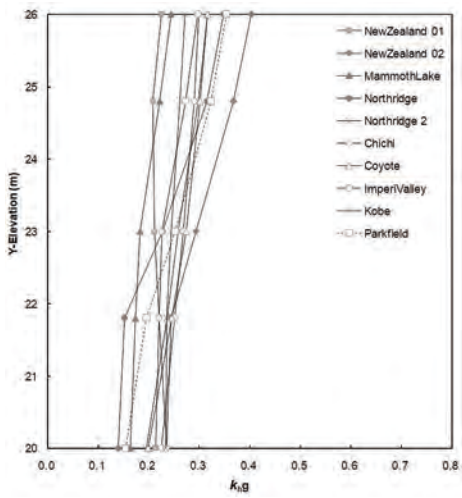


FIGURE 6. Horizontal peak accelerations $k_h g$ triggered at different levels along the height of back of reinforced mass

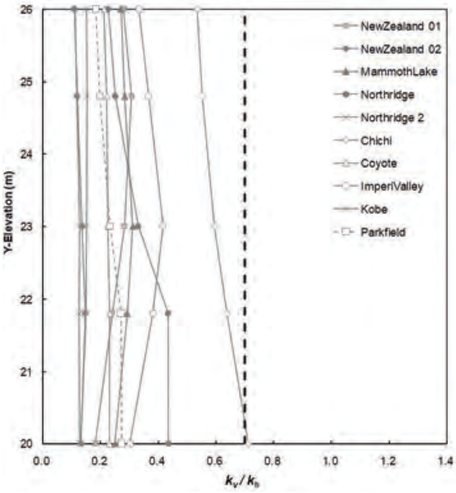


FIGURE 7. Ratios of k_v/k_h along the height of the back of wall mass

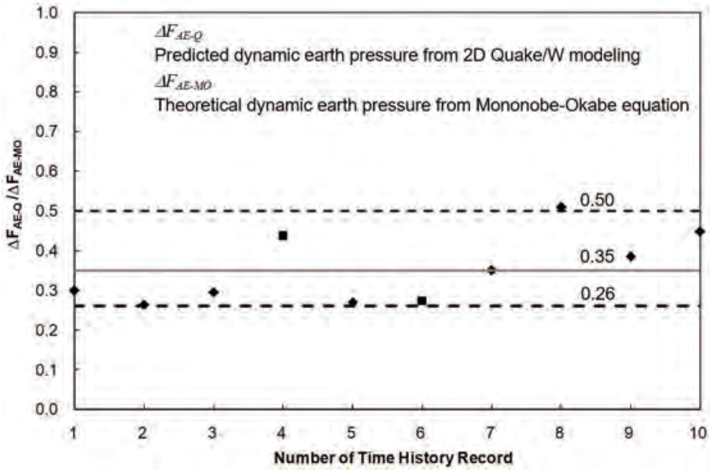


FIGURE 8. Derived ratios of $\Delta F_{AE-Q}/\Delta F_{AE-MO}$ in this study

coefficients produced on the back of wall in every dynamic analysis. The ratios of k_v/k_h along the height of the wall at the vertical plane of X=25.5m are given in Figure 7. It suggests that the magnitude of k_v/k_h changes from 0.1 to 0.7. This observation is well consistent with the estimation of Stewart et al. (1994) who recommended an estimated $k_v=2/3k_h$ based on seismic data in Los Angeles area. No doubt $k_v=(0.67 \text{ to } 0.7)k_h$ is a conservative estimation and it shall be deemed as an extreme case. For the engineering design, it seems that taking $k_v=(0.2 \text{ to } 0.3)k_h$ would be more realistic with a moderate conservatism.

5.3. SEISMIC COEFFICIENT KH

Under ULS case, defined by PGA a_m=0.28g, the Mononobe-Okabe equation can be applied to estimate the seismic active earth pressure coefficient K_{AE} and furthermore to derive the total active pressure to be generated on the back of reinforced MSE wall block in the external stability assessment. This methodology is being adopted in New Zealand as recommended in NZTA BM 3rd Amendment 3 (2018) and NZTA Research Report No. 239 (2003). The equation is written as follows,

$$F_{AE-MO} = \frac{1}{2}(1 \pm k_v)K_{AE} \bullet \gamma H^2 \quad (2)$$

Furthermore, an increment in the active pressure as result of a ground motion is written as:

$$F_{AE-MO} = \frac{1}{2}\left[(1+\ k_v)K_{AE} - K_A\right]\gamma H^2 = \frac{1}{2} \quad K_{AE} \bullet \gamma H^2 \quad (3)$$

Where, K_A refers to the Coulomb's active earth pressure coefficient.

The dynamic analysis performed in this study also gives the dynamic horizontal total active stress acting on the back of wall mass and, therefore, the total active earth pressure can be determined by integrating it over the entire height of the wall. No doubt the maximum pressure (denoted as F_{AE-Q}) over the duration of ground motion is considered to be critical and is of our concern. By using it, active pressure increment ΔF_{AE-Q} can be subsequently derived by subtracting static total force on the active side of the wall computed in the step before shaking. The ratio of $\Delta F_{AE-Q}/\Delta F_{AE-MO}$ is plotted in Figure 8 for all the ten seismic records and it illustrates the ratio changes primarily changes between 0.26 and 0.50 in case that vertical seismic coefficient k_v is zero. In this case, k_h can be calibrated based on this ratio and is calculated to be varying from 0.32 to 0.52 provided k_v=0 is adopted, to match the obtained value of $\Delta F_{AE-Q}/\Delta F_{AE-MO}$. On the other hand, in the event that a vertical acceleration is taken by using 10% to 70% of horizontal acceleration, k_h can be calculated to vary from 0.32 to 0.58. These data indicate that k_v is not sensitive to the derivation of k_h and a generalized range of k_h=0.32 to 0.58 may be appropriate to include two different cases. Cai and Bathurst (1996) proposed a theoretical solution to the critical acceleration coefficient previously given that the factor of safety against base sliding failure is expected to be 1.0. By applying their proposed method and expressions, the critical acceleration coefficient k_h can be approximately calculated as 0.35. This implies for the specific case discussed in the context, Cai and Bathurst (1996)'s recommended value is likely to be the lower bound of horizontal coefficient. The author notes NZTA Research Report No. 239 (2003) recommends $k_h = 0.6a_m$ for external stability analysis. Findings obtained in this study demonstrates this recommendation is reasonable and acceptable although it is slightly conservative.

5.4. DISTRIBUTION OF THE DYNAMIC EARTH PRESSURE

The horizontal total stress mobilized on the back of MSE wall mass obtained from the analysis is presented in Figure 9. It is obvious that its distribution remains close to a triangular shape with a resultant thrust force acting on the level of 1/3*H*. Traditionally accepted dynamic earth pressure profile proposed by Bathurst and Cai (1995)

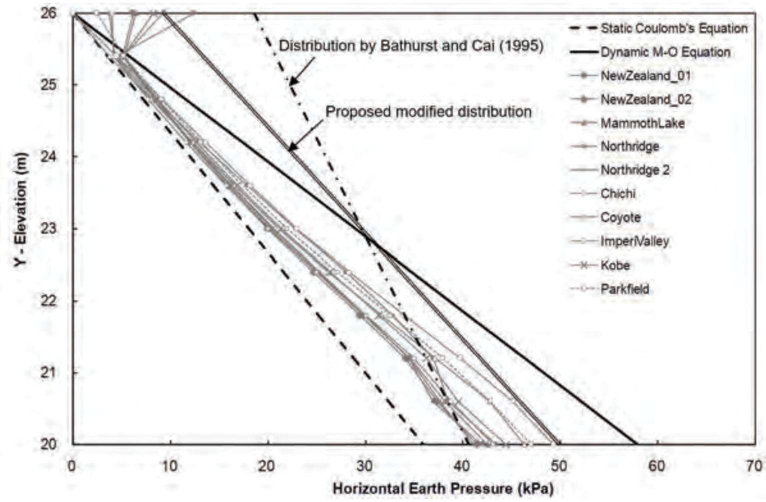
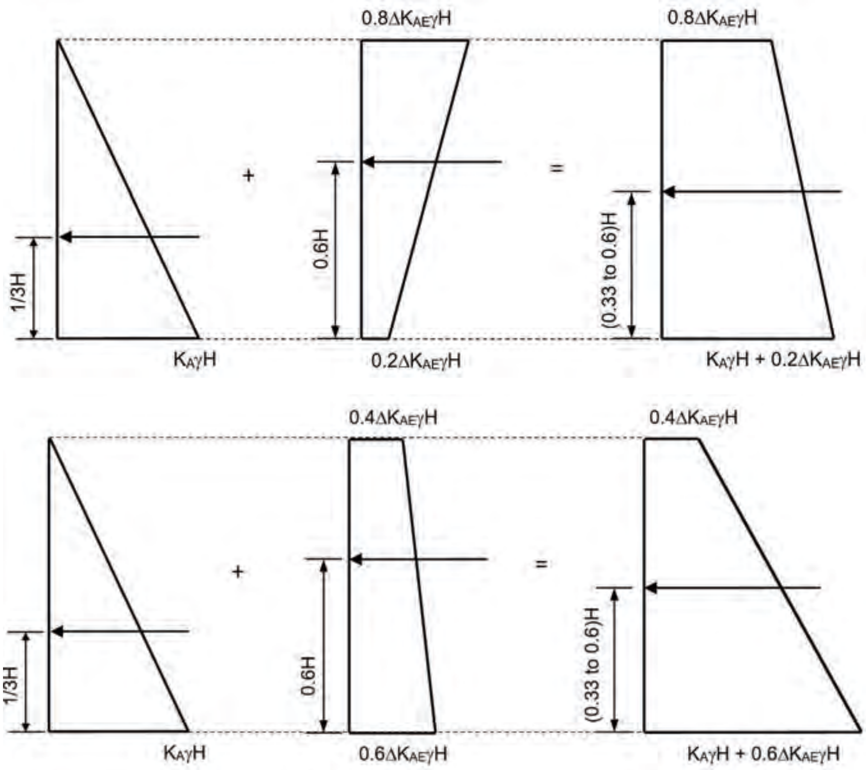


FIGURE 9. Variations of earth pressures corresponding to maximum total dynamic active earth pressure and their distributions

suggests the pressure at the top of wall can be assumed as $0.8\Delta K_{AE}\gamma H$ and $(K_A+0.2\Delta K_{AE})\ \gamma H$ at the toe of the wall and this forms an approximate inverted triangular. For a comparison, this distribution is also plotted in Figure 6. As we can see from the figure, Bathurst and Cai (1995)'s hypothesized distribution overestimates the dynamic earth pressure increment approximately within the higher half of the wall but underestimates the dynamic earth pressure increment within the lower half of the wall.

Results of dynamic analysis performed in the present study enables us to examine the pressure distribution and compare to the assumed one by Bathurst and Cai (1995). The line named as the modified dynamic earth pressure profile in Figure 9, is produced based on the present results and it appears as the envelop of induced dynamic earth pressure. The distribution of the dynamic earth pressure increment proposed by Bathurst and Cai (1995) exhibits a decreasing pattern from the top of the wall to the bottom of the wall. This hypothesized profile was developed on purpose (to some degree) to accommodate the probable amplification effect of horizontal acceleration. Nevertheless, this study gives an entirely different distribution mode, which indicates an increasing profile rather than decreasing. This observation implies that the current model does not present a noticeable amplification overt the full height of the wall. The key reasons for this may be attributed to: 1) comparable dynamic rigidity of MSE fill gravel



(a) Traditional dynamic earth pressure with a decreasing dynamic component by Bathurst and Cai (1995)

(b) Proposed modified dynamic earth pressure distribution with an increasing dynamic component

FIGURE 10. Vertical dynamic earth pressure distribution under seismic loading for external stability

and its retained TG soil, and 2) relatively low ground accelerations. By doing simple calculations, the ratio of dynamic shear modulus between MSE fill and TG soil is found to fall within a range of 4.7 to 2.1 (from the top to the bottom of the wall). These comparable stiffnesses results in compatible soil-wall interactions and subsequent movements, which is supposed to largely suppress the mobilization of amplification effect. This type of decreasing distribution of dynamic soil pressure was also reported by Bathurst and Hatamai (1988). Figure 10 shows a comparison between two pressure distributions in more details. The proposed incremental dynamic earth pressure has a lower pressure of $0.4\Delta K_{AE}\gamma H$ on the level of top of wall and $0.6\Delta K_{AE}\gamma H$ at the level of bottom of the wall. Its resultant force acts at approximately the middle of the wall. Combing with static earth pressure, the resultant total earth pressure forms a different trapezoid and the total active force acts at the level of one third to a half of the wall.

5.5. MAXIMUM LATERAL DISPLACEMENT

The predicted maximum outward lateral displacement of the wall takes place at the top of wall face (at Point (30, 26) in the mesh) and it falls in the range of (0.7% to 2.5%)*H*. Movements of this order are expected to be a sign that the wall has deformed enough so that Mononobe-Okabe theory and the method remain applicable.

6 CONCLUSIONS AND RECOMMENDATIONS

This study has been conducted for the purpose of analyzing the seismic coefficient and dynamic earth pressure distribution for a geo-grid reinforced MSE retaining wall sitting on a representative subsoil Class C in Auckland area under representative seismic ground motions. Major findings and recommendations are summarized as follows:

a) The seismic amplification of horizontal acceleration along the height of wall back is observed at the top of the wall. The vertical acceleration on the back of the wall mass is also driven on the back of the wall mass and in general it has a maximum value of 0.7 horizontal acceleration.
b) The dynamic active earth pressure well matches with predicted values from the conventional Mononobe-Okabe approach when the horizontal seismic coefficient is set to be 0.32 to 0.52 if the vertical seismic coefficient is disregarded. While a vertical acceleration is included by a ratio k_v/k_h ranges from 0.1 to 0.7, the horizontal seismic coefficient is calculated between 0.32 and 0.58. This implies that the recommended $k_h = 0.6a_m$ for external stability analysis by NZTA Research Report No. 239 (2003) is considered conservative and close to the upper bound.
c) The traditional distribution of dynamic earth pressure proposed by Bathurst and Cai (1995) is found conservative in particular within two third of the wall. A modified distribution mode (as shown in Figure 10) is proposed, which appears better fit the calculated results from the analysis. Compared to the traditional distribution, the proposed one displays an increasing dynamic pressure from the top to the bottom of the wall rather than decreasing.

It is important to note that the findings and recommendations presented in this study may be only for those cases in association with similar ground conditions, comparable geometry and design criteria etc. A specific dynamic finite element analysis is always a comprehensive and reliable approach compared to pseudo-static method to produce best solutions for a MSE wall.

REFERENCES

Bathurst, R.J. and Hatamai, K. (1988). Seismic Response Analysis of a Geosythetic-reinforced Soil Retaining Walls. Geosythetics International, 5(1-2), 127-166.

Bathurst, R.J. and Cai, Z. (1995). Pseudo-static Seismic Analysis of Geosythetic-reinforced Segmental Retaining Walls. Geosythetics International, 2(5), 787-830.

Berg, R.R., Christopher, B.R. and Samtani, N.C., (2009). Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes. FHWA NHI-10-024, FHWA GEC 011 - Volume 1.

BRANZ. (2002). Study Report (No.120 (2003)) Soil Expansivity in the Auckland Region. Fraser Thomas Ltd.

Cai, Z. and Bathurst, R.J. (1996). Seismic-induced Permanent Displacement of Geosythetic-reinforced Segmental Retaining Walls. Canadian Geotechnical Journal, Vol.33, 937-955.

Chin, C.Y. and Kayser, C. (2013). Use of Mononobe-Okabe Equations in Seismic Design of Retaining Walls in Shallow Soils. Proceedings of 19th NZGS Geotechnical Symposium. Edited by Chin, C.Y.. Queenstown.

Hashash, Y.M.A., Musgrove, M.I., Harmon, J.A., Groholski, D.R., Phillips, C.A., and Park, D. (2016) "DEEPSOIL 6.1, User Manual". Urbana, IL, Board of Trustees of University of Illinois at Urbana-Champaign.

Hashash, Y.M.A., Musgrove, M.I., Harmon, J.A., Ilhan, O., Xing, G., Groholski, D.R., Phillips, C.A., and Park, D. (2020) "DEEPSOIL 7.0, User Manual". Urbana, IL, Board of Trustees of University of Illinois at Urbana-Champaign.

Ishibashi, I., and Zhang, X. (1993). Unified Dynamic Shear Moduli and Damping Ratios of Sand and Clay, Soils and Foundations, Vol.33, No.1, pp. 182-191.

Kramer, S.L., 1996. Geotechnical Earthquake Engineering, Prentice Hall.

Murashev, A.K. (2003). Guidelines for Design & Construction of Geosynthetic-Reinforced Soil Structures in New Zealand. Transfund New Zealand (2003). NZTA Research Report No. 239.

Mononobe, N. and Matsuo, M. (1929). On the determination of earth pressures during

earthquakes. Proc. World Eng. Congress, 9, pp. 179 – 187.

New Zealand Transport Agency, 2018. Bridge Manual SP/M/022, Third Edition, Amendment 3.

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

Segrestin, P., and Bastick, M. (1988). Seismic Design of Reinforced Earth Retaining Walls - the Contribution of Finite Element Analysis. Proceedings of the International Geotechnical Symposium on Theory and Practice of Earth Reinforcement, Fukuoka, Japan, 5-7 October, 1988, 577-582.

Stewart, J.P., Bray, J.D., Seed, R.B., and Sitar, N. (1994). Preliminary Report on the Principle Geotechnical Aspects of the January 17, 1994 Northridge Earthquake. University of California at Berkeley, Earthquake Engineering Research Center, Report UCB/EERC-94/08.

Piling | Ground Stabilisation | Retaining walls | Ground Anchors | Seismic Strengthening | Subdivisions | Rope Access | Landscaping | Foundations | Civil Structures | Drainage | Slip Stabilisation | Rammed Aggregate Piers

The Sumner Road Reopening Project

Rockfall mitigation and road repair of an important route between Lyttelton and Christchurch

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This paper was prepared for the NZGS 2020 Symposium and will be presented at the rescheduled meeting

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Charlie Watts

Peter Bawden

ABSTRACT

Sumner Road between Lyttelton to Evans Pass was severely affected by rockfall as a result of the 2010-2011 Canterbury Earthquake Sequence. Approximately 2 km of Sumner Road was closed to all traffic following the 22 February 2011 earthquake. Sumner Road was used as a corridor for dangerous goods vehicles between Lyttelton and Christchurch and the absence of this route periodically required temporary closing of the Lyttelton Road Tunnel to the public to allow dangerous goods vehicles to pass. In 2015 Christchurch City Council (CCC) and New Zealand Transport Agency (NZTA) initiated a programme of works to mitigate the geohazards affecting this road corridor. A team comprising Golder and Jacobs staff provided concept designs, tender evaluation and supervisory technical expertise to the client during construction. The physical mitigation of rockfall hazard, repair of earthquake damaged retaining walls and the road pavement took 2.5 years and the road was officially opened to the public on 29 March 2019, more than 8 years following the earthquake that closed it. Significant cost savings were possible due in part to our re-evaluation of the primary rockfall source slope as not having experienced significant rock mass damage, therefore avoiding costly large-scale benching and associated earthworks.

1 INTRODUCTION

The project area was affected by significant rockfall on multiple occasions during the 2010-2011 Canterbury Earthquake Sequence (CES), both as a direct result of specific earthquakes and subsequently. Sumner Road was also affected by rockfall prior to 2010, with anecdotal records of rockfall occurring during most winters. Prior to 2010, rockfalls were typically of small volume, with block sizes less than about 0.5 m in maximum dimension. Debris flows also affect the road corridor, including several that occurred during a heavy rainfall event in 2014. Concept level risk assessments completed by several consultants including Golder concluded that post-earthquake rockfall presented an unacceptably high risk to road users and that mitigation measures were required to reduce the risk to an acceptable level.

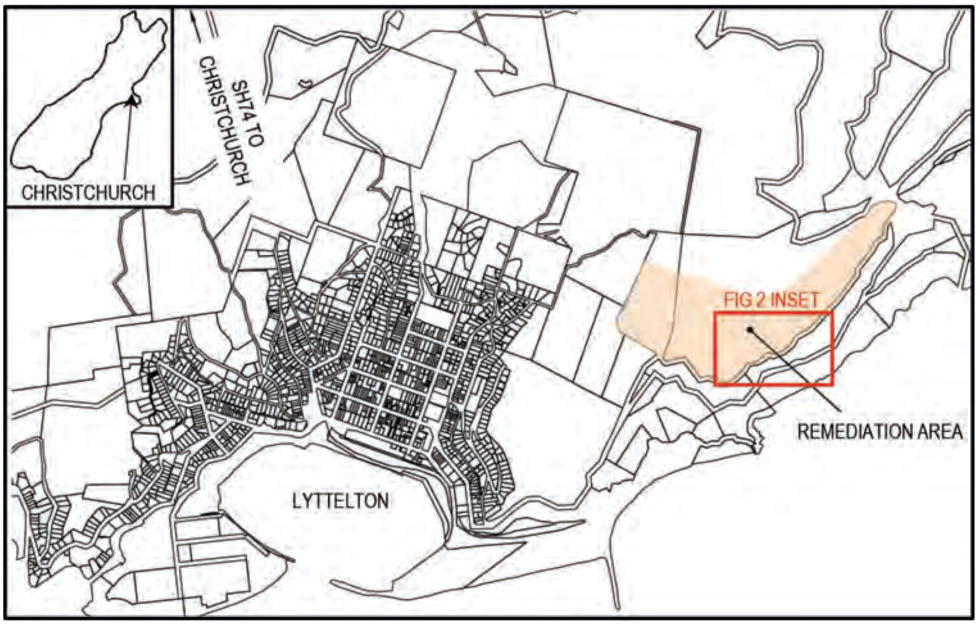

FIGURE 1: Location plan of Sumner Road Remediation Project site

FIGURE 2: Aerial photograph of the eastern area of Sumner Road Remediation Project site prior to construction where Sumner Road passes below 100 m high steep rock cliffs. Yellow and blue zones denote recent, likely earthquake-related rockfall. Red zones are debris flows associated with the 2014 heavy rainfall event.

The road formation and retaining walls were also damaged as a result of earthquake shaking and debris flow, the latter causing disruption of the surface stormwater system. While design of these elements was not part of the original rockfall mitigation work scope, it is relevant to note that rockfall caused relatively minor damage to the road formation (i.e., impact craters and damaged concrete parapet walls) compared to the damage from earthquake shaking.

Concept design for rockfall mitigation at the site (Aurecon, 2013) included reinforced earth bunds at two gullies at the western end of the project area, extensive slope modification and scaling (approximately 1 Mm^2) to remove the potential source rock.

2 SLOPE INSTABILITY HAZARD

Sumner Road traverses steep, south facing slopes adjacent to Lyttelton Harbour (see Figure 1 and 2). The slopes include prominent cliffs up to 100 m high that slope at about 70° or more. The area is underlain by Miocene age basaltic to trachytic volcanic rocks of the Lyttelton Volcanic Group (Forsyth et al., 2008). These lava, tuff and breccia beds dip at about 10° to the north, into the slope. The weaker breccia and tuff beds typically slope at about 45°.

The southern side of Lyttelton Harbour exposes basaltic lava flows of the Late Miocene Diamond Harbour Volcanic Group. This formation suggests that the general configuration of slopes around Lyttelton Harbour is at least Late Miocene in age.

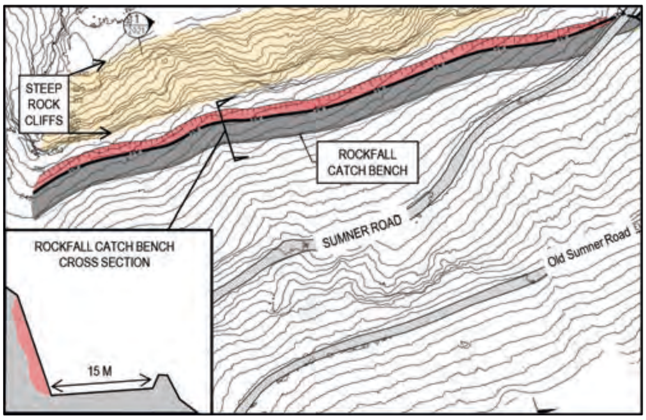

FIGURE 3: Conceptual design of the 15 m wide, 400 m long cut rockfall catch bench

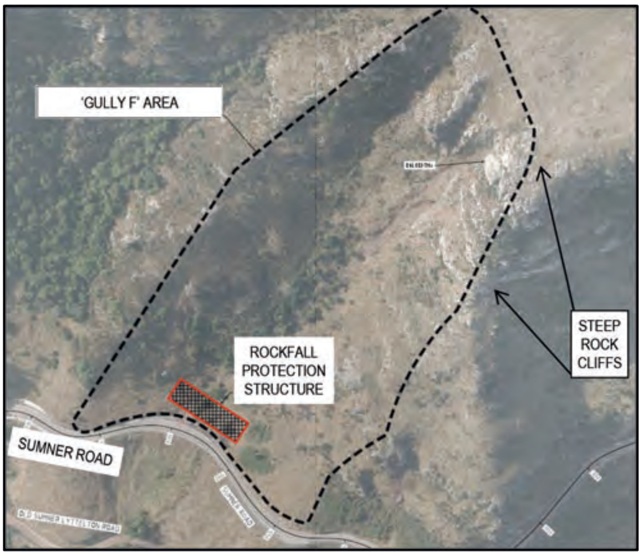

FIGURE 4: Location of rockfall protection structure above Sumner Road at 'Gully F'

A range of slope instability hazards affect Sumner Road including rockfall (at various scales) and debris flows. Minor rockfall has reportedly affected the road on an almost annual basis prior to the CES with failures mainly coming from the road cuttings. Rockfall was widespread along the project site as a result of the earthquakes. Based on aerial photographs and field mapping it is estimated that about 9000 m^2 of rockfall occurred as a direct result of the earthquakes. The dominant source areas for rockfall from the earthquakes were the natural cliffs and to a lesser extent the road cuttings, with about 90% of rockfall being sourced from these areas.

Based on observations of the scars from rockfalls on the slope, the failure mechanisms for individual source areas have mainly comprised toppling, which is consistent with the layered volcanic formation dipping gently into the slope. However, evidence has been recognised for discontinuity-controlled wedge failures, planar failures and isolated blocks being thrown directly into the air. The largest observed slope failures resulting from the earthquakes have extended about 5 m into the slope, but generally the failures are limited to 2 m into the slope behind the pre-earthquake face.

Debris flows followed the 2014 heavy rainfall event affecting several locations within the project area (see Figure 2). However, the debris comprised mainly fine-grained silty material, with relatively minor boulder fraction. Consequently, vehicles impacting debris flow material is relatively minor compared to boulder dominated rockfall debris. As such, the risk to road users associated with debris flows is relatively minor compared to other geohazards.

Concern was previously raised about the potential risk of collapse of the cliffs above Sumner Road during a future earthquake. Previous work on the site suggested that the rock mass strength had been significantly degraded as a result of the CES (e.g. Massey et al., 2012). Our observations during field mapping did not support this theory; we observed no significant tension cracks along the crest of the bluffs and relatively minor dilation of the rock mass in the slope during abseil inspections. We concluded that, despite the bluffs being steep rock slopes, rock discontinuities in the slope are generally favourably oriented. More importantly, the bluffs have not been actively eroded by wave action during the late Quaternary. This contrasts with the well documented location of cliff collapse around Sumner, where coastal erosion had occurred during the Holocene.

3 DESIGN APPROACH

RISK ASSESSMENT

CCC's approach to the mitigation of rockfall risk along Sumner-Lyttelton Road Corridor was to meet a risk based criteria of better than ARL2 (Assessed Risk Level – using the semiquantitative risk assessment approach of RMS, 2011) with respect to road user safety and to ensure that the road could be reopened (at ARL2 or better) within three days of a future earthquake similar to the CES events.

Golder and Jacobs undertook a semi-quantitative risk assessment for rockfall risk affecting Sumner Road users following the RMS (2011) method. We divided the corridor into 100 m long segments and calculated an ARL for each. This assessment indicated that about 90% of the road corridor was estimated to be ARL1, the highest risk level assigned by the RMS method.

PRELIMINARY DESIGN

Golder and Jacobs developed a preliminary design for rockfall mitigation incorporating:

- Scaling of the main source areas above Sumner Road using abseil crews,
- Construction of an approximately 400 m long rockfall catch bench below the bluffs which was recognised as the main source of rockfall material (see Figure 3).
- Construction of a rockfall protection structure in 'Gully F' above the Lyttelton Port Coal storage facility where rockfall boulders from steep cliffs concentrated in the natural gully (see Figure 4).
- Scaling and trimming of the road batters adjacent to Sumner Road.

FIGURE 5: Photo of completed rockfall catch bench above Sumner Road. Stormwater drains denoted by grey narrow gravel-filled trenches on catch bench.

FIGURE 6: Green Terramesh rockfall bund above Sumner Road at Gully F

FIGURE 7: Windy Point, which is located above Lyttelton Port in the southwestern end of the project. Excavation of the slope was undertaken to enable realignment of the road away from the failed supporting edge (red dashed line) to the present location (yellow lines).

Significantly, previous advice to the CCC was that the slopes providing the main rockfall source area above Sumner Road required quarry-style benching due to earthquake-related damage to the rock mass. The Golder/Jacobs preliminary design described above did not adopt this approach as the rock was assessed to be reasonably intact.

CCC let the rockfall mitigation works tender on a design build NZS3916 contract basis using the Golder/Jacobs preliminary design as a specimen design for tenderers. McConnell Dowell (MacDow) was selected as the successful tenderer for the rockfall mitigation works, with Beca providing design services. Beca and MacDow developed detailed designs for the works that largely followed the preliminary design.

4 ROCKFALL MITIGATION WORKS

Scaling was completed by multiple (up to six three-person crews at a time) rope access teams working from the crest of the escarpment. The scaling work took approximately six months to complete and required only minor high-velocity blasting effort to remove selected large rock pinnacles that were too large to remove by hand tools, airbags or low-velocity blasting. Physical support was available to support unstable areas of slope, but this was not required as rock removal by scaling was considered as being adequate to achieve the necessary safety threshold.

The 400 m long catch bench was excavated using conventional excavators and 6-wheeled ADTs with in-ground blasting needed to improve excavatability. The bench is approximately 15 m wide and has a minimum 2 m high rock wall lip left in place along the outside edge to improve retention of rockfall (see Figure 5). The floor of the catch bench comprises ripped in place rock debris to provide a substrate to mitigate bouncing and rolling of rockfall. The bench falls to the east towards Sumner Road at a gradient of 1V:8H. Stormwater is diverted off the bench at seven intervals to prevent the full catchment of the bench from discharging directly onto Sumner Road. In line with the Principal's requirements, rockfall modelling estimated that the bench would arrest 95% of rockfall from the escarpment and also has ample capacity to accommodate a large-scale failure, should that occur.

At Gully F MacDow and Beca opted for a 55 m long and 7 m high reinforced-earth 'Green Terramesh' rockfall bund as the rockfall protection structure where significant rockfall was concentrated (see Figure 6). The maximum design boulder size was in the order of 2 m in the maximum dimension.

Finally, the road cuttings were trimmed and scaled to remove obvious rockfall source material and to improve the line of site at some locations. Many of the road cuttings are almost vertical, in excess of 10 m high and were cut during the 1930s when Sumner Road was originally constructed. These road cuttings have typically performed well and only minor trimming and scaling was undertaken as part of the project. Significant reprofiling of the cut slope at Windy Point was completed due to slope failure below the outside (supporting) road edge and to optimise the road alignment (see Figure 7).

FIGURE 8: A retaining wall in the process of being repaired. Sumner Road is above the retaining wall.

Following completion of the rockfall mitigation works, we completed a 'post-works' risk assessment. The assessment concluded that all of the road corridor now meets ARL2 or better and that most of the corridor is better than ARL3.

5 POST ROCKFALL REMEDIATION ROAD LEVEL REPAIRS (ROAD FORMATION, RETAINING WALLS, STORMWATER AND PAVEMENT)

Following completion of rockfall mitigation works, safe access was available to Sumner Road for repair of retaining walls and road pavement. Golder and Jacobs prepared an inventory of retaining walls and developed concept designs and cost estimations for repair. Many crib retaining walls were found to be damaged, while concrete retaining walls performed well. Informal guard rails and parapet walls were extensively damaged from rockfall. In all, work was undertaken on 30 retaining walls along Sumner Road some of which were more than 50 m long and 8 m high. An example of a retaining wall under repair is shown in Figure 8.

Concrete crib retaining walls were typically repaired by anchoring back the structure into rock and placing multiple coatings of structural shotcrete to reinforce the crib elements (see Figure 9). Some gabion basket retaining walls were removed and entirely replaced and some new retaining walls were required where no retaining wall was previously present. New ground beams were constructed to support crash barriers.

Stormwater control was typically designed to replicate the pre-earthquake condition and this required replacement of many culverts, sumps, flumes and gutters.

Where appropriate, the pre-earthquake pavement was retained, however, most of the pavement was damaged

FIGURE 9: Rock anchoring in progress as part of the repair of an existing, damaged wall.

by rockfall and retaining wall deformation requiring widespread replacement with stone mastic asphalt (SMA) overlying bitumen treated basecourse (BTB). In areas of over excavation cement treated basecourse (CTB) was also utilised. During pavement works historically placed coal tar was excavated and disposed of at an appropriate offsite facility.

6 CONCLUSIONS

Implementation of rockfall mitigation works including scaling of rockfall source areas and a rockfall catch bench have been assessed to meet CCC's risk-based criteria for road users and resilience against road closure following a large earthquake. The mitigation includes an innovative catch bench beneath the main source of rockfall during the earthquakes. The construction cost for the project was about $20M below the pre-construction cost estimate of $60M, due in part to the re-evaluation by Golder/Jacobs of previous assumptions that the largest rockfall source required large-scale benching.

7 REFERENCES

Aurecon. 2013. Sumner Road Stage 3 A&B Concept Design Report. Report for Christchurch City Council.

Forsyth, P.J.; Barrell, D.J.A.; Jongens, R. (compilers) 2008. Geology of the Christchurch area. Institute of Geological and Nuclear Sciences 1:250 000 geological map 16. 1 sheet + 67 p. Lower Hutt, New Zealand. GNS Science.

Massey, C.I.; McSaveney, M.J.; Yetton, M.D.; Heron, D.; Lukovic, B and Bruce, Z.R.V. 2012. Canterbury Earthquakes 2010/11 Port Hills Slope Stability: Pilot study for assessing life-safety risk from cliff collapse, GNS Science Consultancy Report 2012/57.

Roads and Maritime Services (2011) Guide to Slope Risk Analysis Version 4 (Draft)

Palmerston North 'Priority Route'

Preliminary Geotechnical Assessment and Seismic Site Classification Discussion

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ABSTRACT:
Palmerston North City Council (PNCC) have identified a number of earthquake-prone buildings requiring seismic strengthening located along the 'Priority Route' in the City Centre. Seismic assessment (i.e. estimation of %NBS) and the subsequent design of strengthening schemes is heavily influenced by the Seismic Site Classification (most commonly NZS1170.5:2004 Site Sub-Soil Class). The subject area has very little available ground investigation data and structural seismic assessments are often based on crude assumptions of the ground conditions.

This paper presents the findings of the initial stage of a project currently being undertaken collaboratively between PNCC and Miyamoto International NZ Ltd. (Miyamoto) with the main objective to provide PNCC, building owners and practitioners with additional information, at a preliminary level, to assist with seismic assessment and / or seismic strengthening, repair or maintenance works to building along the Priority Route.

1. INTRODUCTION

Palmerston North City Council (PNCC) have identified a number of earthquake-prone buildings along the 'Priority Route' in the City Centre (Figure 1). PNCC encourage the active protection and management of the natural and cultural heritage of Palmerston North City, a significant portion of which is directly related to Heritage buildings. PNCC offers targeted financial assistance to property owners and other relevant parties to address the conditions imposed on the owners of private properties. Miyamoto International NZ Ltd (Miyamoto) was commissioned by PNCC initially to undertake high-level structural assessments for the buildings and subsequently to undertake a geotechnical assessment for the area encompassing the priority route.

As part of the commission we have undertaken a Multi-channel Analysis of Surface Waves (MASW) geophysical survey of the entire Priority Route and collated available information from the surrounding area including information held in the council archives.

This paper summarises the findings of our investigation and presents a discussion on the Seismic

FIGURE 1: Site and MASW Survey Location Plan

Site Classification for the area, alongside a number of useful parameters suitable for preliminary assessment and design.

The MASW data will be uploaded to the New Zealand Geotechnical Database.

2. GEOLOGY

Palmerston North is located to the eastern extent of the Whanganui sedimentary basin, as shown in Figure 2.

The local GNS Geological Map (Lee & Begg, 2002) shows the surface geology of the Priority Route to be Late Pleistocene river deposits, classified as "poorly to moderately sorted gravel with minor sand or silt underlying terraces (unit Q2a)". The surface geology just south of the Priority Route is mapped as Holocene river deposits, classified as "alluvial gravel, sand, silt, mud and clay with local peat; includes modern river beds (unit Q1a)". An extract of the geological map is shown in Figure 3.

Within the local area, the depth to the 'actual' greywacke bedrock may be in the order of 1000 m or greater distance from ground level, however, 'effective' or engineering bedrock with shear wave velocity (V_s) values greater than 700 or 800 $^m/_s$ can be considered at significantly shallower depths (as discussed later in this paper).

3. GROUND INVESTIGATION

As part of the initial phase of the investigation and to guide the following phases, available existing ground investigation information was collated, including a rigorous search of the Council archives (refer to Section 4.2).

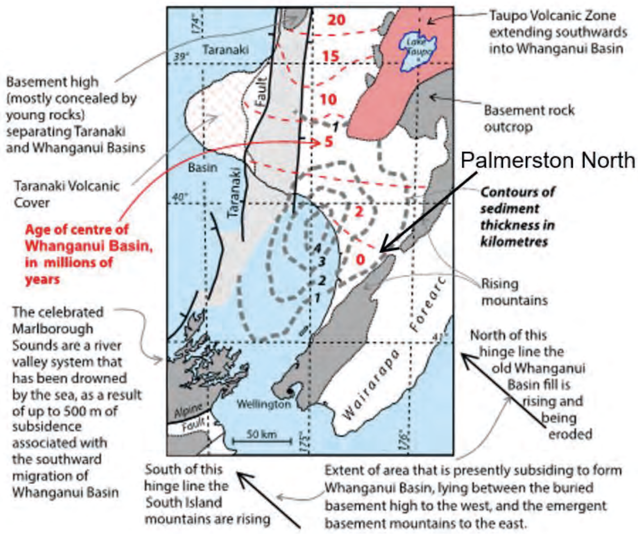


FIGURE 2: Whanganui Basin (extract from P. Balance, 2009)

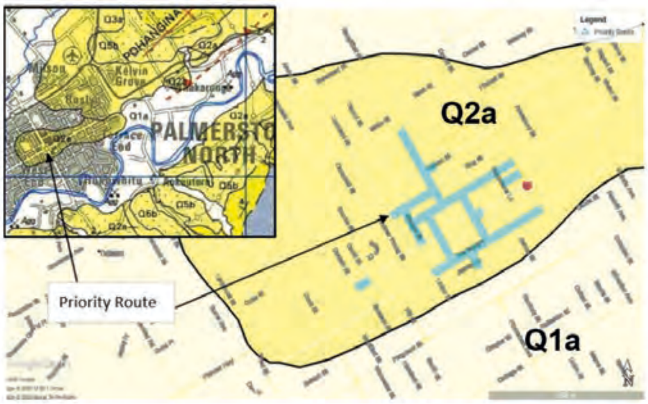


FIGURE 3: Geological Map Extract (Lee & Begg, 2002)

TABLE 1: MASW Survey Summary

Ref.	Road Name	Ch. (m)
MASW 1	Rangitikei Street	0 - 555
MASW 2	The Square-Broadway Ave	0 - 525
MASW 3	Berrymans Lane E	0 - 88
MASW 4	Queen Street	0 - 195
MASW 5	Cuba Street	0 - 285
MASW 6	George Street	0 - 182
MASW 7	Main Street NE	0 - 80
MASW 8	The Square NE	0 - 118
*MASW 9**	*King Street*	*0 - 55*
MASW 10	Coleman Place	0 - 60
MASW 11	The Square SE	0 - 242
MASW 12	Main Street SW	0 - 86
MASW 13	Church Street	0 - 620
MASW 14	Fitzherbert Avenue	0 - 102
*MASW 15**	*Fitzherbert Avenue*	*0 - 66*

* MASW lines 9 & 15 do not form part of the Priority Route and were collected for the purpose of correlation with existing machine drilled boreholes

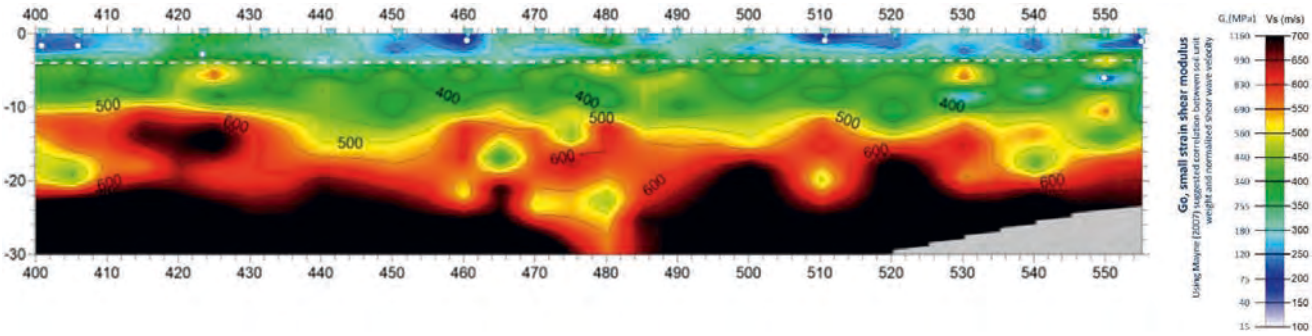


FIGURE 4: Indicative Shear Wave Velocity Profile from MASW Survey

A MASW geophysical survey was selected for the initial phase of the investigation in order to provide the most coverage over the entire Priority Route for a relatively low cost compared to other investigation methods. The MASW survey was undertaken by Southern Geophysical Ltd. on behalf of Miyamoto and included the collection of data over 15No. lines located within the road carriageways (Figure 1 & Table 1).

Thirteen of the lines are located along the Priority Route with an additional 2No. lines (MASW 9 & 15) undertaken at the location of existing machine boreholes to facilitate correlation of parameters and 'ground truthing'.

The resulting data has been processed into 2D shear wave velocity (V_s) subsurface profiles (to >25 m depth) identifying changes in stratigraphy and lithology along the survey lines, a typical example of which is shown in Figure 4. The full survey will be uploaded to the New Zealand Geotechnical Database (NZGD).

A number of engineering boreholes and well bores, and a GNS microtremor study available from the subject area have also been utilised in our assessment and for correlation purposes.

4. DATA REVIEW AND ASSESSMENT

Using the measured V_s, and the resulting velocity profiles from the MASW survey as a baseline, a preliminary understanding of the ground conditions has been developed through correlation with the existing available information, and correlation with geotechnical properties through published correlations.

4.1. MASW AND SHEAR WAVE VELOCITY DATA

The in-situ soil V_s measurement taken in the field is a small-strain soil property related to the 'undisturbed', small-strain or maximum shear modulus (G_o) of the soil and the soil-mass density (ρ).

As V_s is dependent on the stiffness and density of a soil generally, as soil depth increases so too does stiffness, thus V_s also increases. Table 2 shows the dependency of G_o and V_s on varying geological / geotechnical parameters.

TABLE 2: Effect of Various Factors on G_0 and V_s (after Dobry and Vucetic, 1987)

Factor / Parameter	Influence on G_0 & V_s
Confining pressure or overburden stress (i.e. depth)	Increases with σ'_v
Void ratio (e_o)	Decreases with increased void ratio (e_o)
Relative density (D_r)	Increases with relative density (D_r)
Geological age	Increases with geological or depositional age
Cementation	Increases with cementation
Overconsolidation ratio (OCR)	Increases with OCR
Strain rate or Frequency of cyclic loading	Increases with strain rate

The shear wave velocity was recorded to depths in excess of 30 m over the majority of the Priority Route, reaching maximum depth of ~44 m. Whilst there is some variability in the V_s measurements across the site, in general the data shows a fairly consistent underlying profile for such a large area, with:

- A time-average $V_{s,5}$ around 310 $^m/_s$ (~250-420 $^m/_s$) for the upper 5 m across the site;
- $V_{s,10}$ around 350 $^m/_s$ (~280-440 $^m/_s$), for the upper 10 m across the site;
- $V_{s,30}$ around 400 - 480 $^m/_s$ (SD) for the top 30 m.

Exceptions to the above generalisation are MASW lines 12 and 15 which show a greater thickness of material with lower shear wave velocities, indicative to softer/ looser soil layers. Both MASW 12 and 15 are fairly isolated (~300 m southwest and ~250 m southeast respectively) from the remainder of the Priority Route and located close to an apparent historic stream (named "Terrace" Stream as shown in GNS, 2011), so this may be expected. In addition, MASW 15, with lower $V_{s,5}$ and $V_{s,10}$ values, is located in a different mapped geology

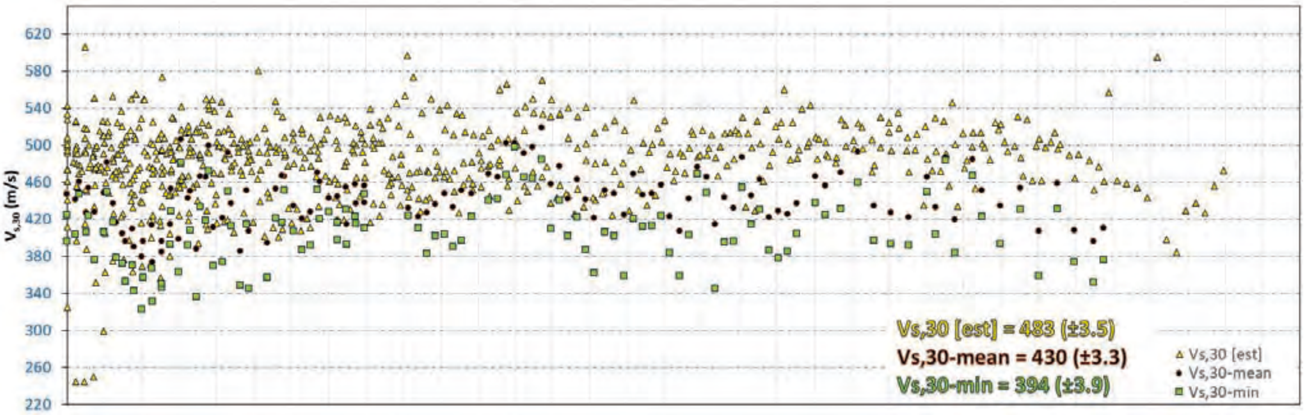


FIGURE 5: Vs,30 Interpretation (95% confidence interval shown in parenthesis)

and does not form part of the Priority Route.

$V_{s,30}$ is a widely used parameter in international practice for the classification of site class (discussed in more detail later in this paper) and is defined as the time for a shear wave to travel from a depth of 30 m to the ground surface. The time-averaged $V_{s,30}$ is calculated as 30 m divided by the sum of the travel times for the shear waves to travel through the individual layers as follows:

$$V_{s,30} = 30 / \sum (d/V_s) \qquad \text{[Equation 1]}$$

where: d = individual layer thickness and V_s = the shear wave velocity of the individual layer

As part of our assessment we have assessed $V_{s,30}$ for each of the MASW lines including minimum, maximum and mean values as shown in Figure 5.

Where V_s data was not recorded to 30 m depth (i.e. standard $V_{s,30}$ calculation invalid) minimum and maximum values of $V_{s,30}$ have been estimated by:

- Minimum: assuming the minimum value recorded for the missing data to 30 m depth;
- Maximum: assuming the deepest value recorded extends to 30 m depth (considered more probable than the minimum estimation).

4.2. CALIBRATION / CORRELATION WITH AVAILABLE DATA

As part of the initial stages of the project the following existing information was sourced:

- 3No. machine drilled boreholes with associated engineering logs in proximity of the Priority Route;
- GNS Science, Palmerston North City Council Buildings - microtremor analysis report (dated 1 February 2013);
- GIS data referenced in the GNS liquefaction assessment (Beetham et all, 2011) including gravel and groundwater depth information;
- A list of PNCC boreholes / well bores in the wider area, including some with lithology descriptions.

In general the, SPT N values from the machine boreholes correlated well with the V_s profiles in that they provide an indication of the transitions between denser and looser layers, however, there was a reasonably large spread in the data. Additionally, comparison of V_s with SPT N values is limited for dense soils such as the underlying gravel as the majority of the SPTs refused resulting in a value of 50+ and therefore no differentiation between denser soils.

GNS undertook a microtremor study (including HSVR[1] and SPAC[2] analyses) for a building on the Priority Route as part of an assessment of the NZS1170.5 Site Class for the building. A reasonable correlation can be seen between the data sets below ~4 mbgl, however, there is a fairly large spread in the data and GNS have derived higher V_s values (300 to 320 $^m/_s$) than those recorded during the MASW survey (~200 to 350 $^m/_s$) above 4 mbgl (refer to Figure 6).

The GNS study incorporated the lithology descriptions from the above referenced PNCC well bores, derived V_s profiles and estimated the depth to 'effective bedrock' (also referred to as 'engineering bedrock') where theV_s increases above a certain threshold (commonly 700 $^m/_s$). The depths to engineering bedrock presented by GNS are 76 m and 83 m for the geological and microtremor (SPAC) models respectively, the latter being GNS's preferred model. The study concluded that the site may be classified as NZS1170.5:2004 Site Sub-soil Class D, however, with a natural site period (T_n) of less than 0.7 seconds, the site falls very close to Class C.

A number of well bores were obtained from PNCC / GNS archives, however, the boreholes only have information related to lithology and are located in or close to the boundary of an area with a different mapped geology. As such, these boreholes have been used with caution in the assessment and as an indication of the deeper ground conditions.

Several GIS files were obtained from GNS Science, including contour plans of 'depth to groundwater', 'depth to gravel' and 'thickness of cover material above gravel'.

1 HVSR - Horizontal-to-Vertical Spectral Ratio, also referred to as the Nakamura Ratio

2 SPAC - Spatial Auto Correlation

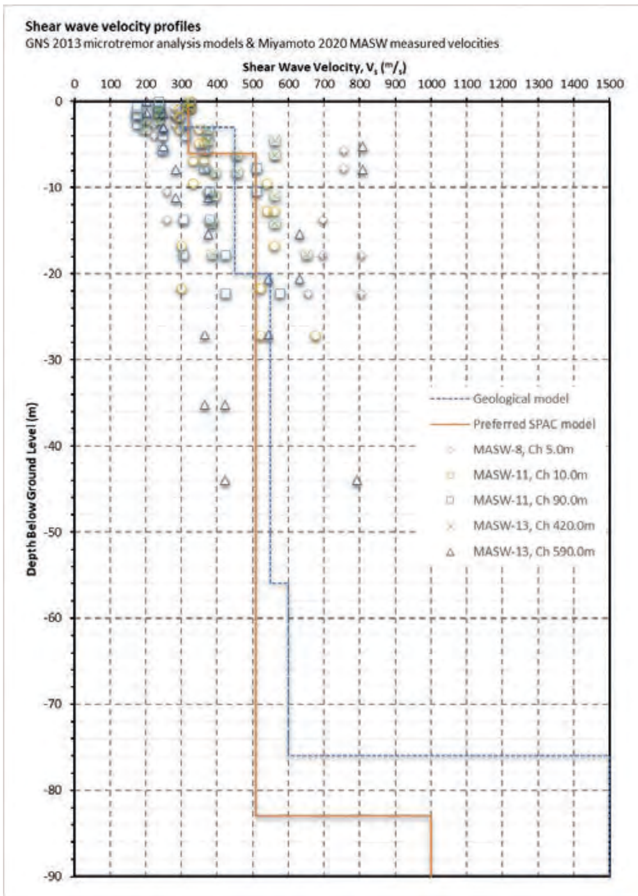


FIGURE 6: Vs Profiles - GNS 2013 Microtremor & Miyamoto 2020 MASW Surveys

The contour plans indicate the following:

- The depth to groundwater is indicated to range from 5.0 m to the north of the site to 7.0 m at the southern extent, while the depth to saturated soils above the gravels ranges from 2.0 to 5.0 mbgl which is closer to the recorded water levels in the boreholes along the priority route (1.2 m to 4.4 mbgl);
- The depth to gravel (and thickness of cover material) across the site ranges from 0.0 m (i.e. ground level) to the north of the site increasing to 4.0 m to 5.0 m depth to the south of the site. This loosely correlates with the other available information.

5. EVALUATION AND ASSESSMENT

5.1. PRELIMINARY GROUND MODEL

Based on our review of all available data, the ground conditions underlying the majority of the Priority Route are considered to be fairly consistent with dense to very dense gravel with varying sand and silt content and relatively thin isolated layers of silt / sand present from between ground level and ~4.0 mbgl. Where present, the material overlying the gravel is considered to comprise predominantly fine-grained material (silt & clay) with varying sand content.

The ground conditions as per the current study can be summarised as:

- Soil formations of dense to very dense sand – sand gravel and/or stiff to very stiff cohesive soils (silts and clays), of great thickness (> 60.0 m), whose mechanical properties and strength are constant and/or increase with depth.

With the exception of MASW 12 and 15 where:

- Soil formations of great overall thickness (> 60.0 m), interrupted by layers of recent soil deposits with soft and/or fairly loose materials of a small thickness (5 - 15 m), within soils of evidently greater strength and $V_s \geq 300\ ^m/_s$.

5.2. SEISMIC SITE CLASSIFICATION

Of critical importance in earthquake engineering and engineering seismology are the underlying soils and site characterisation. Existing seismic codes 'cover' ordinary structures for ground shaking characteristics for 'normal' soil-site conditions (i.e. not necessarily covering special soil-site conditions such as liquefaction, slope stability, etc).

For seismic design of structures, and / or for seismic strengthening projects, using the current seismic codes, the site of interest must be classified into one of the soil categories adopted by the code. Based on the soil class, the appropriate site-dependant design spectrum can be defined and used.

5.2.1. International Methods of Seismic Site Characterisation

Site characterisation schemes of the seismic codes use different descriptions of geological and geotechnical parameters to define soil classes.

As discussed earlier in this paper, a widely used parameter in the classification of site class is $V_{s,30}$. The U.S. seismic code (or International Building Code) proposes six soil types using the $V_{s,30}$ values as the main characterisation parameter, together with the SPT N (standard penetration test blow count) and S_u (undrained shear strength) values as 'secondary' index parameters.

Similarly, the current version of Eurocode 8 (EC8), $V_{s,30}$ is used as the main classification parameter, following the U.S. practice, along with SPT N, plasticity index (PI) and S_u as 'secondary' index parameters, for the five main and two special defined soil classes.

It is not the intention of this paper to discuss these methods in any detail, rather to touch on the different assessment criteria therein.

For comparison purposes, Figure 7 presents an extract of the Eurocode 8 and ASCE 7-16 site classification based on $V_{s,30}$ (Note - other classification criteria omitted), and Figure 8 presents a comparison of several seismic codes.

Eurocode 8			ASCE 7-16 (U.S. & IBC)		
Soil Type	Description	$V_{s,30}$ ($^m/_s$)	Soil Type	Description	$V_{s,30}$ ($^m/_s$)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface	>800	A	Hard Rock	V_s>1500
			B	Rock	750 <V_s≤1500
B	Deposits of very dense sand, gravel or very stiff clay, at least several tens of meters in thickness, characterized by a gradual increase of mechanical properties with depth	360 - 800	C	Very dense soil and soft rock	360 <V_s≤750
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres	180-360	D	Stiff soil profile	180 <V_s≤360
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil	< 180	E	Soft soil profile	V_s < 180
E	A soil profile consisting of a surface alluvium layer with V_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with V_s > 800 m/s	-	F	Other *	-

* Not reprinted here - Refer to Standard for details

FIGURE 7: Eurocode 8 & ASCE 7-16 Site Classification Criteria (extract only)

Code	Categories	Qualitative Criteria	Soil Stratigraphy	Vs	Additionally Criteria
ATC3	4	✓	✓	Vs	-
IBC2000	**6 (5+1*)**	✓	✓	**Vs,30**	**N_{SPT}, S_u**
EC8-EN	**7 (5+2*)**	✓	✓	**Vs,30**	**N_{SPT}, S_u**
Japan 2001	3	✓	✓	Vs,T1, T2, H	Descr.
France 1990	5 (4+1*)	✓	✓	Vs, Vp	N_{SPT}, S_u Dr, Cc, etc.
Turkey 2007	4	✓	✓	Vs, H	N_{SPT}, S_u, Dr
Norway 1998	3	✓	✓	Vs	Descr.
New Zealand	5 (4+1*)	✓	✓	Vs, Vs,30	N_{SPT}, S_u, T_o H< 100m

FIGURE 8: Site Soil Classification in Seismic Codes Comparison (Pitilakis et. al., 2015)

5.3. NZS1170.5:2004 SITE SUB-SOIL CLASS

Although the most commonly used parameter is the time-averaged vertical shear wave velocity in the uppermost 30 m ($V_{s,30}$), NZS 1170.5 adopts a mixture of parameters for different soil classes including but not limited to T_o (fundamental period), V_s, $V_{s,30}$, SPT N (standard penetration test blow count) and S_u (undrained shear strength) values.

The general classification criteria as per NZS1170.5 are shown in Figure 9.

The preferred methodology for assessment of the NZS1170.5 Site Sub-soil Class is based on measurements of shear wave velocity (V_s) or shear wave travel times through estimation of the site period, T_s using the following (general) equation:

$$T_s = 4H/V_s \qquad \text{[Equation 2]}$$

where: H = depth to bedrock (m)

As can be seen from Equation 2, the site period is highly dependent on the depth to bedrock, and with the depth

NZS1170.5:2004 Site Subsoil Classes.

Class	Description	Definition
A	Strong Rock	UCS > 50 MPa & Vs30 > 1500 m/s & not underlain by < 18 MPa or Vs 600 m/s materials.
B	Rock	1 < UCS < 50 MPa & Vs30 > 360 m/s & not underlain by < 0.8 MPa or Vs 300 m/s materials, a surface layer no more than 3 m depth (HW-CW rock/soil).
C	Shallow Soil	not class A, B or E, low amplitude natural period ≤ 0.6s, or depths of soils not exceeding those in Table 2.
D	Deep or Soft Soil	not class A, B or E, low amplitude natural period > 0.6s, or depths of soils exceeding those in Table 2, or underlain by < 10 m soils with undrained shear strength < 12.5 KPa, or < 10 m soils SPT N < 6.
E	Very Soft Soil	> 10m soils with undrained shear strength < 12.5 KPa, or > 10m soils with SPT N < 6, or > 10m soils with Vs ≤ 150m/s, or > 10m combined depth of previous properties.

Maximum depth limits for site subsoil class C.

Soil type and description		Maximum depth of soil (m)
Cohesive Soil	Representative undrained shear strengths (KPa)	
Very soft	< 12.5	0
Soft	12.5-25	20
Firm	25-50	25
Stiff	50-100	40
Very stiff or hard	100-200	60
Cohesionless Soil	Representative SPT N values	
Very loose	< 6	0
Loose dry	6-10	40
Medium dense	10-30	45
Dense	30-50	55
Very dense	> 50	60
Gravels	>30	100

FIGURE 9: NZS1170.5:2004 Site Sub-soil Classification Definition

to 'actual' greywacke rock understood to be in the order of 1 km at the site, NZS1170.5 Site Sub-soil Class D (Deep or soft soil sites) may seem an appropriate classification. However, as discussed earlier the underlying very dense gravels may be considered 'engineering bedrock' at a certain depth where the V_s increases above a threshold (700 $^m/_s$ commonly adopted) and consideration as such may give rise to much shorter site periods which may be more in line with Site Sub-soil Class C (Shallow Soil Sites).

Currently there is a considerable increase in amplification between the NZS1170.5 design spectra for Site Class C (shallow soil) and D (deep or soft soil) sites, leading to significantly different design criteria when undertaking seismic performance assessment of buildings (i.e. estimation of %NBS) and seismic structural design (i.e. design of a seismic strengthening scheme). As a generalisation, the adoption of Class D over Class C results in an increase of up to 63% (for a certain period range) the design response spectra leading to more onerous structural demands and ultimately increased development costs (i.e. overall building, building renovations, seismic strengthening schemes etc.).

With the Priority Route being assessed as very close to or below the boundary between Class C and D, i.e. close to a 0.6 s site period, potential benefit may be realised through more thorough investigation and assessment, of course depending on the proposed development.

To demonstrate the potential benefits of further site-specific investigation and more rigorous assessment, a 'hypothetical' sensitivity assessment of the site period along the Priority Route has been undertaken. The assessment incorporated the Miyamoto MASW measured V_s data, where available (i.e. up to 30 - 44 m depth), in combination with the GNS derived V_s profiles (Section

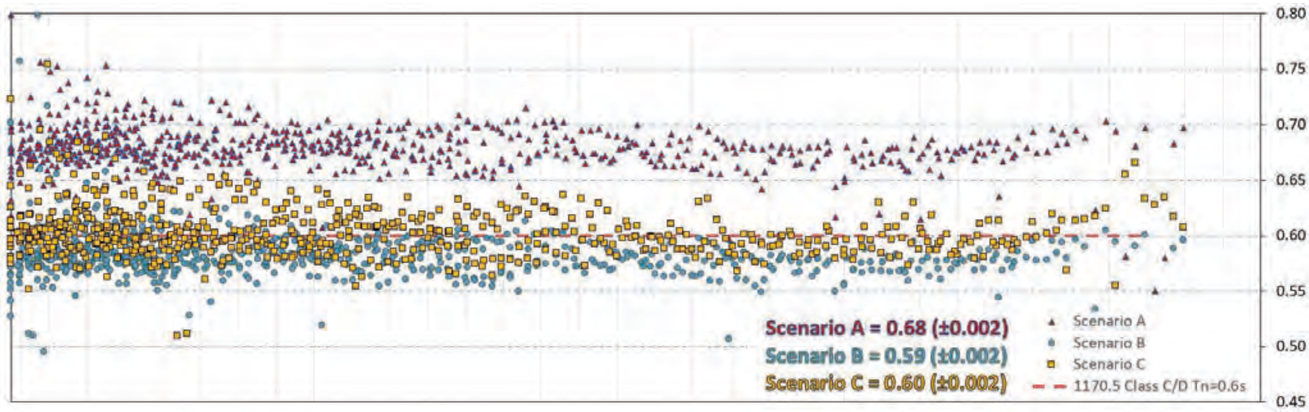


FIGURE 10: 'Hypothetical' Assessment of Potential Site Period (95% confidence interval shown in parenthesis)

4.2), and slight variations of such, below. The following three scenarios were assessed:

- Scenario A: MASW measured V_s data to max. 44 m depth over the GNS 'preferred' V_s profile (i.e. V_s = 509 $^{m}/_{s}$ to 'engineering' bedrock at 83 m).
- Scenario B: MASW measured V_s data to max. 44 m depth over the GNS 'geological model' V_s profile (i.e. V_s = 550 to 600 $^{m}/_{s}$ to 'engineering' bedrock at 76 m).
- Scenario C: MASW measured V_s data to max. 44 m depth and V_s = 600 $^{m}/_{s}$ to effective bedrock at 83 m.

The results of the assessment are plotted in Figure 10 and demonstrate how sensitive the assessment is to fairly small changes in the underlying ground conditions (V_s and / or depth to bedrock), with the 'possible' ranges of site period for the three scenarios straddling 0.6 s (Site Sub-soil Class C / Class D).

Furthermore, in the event that further site-specific investigation and assessment resulted in NZS1170.5 Site Sub-soil Class D for a specific development project, as opposed to full compliance with the Class D design criteria, alternative assessment may be feasible such as site specific response analysis or the procedure put forwards by McVerry (2011).

To reiterate, this assessment is hypothetical (albeit potentially realistic) and is provided to demonstrate the significant potential for benefits to be realised through site-specific investigation and more rigorous assessment.

6. CONCLUSIONS AND RECOMMENDATIONS

The main objective of this study was to provide PNCC, building owners and practitioners with additional geotechnical information, at a preliminary level, to assist with seismic assessment and / or seismic strengthening, repair or maintenance works to the buildings along the Priority Routes.

To provide the most coverage over the entire Priority Route for a relatively low cost compared to other investigation methods, a MASW geophysical survey was undertaken and calibrated with available existing data. The data from which (shear wave velocity, V_s profiles) in itself is an extremely valuable indicator of dynamic soil properties and a useful parameter for seismic assessment.

Through interrogation and assessment of the data it is demonstrated that there may be significant benefit in further site-specific investigation and assessment of the NZS1170.5 Site Sub-soil Class, and that Class C may be appropriate for sections of the route. Furthermore, for sites that do fall into Class D, alternative methods of assessment may be warranted so that full compliance with Class D design criteria may not be required. However, the level of additional investigation and assessment will be dependent on the scale of the proposed development and potential cost savings.

The adoption of Site Sub-soil Class C as opposed to Class D will have significant beneficial effects to seismic performance assessment of buildings (i.e. estimation of %NBS) and seismic structural design (i.e. design of a seismic strengthening scheme).

Although this study provides significantly more geotechnical information to that previously available, further targeted investigation and assessment is required.

The project is ongoing we plan on publishing the findings of the future stages.

7. ACKNOWLEDGEMENTS

The authors would like to acknowledge PNCC for the positive impact this project will have for the city centre building owners and the wider Palmerston North community. We would also like to thank and dedicate this work in memory of our late colleague and friend Brent Neale for his encouragement and support. Lastly, we thank Clem Gibbens for his collaborative work on data interpretation also informed this study.

8. REFERENCES

Balance, P., 2009. New Zealand Geology: an illustrated guide. Geoscience Society of New Zealand, miscellaneous publication 148

Beetham et al., 2011. Assessment of liquefaction and related ground failure hazards in Palmerston North, New Zealand, GNS Science Consultancy Report 2011/108, July 2011

Begg, J.G., Johnston, M.R. (compilers) 2000. Geology of the Wellington Area. Institute of Geological and Nuclear Sciences 1:250,000 geological map 10. 1 sheet +64p. Lower Hutt, NZ

Boore, D.M, et al., 2011. Regional Correlations of V_s30 and Velocities Averaged Over Depths Less Than and Greater Than 30 meters. Bulletin of the Seismological Society of America, 101(6):3046-3059.

Dobry, R., Vucetic, M. 1987. Dynamic Properties and Seismic Response of Soft Clay Deposits. Proceedings of the International Symposium on Geotechnical Engineering of Soft Soils. August 1987.

GNS Science. 2013, Palmerston North City Council Buildings - microtremor analysis report.

Hardin, B. O., Drnevich, V.P. 1972. Shear Modulus and Damping in Soils: Design Equations and Curves. Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, July 1972

Institute of geological and Nuclear Science, Palmerston North City Council Buildings – Microtremor analysis, 1 Feb 2013

Landcare Research - Manaaki Whenua, 2010. Soil Map of Palmerston North City and environs, NZ. Published on LRIS portal.

Lee, J.M., Begg, J.G. (compilers) 2002. Geology of the Wairarapa Area. Institute of Geological and Nuclear Sciences 1:250,000 geological map 11. 1 sheet +66p. Lower Hutt, NZ

Mayne, P.W. 2007. In-Situ Test Calibrations for Evaluating Soil Parameters

McVerry, G.H. 2011. Site-effect terms as continuous functions of site period and V_s30. Proceedings of the Ninth Pacific Conference on Earthquake Engineering, Auckland, New Zealand

New Zealand Geotechnical Database, 2020. Accessed via Google Earth from https://www.nzgd.org.nz/

New Zealand Geotechnical Society (NZGS) and Ministry of Business, Innovation and Employment (MBIE) (2016). Earthquake geotechnical engineering practice Module 3: Identification, assessment and mitigation of liquefaction hazards, May 2016.

New Zealand Standard NZS1170.5 2004. Structural Design Actions, Part 5: Earthquake Actions - Standards New Zealand, 2004

Palmerston North City Council Local Maps Viewer. Accessed via https://geosite.pncc.govt.nz/MapGallery/

Pitilakis et al., 2015. Site characterization and seismic codes, Orfeus Annual Observatory Coordination Meeting

Semmens et al., 2011. NZS 1170.5:2004 site subsoil classification of Wellington City. Proceedings of the Ninth Pacific Conference on Earthquake Engineering, Auckland, New Zealand

Observations on Expansive and Shrinkable Soils

Laurie Wesley, retired geotechnical engineer

Laurie Wesley
At the start of his career as a geotechnical engineer Laurie spent two terms of four years working for the Indonesian government interspersed by five years with the New Zealand Ministry of Works. Following this he completed a PhD at Imperial College, and returned to Auckland to work for Tonkin and Taylor for eleven years. He then lectured at Auckland University for 15 years, and still does some part time teaching, in Auckland, Indonesia, and Chile.

INTRODUCTION

I am writing this in response to the paper by Nick Rogers and colleagues in the June, 2020, issue of NZ Geomechanics News. I should say at the start that I agree totally with their conclusion regarding the shrink/swell test. The swell shrink behaviour of soils is not something I claim to be an expert on but I have read numerous papers on the subject, and have had some experience of the problems associated with such soils. The criticism I would make of many published papers is that their authors seem not to understand the basic physics or mechanics that give rise to shrink and swell problems. In particular too much focus is given to the properties of the soil without giving adequate attention to the environment in which it exists. The focus of this article is therefore on basic principles. My apologies if what I am saying is already well known.

WHAT ARE SWELLING AND SHRINKING SOILS?

This is not really the appropriate question to ask. The more appropriate question is: what are the conditions that give rise to swelling or shrinking problems? Focussing solely on the properties of the soil does not provide an answer to this question. For example, on the West Coast of the South Island, there could be soils that laboratory tests classify as shrink-swell soils but it is very unlikely that they would cause either swelling or shrinkage problems (because of the wet climate of that region). It is evident therefore that of equal importance with the nature of the soil is the state in which it exists in the ground, which in turn is governed by the local climate. For another example, the climate in New Zealand is generally wet and our temperatures only moderate, while those in large parts of Australia are the opposite. Thus, in Auckland damage to houses is almost always caused by shrinkage while that in Adelaide by swelling. The soils involved may have identical properties

WHAT SORT OF SOIL HAS THE POTENTIAL TO CREATE SHRINKAGE OR SWELL PROBLEMS?

This question is relatively simple to answer. Any clay of high plasticity has this potential, and Atterberg limit tests plus the Casagrande plasticity chart provide reliable identification of such soils. Figure 1 illustrates this. Soils

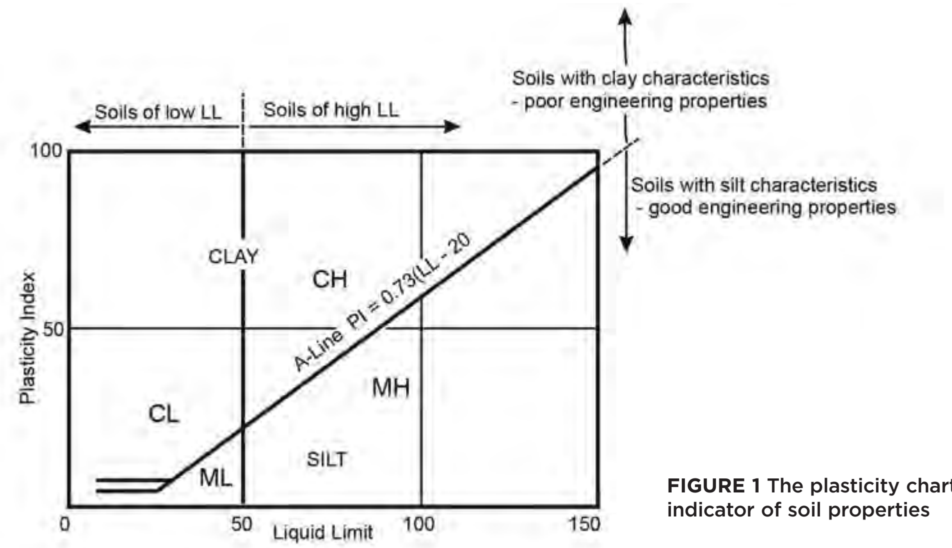


FIGURE 1 The plasticity chart as an indicator of soil properties

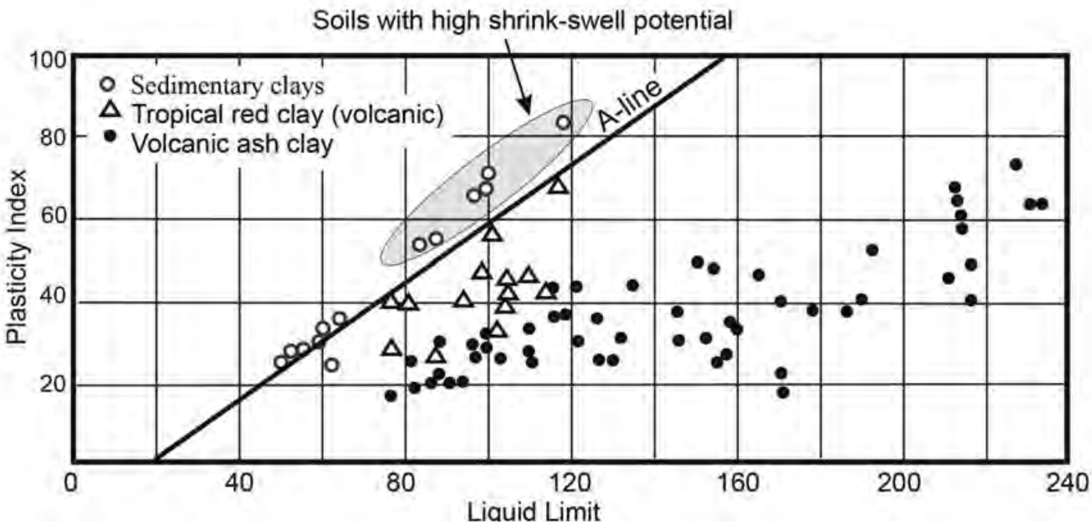


FIGURE 2 Plasticity Chart showing natural soils expected to have the potential to cause shrink or swelling problems

with liquid limit above 50 that plot significantly above the A-line will almost certainly be of high plasticity and have less than favourable soil properties. In particular they will have the potential to cause swell or shrink problems . Regarding the plasticity chart, the following points should be noted.

- The horizontal scale is of Liquid Limit so that CH means clay of high liquid limit and not high plasticity. Casagrande made this clear when he created the chart and it still has the same meaning today despite the frequent but mistaken assumption that the H indicates high plasticity.
- It is not sensible to try to relate soil properties to either plastic limit or liquid limit. It is the position of the soil in relation to the A-line that is the most reliable indicator of soil properties (see Wesley, 2003)
- One "weakness" of this approach is that the Plasticity Chart is only useful if the Atterberg Limit values are reliable. This may be stating the obvious, but it is often assumed that the tests are so elementary that no special attention is needed to ensure their accuracy. In my experience this is not always the case. As readers may know I have published various papers containing Atterberg Limits. I did most of the tests myself, the remainder by technicians that I trained or supervised.

With regard to Figure 1, my text book (Wesley, 2010) contains the following statement:

The Atterberg Limit values by themselves are not particularly reliable as indicators of soil behaviour, but become very useful when plotted on the Plasticity Chart. Soils that plot well above the A-line generally have poor engineering properties. They are likely to be of high compressibility and low shear strength, and display shrink/swell behaviour. On the other hand soils that plot below the A-line tend to have the opposite characteristics and be good engineering materials.

Along with this general trend will be another trend dependent on the Liquid Limit. For a given position in relation to the A-line, the higher the Liquid Limit the less desirable will be the engineering characteristics.

Figure 2 shows some data I have collected over the years, mostly from Indonesia; it reinforces the point made with Figure 1.

	Fully saturated	Partially saturated
In natural state, no external stress, no water available to soil, (undrained).	Pore pressure = u $\sigma' = \sigma - u = -u$ $\sigma' = -u$	Pore pressure = u $\sigma' = \sigma - \chi u = -\chi u$ $\sigma' = -\chi u$
Confined in apparatus, external stress applied, to prevent volume change, water freely available to soil, (drained).	σ_s Pore pressure = 0 $\sigma' = \sigma_s - u = \sigma_s - 0 = \sigma_s$ $\sigma' = \sigma_s$ Therefore $\sigma_s = -u$	σ_s Pore pressure = 0 $\sigma' = \sigma_s - u = \sigma_s$ $\sigma' = \sigma_s$ Therefore $\sigma_s = -\chi u$

FIGURE 3 Stress state in a soil sample when first set up in an apparatus and after it is allowed to soak up water.

A NOTE ON THE SWELLING PRESSURE AND ITS RELATION TO NEGATIVE PORE PRESSURE

Various methods are available to measure the swelling pressure, which is of some use although it is the magnitude of the swell or shrinkage that is of primary interest. The swelling pressure is normally taken to be the pressure that must be applied to the soil to prevent it from swelling when water becomes available to it. An undisturbed sample can be prepared in the same way as an oedometer test sample, and then the pressure measured to prevent the soil swelling when water is made available to it. Figure 3 illustrates the stress state when the sample is set up in the apparatus, and the same state after freely absorbing water.

Application of the simple equation relating effective stress to total stress and pore pressure shows that for a fully saturated soil the swelling pressure is the same as the negative pore pressure. For a partially saturated soil it is less than the negative pore pressure by the Bishop factor χ, the value of which is approximately given by the degree of saturation. The negative pore pressure is frequently called "suction" although the more logical technical term is "pore water tension".

THE BASIC MECHANISM OF SOIL SWELLING OR SHRINKAGE.

Swelling and shrinkage phenomena involve volume change and are thus governed by the same laws as any other volume change, namely a change in effective stress. The simplest law is

.
$$\frac{\Delta L}{L} = m_v \, \Delta\sigma'$$

If we determine the value of the soil compressibility coefficient m_v and the change in effective stress $\Delta\sigma'$ then we can calculate the strain in any particular layer. Expansive and shrinkage usually (but not always) occurs under constant total stress. This total stress is the weight of the soil, or of the soil plus a foundation load. This means that $\Delta\sigma' = -\Delta u$, the change in the pore pressure. Thus to understand expansive or shrinkage behaviour we need to understand the way in which pore pressures change with seasons or weather.

THE INFLUENCE OF CLIMATE AND PORE PRESSURE CHANGES IN THE GROUND .

For simplicity we will consider the situation of a flat site with a fully saturated clay and an average water table depth some distance below the surface. If the situation is static then the only possible pore pressure state is the hydrostatic one. The pore pressure below the water table increases linearly with depth, and that above decreases linearly as shown in Figure 4. This hydrostatic state will exist if the following two conditions exist:

1. the ground surface is sealed, such as by a wide foundation or a sealed pavement (most likely concrete or bitumen)
2. the average depth of the water table is constant.

We need to recognise that clays can remain fully saturated for many metres or tens of metres above the water table, due to the combined effects of capillary forces and the very high air entry value of fine grained soils. Figure 4 also illustrates the possible changes in the pore pressure state due to seasonal changes. During a dry season, water will be drawn upwards towards the surface due to evaporation, and the probable pore pressure state resulting from this is indicated in Figure 4. During a wet season water will seep into the ground resulting the state also indicated in the figure.

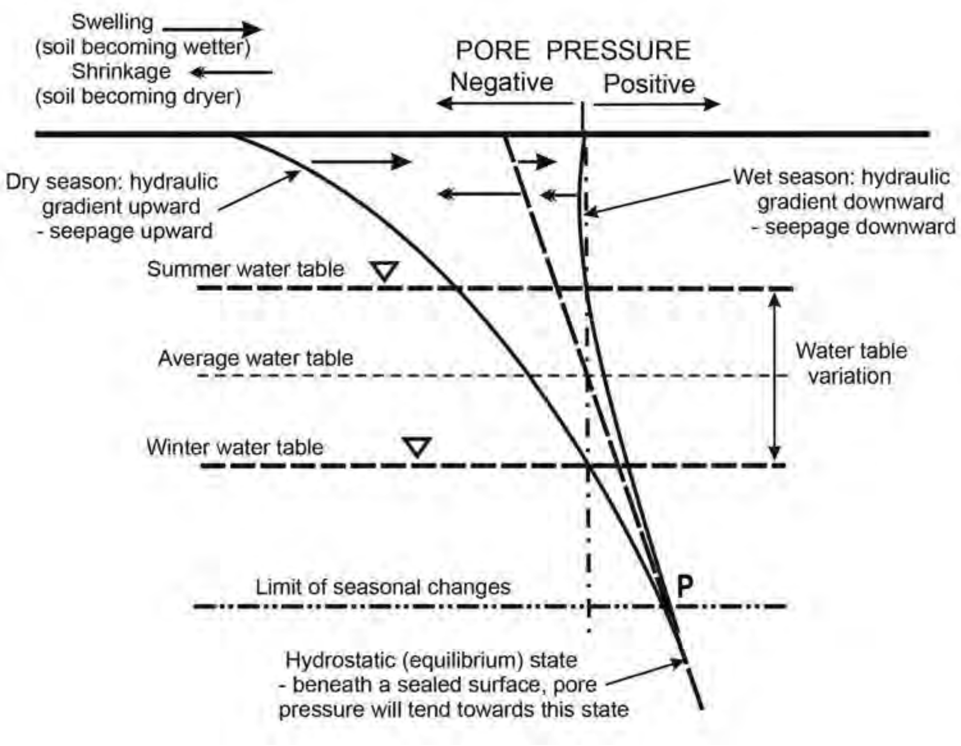


FIGURE 4 Pore pressure changes in a clay layer of moderate permeability

In drawing Figure 4 the soil is assumed to be of moderate permeability, in which case seasonal influence penetrates deep enough to cause changes in the water table. It rises during the wet season and goes down in summer. In very low permeability clays the seasonal influence may not penetrate deep enough to cause changes in the water table. This is generally the case for London clay and other heavily over-consolidated sedimentary clays.

When the ground surface is covered by a building foundation or a pavement, water can neither enter nor leave the surface and the pore pressure will move towards the hydrostatic position shown in Figure 4. This tells us directly the change in the pore pressure and thus the change in effective stress from which compression can be calculated. The following points should be noted:

1. The volume changes that will occur when a foundation is built, or a pavement laid, are very dependent on seasonal or climate effects. If construction is toward the end or a long dry summer then the pore pressure will steadily rise toward the hydrostatic state and the effective stress will decrease, causing the soil to swell. If construction is towards the end of a wet season, the reverse will occur.
2. The only pore pressure information that can be known with reasonable certainty is the depth of the water table and the associated hydrostatic equilibrium condition. The wet weather condition can be estimated as it clearly has an upper limit, but the dry weather state can only be guessed at unless local measurements have been made.
3. To determine the value of the compression parameter m_v a sample can be set up in an oedometer and then subjected to several loading and unloading cycles simulating the expected in situ stress cycles on the soil caused by seasonal changes.
4. It is probable that the soil close to the surface will become partially saturated during long dry seasons. This will complicate the stress state in the soil near the surface, although the general principles will remain the same

SOME GENERAL OBSERVATIONS AND CONCLUSIONS

An adequate definition of an expansive clay would be on the following lines:

> *A clay that contains a significant content of active clay minerals that is normally held in a compressed state by high pore water tension.*
> Such a clay is likely to swell when water becomes available to it

A similar definition of a clay likely to cause shrinkage would be:

> *A clay that contains a significant content of active clay minerals that is not normally compressed by high pore water tension.*
> Such a clay is likely to shrink during hot dry weather

The issue of foundation damage to buildings, especially normal residential houses, received some attention during the time I worked for Tonkin and Taylor (1976-86), and I recall doing a review of the cases in T&T files involving house damage. Almost invariably, damage was the result of soil shrinkage during long dry summers and the soil was weathered Waitemata clay. Damage was most frequently observed in brick veneer cladding or concrete block walls of basements, especially on ground sloping to the north. The shrinkage could be reversed to some extent by telling owners to water the ground adjacent to the affected areas. Houses on volcanic soils appeared exempt from such damage. Moves were made to examine the situation in Australian and possibly adopt their approach in New Zealand. This did not seem sensible to me because of the great difference in our climates.

The only time I have encountered an expansive clay problem in Auckland was when a concrete floor was laid on the surface of clay fill that had been placed and compacted in a very dry state. The clay slowly soaked up water and swelled. The walls of the house were on strip foundations taken down to natural ground so there was differential movement between the floor slab and the walls.

Regarding London Clay, the case of swelling following the removal of elm trees mentioned by Rogers et al, is I think, a rather unusual exception to the shrinkage that normally causes foundation damage in London. The event that brought soil shrinkage to everyone's attention in London was the so called "great drought" of 1976 which caused shrinkage damage to thousands of buildings, especially houses.

Regarding the Rogers et al paper I would make the following comments:

1. The conclusion that the results of shrink swell tests are strongly dependent on the initial water content is correct and to be expected if we consider Figure 4. There is a close connection between the pore pressure state and the water content.
2. The terminology is a bit "loose". As indicated above, the climate in most parts of New Zealand is sufficiently wet that problems from soil swelling are rare. Our conditions are quite different from Australia's and our soils should not be referred to as expansive.
3. The Rogers paper is very valuable as it clearly shows the unreliability of the shrink swell test, and makes useful suggestion for a better approach.

My own view is that trying to devise a single test applicable it to all parts of New Zealand is probably not a good idea. A better approach would be to conduct a survey of the major cities of New Zealand to establish the prevalence of foundation damage caused by soil shrinkage or swelling. It may be that some cities are free of such damage, even though they may contain highly plastic clays with the potential to cause shrink swell problems. A more detailed survey might be able to relate damage severity to both the position of the Atterberg Limits on the Plasticity Chart, and the local climate.

Just for interest, I built a room onto my house in Birkenhead shortly after joining T&T in September, 1976. My lawn at the time had many cracks in it because of a dry summer. To minimise movement from seasonal changes I put the concrete foundation strip on bored piles taken to a depth of 1.2 m if I remember correctly. This was just a simple hand auger job, and was very successful in stopping movement of the new room. However, it was not particularly clever as the rest of the house still moves with the seasons. Its influence is quite minor - some doors or windows jam a little more at some times of the year than others.

REFERENCES

N. Rogers, N.McDougal, G.Twose, J.Teal, T.Smith. (2020). The Shrink Swell Test: a Critical Analysis. NZ Geomechanics News June, 2020.

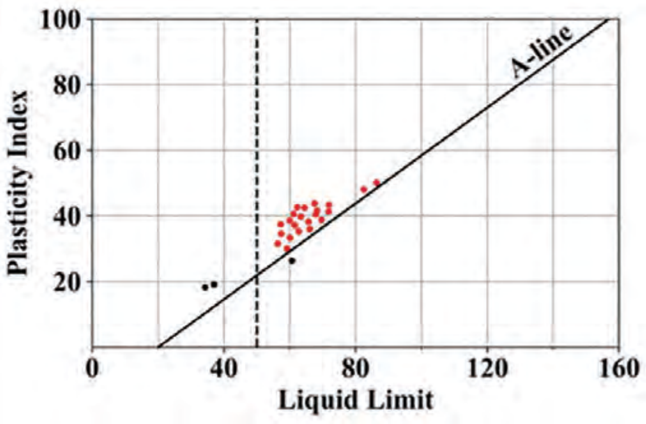
Wesley, L.D. (2003) Residual strength of clays and correlations using Atterberg limits. Geotechnique, Vol 53, No7 pp381-385.

Wesley, L.D. (2010) Fundamentals of Soil Mechanics for Sedimentary and Residual Soils. John Wiley and Sons New York

Authors' response

We appreciate the comments by Dr Wesley, who makes some very good suggestions on what our focus should be going forward as a technical group. Wesley is correct in suggesting that terms should reflect the problem: "expansive" is not a particularly helpful term for describing the behaviour of soils in New Zealand where most of the damage is caused by clay shrinkage. However, that is the term used in NZS 3604. Hopefully MBIE will consider changing the term. Volume Change Potential, as used in the UK, seems a much more appropriate term.

Dr Wesley also reinforces a really important finding (reported in our original paper) by Wesseldine (1980) around plasticity. Wesseldine tested soils from beneath 25 houses that became damaged due to soil shrinkage. Although we did not produce the figure in our paper, we did present it to the NZGS earlier in the year (Rogers and McDougall, 2020), and it is reproduced in the figure below. This figure strongly supports the findings of Wesley that soils exhibiting high plasticity, with LL greater than 50 and plotting above the A-line, have a high volume change potential. Of the 25 test results reported by Wesseldine, 22 (or 88%) fell into this category (shown in red).



REFERENCES

Wesseldine, M (1980). House Foundation Damage Caused By Shrinkable Clays. Presented to the Auckland Branch, NZIE, 13 November 1980

Rogers, N. W. and McDougall, N. J. (2020) Shrink/Swell Testing of Expansive (Reactive) Soils: A Critical Analysis. Online Presentation for the New Zealand Geotechnical Society, 24 April 2020. Accessed 22 June 2020 from https://www.nzgs.org/library/shrink-swell-testing-of-expansive-reactive-soils-a-critical-analysis/

Shallow replacement of Expansive or Swelling Soils

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Andreas Giannakogiorgos

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Fabio Parodi

Charles McDermott

David Sullivan

IN THE JUNE 2020 issue of NZ Geomechanics News 'The Shrink Swell Test: A Critical Analysis', Rogers et al., 2020, was published. This article provided an overview around the use of the Shrink Swell Test and current design practice for foundation design in NZ's expansive soils in accordance with AS 2870:2011 site soil class.

This paper provides an additional summary of the currently existing methods/ways for identification of expansive soils (defined herein as soils susceptible to both swelling and shrinkage) and presents a discussion on recently observed construction practice comprising shallow 500mm replacement of expansive soils with compacted granular fill as a 'measure' to reduce the deformation of the expansive soils and mitigate the risk of structural damage to a slab foundation.

INTRODUCTION

'**Expansive or swelling soils'** are soils that are likely to experience volume change due to changes in moisture content (defined herein as soils susceptible to both swelling and shrinkage). Characteristic expansive or swelling soils include highly plastic clays and clay shales that often contain colloidal clay minerals such as montmorillonites.

Expansive soils absorb large quantities of water after rainfall or due to local site changes (such as leakage from water supply pipes, sewage or drains), becoming sticky and heavy (Adem and Vanapalli, 2015). Equally, they can become stiffer when dry, resulting in shrinking (reduction of volume) and cracking of the ground (i.e. desiccation cracks).

Types of structures most often damaged from this 'hardening-softening' behaviour (Jones and Jefferson, 2012) of shrink and swell cycles include foundations and walls of residential and light (one or two-story) buildings and retaining walls. Lightweight dwellings, warehouses, garages/sheds, services and pavements are especially vulnerable to damage from swelling soils as these structures are less able to suppress the differential heave of the swelling soil than heavy, multistory structures.

IDENTIFICATION OF SWELLING (EXPANSIVE) SOILS

It is important to recognize the existence of swelling soils and to understand the problems that can occur with these soils as early as possible.

Field study is used to assess the presence, extent, and nature of swelling soil and groundwater conditions. The two major phases of field exploration are surface examination and subsurface exploration.

It is not hard to recognize expansive soils in the field. The presence of surface fissures is quite common in expansive soil deposits, and an indication of their swelling potential.

The swelling potential of a soil basically depends upon its mineral composition along with its in situ moisture content, voids ratio and density. Soil permeability also affects the rate of swelling on site.

In general, clays with plasticity indices (PI) > 25, liquid limits (LL) > 40, and natural water content (w) near the plastic limit or less are more likely to swell.

A number of testing methods and proposed classification systems exist to detect whether a soil is expansive and to predict the magnitude of shrink/swell.

Moreover, although there is no internationally recognised test for soil shrink swell potential or design methodology, a lot of work has taken place for possible correlations between other well-established measures of soil characteristics/behaviour.

AASHTO (T 258-81: Standard Method of Test for Determining Expansive Soils) prescribe a method to detect whether a soil is expansive and to predict the amount of swell. This is done by relating the Atterberg Limits of the soil to the natural soil suction at the time of construction, as shown in the Table 1 below.

TABLE 1: AASHTO guidelines on assessing expansive soils

Degree of expansivity	Liquid Limit %	Plasticity Index %	Soil suction	
			kPa	pF
Low	<50	<25	<144	<3.17
Marginal	50-60	25-35	144-383	3.17-3.59
High	>60	>35	>383	>3.59

In the United Kingdom, the National House Building Council (NHBC), amongst others, provides guidance on the identification of ‘shrinkable’ soils and has adopted a procedure whereby a ‘Modified Plasticity Index (I’p)’ is assessed that gives consideration to the fines content as follows:

$$I'p = Ip \times \frac{\% < 425\mu m}{100\%}$$

The ‘modified plasticity index’ is then related to ‘Volume Change Potential’ as shown in Table 2.

TABLE 2: UK NHBC Volume Change Potential

Modified Plasticity Index	Volume Change Potential
≥40%	High
20% to <40%	Medium
10% to <20%	Low

In New Zealand, expansive soils are excluded from the **NZ3604:2011** ‘good ground’ and defined as those having:

- A liquid limit ≥ 50% when tested in accordance with NZS4402 Test 2.2, and
- A linear shrinkage of more than 15% when tested from the liquid limit in accordance with NZS4402 Test 2.6

If soils are identified as ‘expansive’, the building designer is referred to section 17 (expansive soils), which in turn refers to various site soil classes described in AS 2870:2011 (Rogers et al, 2020). Table 3 presents the likely anticipated surface movements and typical foundation depth to protect the structure against damage by swelling or shrinking soils, as proposed by AS 2870 and MBIE November 2019 amendment.

TABLE 3: AS 2870:1196 & MBIE November 2019 B1/AS1 amendment NZ soil classes

Site Soil Class	Relative Expansion	Characteristic surface movement (mm)	Soil embedment D_e (mm)*
S	Slight	22	375/500
M	Moderate	44	525/550
H	High	78	575/775
E	Extreme	90	625/800

* for reinforced concrete foundations in expansive soils for light and medium to heavy wall cladding respectively.

The site soil class as per AS 2870 is commonly determined based on the shrink-swell index (I_{SS}), as assessed through the shrink-swell test in accordance with AS 1289:2003. Rodgers et al (2020) have provided a comprehensive assessment of the reliability of the shrink-swell test which is not discussed further herein.

Tables 4 and 5 (modified from Asuri and Keshavamurthy, 2016), provide an overview of a number of currently available relationships between index soil properties and swell potential, which may be an indication for the reasons why there is not a unified or internationally accepted method or criteria.

TABLE 4: Soil classes based on liquid limit and plasticity index

Swell potential	LL - Liquid Limit (%)		
	Chen	Snethan et al	IS: 1498
Low	<30	<50	20-35
Medium/marginal	30-40	50-60	35-50
High	40-60	>60	50-70
Very high	>60	-	70-90
	PI - Plasticity Index (%)		
	Chen	Holtz & Gibbs	IS: 1498
Low	<15	<18	<12
Medium	10-35	15-28	12-23
High	20-55	25-41	23-32
Very high	>35	>35	>32

Overlap of categories shown in Table 4 reflects the fact that the values were obtained from multiple sources.

TABLE 5: Soil classes based on particle size, activity and oedometer tests

Swell potential	% clay size fraction	Activity (A_c)*	% Expansion in oedometer cell	
	Chen		Holtz & Gibbs	Seed et al
Low	<30	<0.75	<10	<1.5
Medium	30-60	0.75-1.25	10-20	1.5-5
High	60-95	>1.25	20-30	5-25
Very high	>95	-	>30	>25

* $A_c = {}^{PI}/_{\% \text{ clay}}$, where clay defined as d<5Qm

NZ FOUNDATION DESIGN IN EXPANSIVE SOILS / AS 2870:2011 FRAMEWORK

The Australian Standard AS 2870:2011 is the 'codified' design approach for slab foundations in areas with expansive soils. The approach to the calculation of the potential shrink-swell movements presented in AS 2870 is based on the moisture flow model developed by Richards (1967) and is generally accepted as being the most appropriate method available for the design of residential slabs and footings.

As mentioned, AS 2870, and NZS 3604, classifies the expansiveness of soil by the characteristic surface movement (y_s), which corresponds to ground settlement or heave within the design life of the structure due to change in soil moisture content.

Mitchell (1980) assumed a "trumpet" shaped suction profile (Figure 1) to describe the concept of swelling of expansive soil.

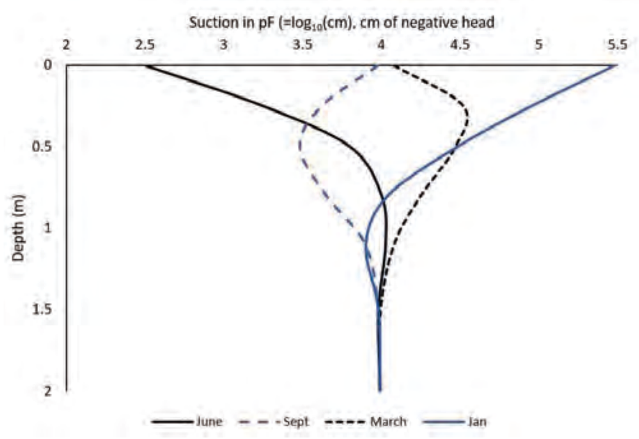


FIGURE 1: Theoretical suction profiles with depth for June, September, March and June (reworked after Mitchell, 1980)

The suction profile is approximated to a triangular shape in AS2870 (1996 and 2011) with values of change in suction at the soil surface (i.e. Δu in pF), as well as the recommended depth of design soil suction change are given for various locations in Australia. Soil suction is referred to as the potential energy state of water in the soil (Jury et al., 1991), and therefore, the soil suction depends on the soil moisture condition (and its variation).

It is important to note that in general, within the proposed framework, foundations with adequate stiffness and strength (e.g. shallow waffle rafts designed in accordance with Appendix F of AS 2870:2011) may be designed for any class of expansivity.

DISCUSSION

Differential heave, and settlement, is the major concern relating to 'expansive soils' and can be caused by nonuniform changes in soil moisture and variations in thickness and composition of the expansive soil.

Non-uniform moisture changes below a foundation system/dwelling can occur from local concentrations of water from a number of sources including:

- surface ponding
- broken water and sewer lines
- leaky faucets, defective rain gutters and downspouts
- transpiration of moisture from nearby trees
- diffusion of moisture away from heat sources

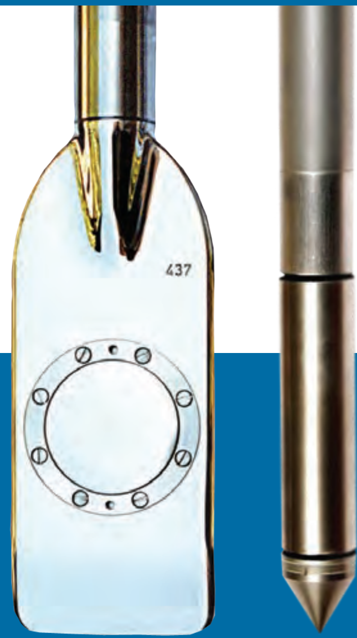
Managing the water below the foundation/backfill and subgrade is considered as the most critical factor to control the heave potential of an expansive soil.

Essentially treatment of expansive soils can be grouped under two categories:

A. Soil Stabilisation including:
   - removal/replacement
   - remould and compact
   - pre-wetting
   - chemical/cement stabilisation

IN SITU TESTING: CPT - Seismic - DMT

GROUND INVESTIGATION



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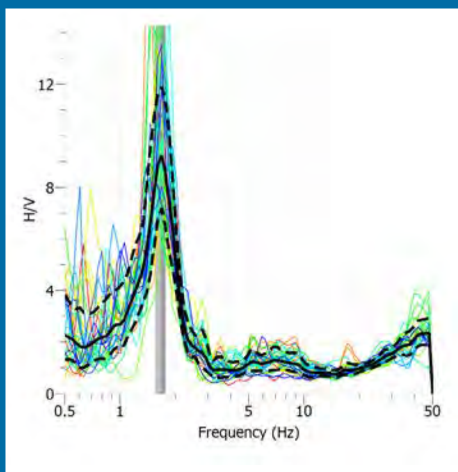

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B. Water content control methods including:
- horizontal and/or vertical barriers
- electrochemical soil treatment
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Partial replacement of expansive soils with granular fill is a widely implemented solution when moderately expansive soils of low thickness are present at the surface of the subsoil foundation. It has been recently observed that a replacement depth limited to 500mm is being specified.

In particular, the expansive soils can be removed and replaced by:

- non-expansive, impervious, properly compacted backfill (which also extents past the foundation laterally);
- non-expansive pervious (granular) backfill provided:
  - i. either equipped with drains to carry off infiltrated water, with the addition of an impervious moisture barrier(s) depending on the site-specific details;
  - ii. or, the depth of replacement is sufficiently thick for the resulting bearing pressure at the top of the underlying expansive soil to be greater than the swelling pressure.

Chen (1975) states from experience that if the subsoil consists of more than 1.5m (i.e. 5 feet) of granular soils between the base of the foundation and the top of a highly expansive subgrade layer, there is a lower likelihood of foundation movement. Moreover, he recommended a minimum thickness range between 900 to 1500mm below the foundation depth and the top of expansive soils for the backfill.

Providing a granular backfill below and around the foundation is considered an effective technique to mitigate detrimental effects of swelling on the foundations, but not without making sure the water content of the foundation subgrade (especially below the backfill) is controlled, and the replacement is sufficiently thick and extends a suitable distance beyond the foundation footprint.

Shallow replacement of expansive soils with compacted granular fill may be assumed as a measure to balance the net pressure at the base of the foundation (or footings), however, this is probably not the case for a 500mm deep replacement for the majority of sites, due to:

- The excavation and the granular nature of the backfill will potentially introduce more water to the subgrade;
- If the excavation is allowed to dry prior to placing the granular material it will propagate the ‘suction profile’ as shown in Figure 1 deeper creating an increased expansion potential;
- The corners of granular backfill will dry more than the centre, which in turn may cause differential heave / shrinkage.

CONCLUSION

It is the authors opinion that, although it may be 'reasonable' to consider partial replacement of the expansive soils a suitable means to mitigate structural damage, this is not necessarily the case, as (dependant on site-specific conditions), it may have the opposite effect by worsening the condition of the soil (behaviour) due to deeper infiltration (saturation / loss of suction) of the subgrade resulting in heave.

Site-specific geotechnical investigation with targeted laboratory testing to identify the soil characteristics (moisture content, Atterberg limits etc.) and swelling potential (e.g. ASTM D4546 One-dimensional Swell or Collapse of Soils), is considering a 'probable' best methodology for a detailed foundation design in expansive soils.

REFERENCES

AASHTO T 258-81 (2018) Standard Method of Test for Determining Expansive Soils, standard by American Association of State and Highway Transportation Officials

Adem and Vanapalli, Review of Methods for Predicting In-Situ Volume Change Movement of Expansive Soil Over Time, Journal of Rock Mechanics and Geotechnical Engineering, 2015, 73-86

ASTM D4546-14e1, Standard Test Methods for One-Dimensional Swell or Collapse of Soils, ASTM International, West Conshohocken, PA, 2014, www.astm.org

Asuri, S., Keshavamurthy, P. Expansive Soil Characterisation: an Appraisal. INAEL 1, 29–33 (2016). https://doi.org/10.1007/s41403-016-0001-9

Australian Standard AS 2870:2011, Residential slabs and footings

Chen, F.H. Foundations on Expansive Soils, Elsevier Scientific, 1975

Ice Manuals, Chapter 5 – Expansive Soils

Jones LD, Jefferson I. Expansive soils. In: Burland J, editor. ICE manual of geotech-nical engineering, Vol.1: geotechnical engineering principles, problematic soilsand site investigation. London, UK: ICE Publishing; 2012.

Jury W.A., Gardner W.R. & Gardner W.H. (1991) Soil Physics. John Wiley & Sons, Inc, New York

Mitchell, P. W. The Concepts Defining the Rate of Swell of Expansive Soils. 4th International Conference on Expansive Soils, 1980 Denver. 106-116

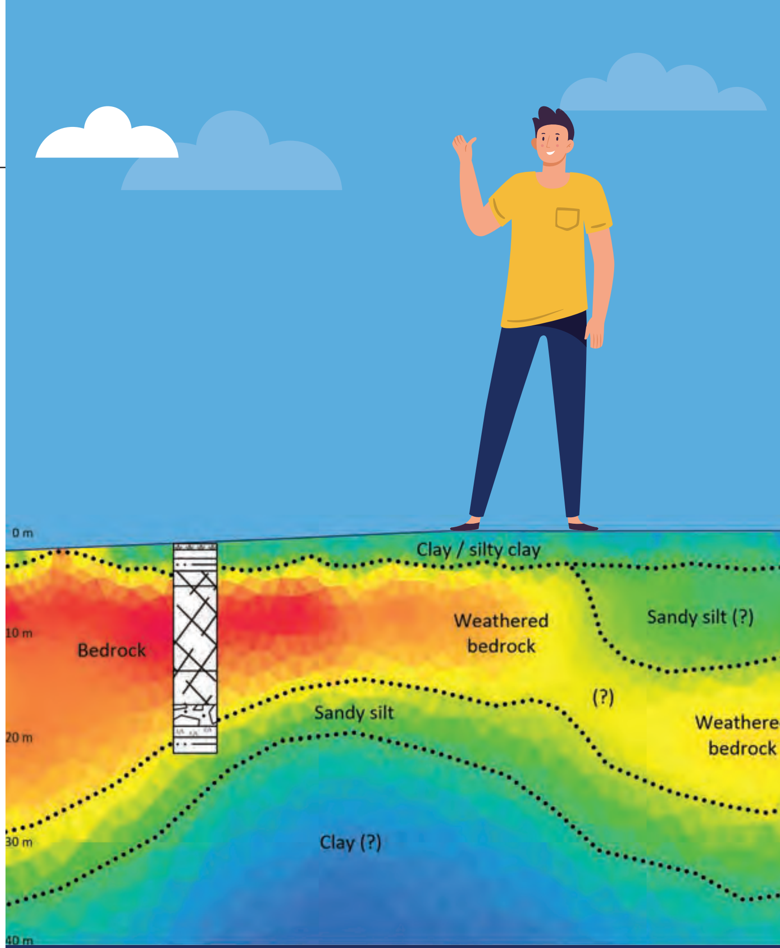
NHBC Standards online, Foundations (shrinkable soils), Section 4.5-D5, http://nhbccampaigns.co.uk/landingpages/techzone/previous_versions/2011/Part4/section2/default.htm#D5

Nelson, J.D., Miller, D.J. (1992). Expansive soils: problems and practice in foundation and pavement engineering. John Wiley & Sons, inc. New York, 1992

New Zealand Standard NZS 3604:2011, Timber-framed buildings

Richards BG. Measurement of the free energy of soil moisture by the psychrometric technique using thermistors. In: Aitchison GD, editor. Proceedings of the symposium on moisture equilibria and moisture changes in soils beneath covered areas. Sydney, Australia: Butterworths; 1965. p. 39e46.

Rogers et al. The Shrink Swell Test: A Critical Analysis, NZ Geomechanics News, June 2020, Issue 99

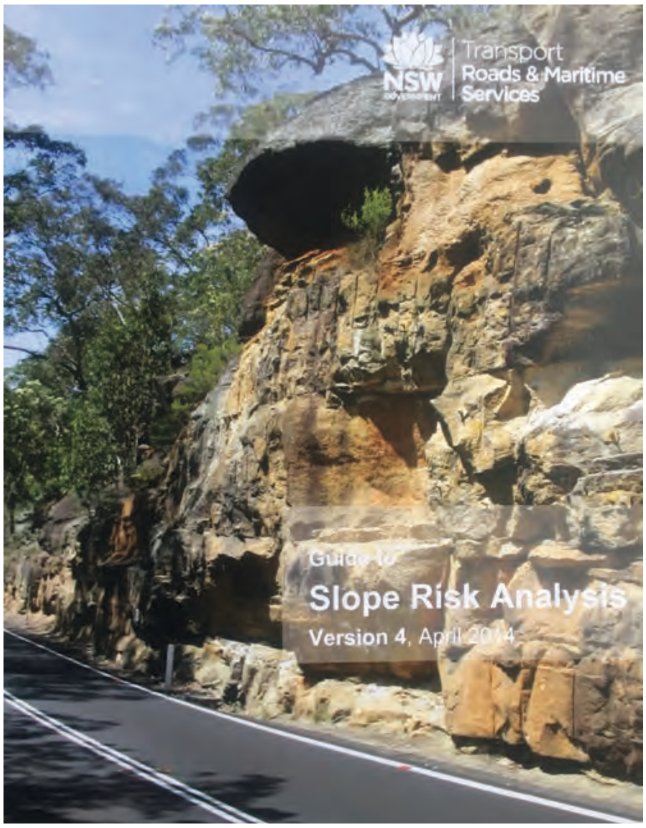


What's On at Waka Kotahi

Stuart Finlan
Lead Advisor Geotechnical

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

Stuart Finlan



SLOPE ASSESSED RISK LEVEL COURSE(S)

The Waka Kotahi/NZTA ARL slope assessment courses previously detailed in the June 2020 *Geomechanics News*, have been delayed by Covid-19 due to the international travel restrictions applied by the New Zealand Government affecting the Australian trainers. Currently a revised date of February 2021 is being worked on with two courses running centred in the North Island and a further course (or two) in October/November 2021 likely in the South Island. Those who were accepted for the first course will retain first right of refusal for that course with the additional course to provided to enable further participation.

Interest in the courses has been significant and clearly indicates an ongoing need to run the courses once they are established.

The Waka Kotahi country amendment, which will details the parts of the NSW RMS approach that will be adopted together with a number of modifications to align better to New Zealand conditions, nears completion and will form part of the course documentation.

Upon successful completion of the course, participants will be accredited to undertake slope assessments for Waka Kotahi. Accreditation will be valid for five years at which time evidence that slope assessments have been undertaken will be required and, subject to satisfactory demonstration, accreditation will be extended for a further five years.

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

FIRST ROCKFALL CANOPY IN NEW ZEALAND

Kaikoura has seen many rockfall protection systems being constructed as part of the NCTIR works; from mesh's to bunds and fences.

Risks at a couple of sites set the project team and Waka Kotahi a challenge that, thanks to the NCTIR team and suppliers, will see a unique solution to New Zealand: the FIRST rockfall canopy in New Zealand. Significantly cheaper than any other option to mitigate the risks it has still taken some considerable effort to bring the project to fruition. Could well be the answer to issues in many of our other regions! Look out for it on your drive south of Kaikoura over the Christmas holidays.

Gavin John Alexander

NZGS Life Member

GAVIN ALEXANDER PASSED away on 19th November 2020 after a battle with cancer. A highly respected engineer and trail motorcycle enthusiast, Gavin has left a significant legacy through his work with the New Zealand Geotechnical Society, Beca, and in his personal life.

Engineering is fundamentally about finding solutions that improve our communities, which requires critical thinking, clear communications, and an understanding of people. Not all engineers are well rounded enough to be able to work with a wide range of stakeholders and apply their technical skills in a way that is readily understood and accepted. Gavin was one of these rare few.

NZGS

Gavin joined the NZGS National Management Committee in 2011, becoming Vice-Chair and Treasurer in the same year under the Chairmanship of David Burns. This period immediately following the Canterbury earthquakes was a challenging time for everyone, including the NZGS Management Committee as it was a time of refreshed scrutiny on professional performance, and also of working together to rebuild a stronger Christchurch and more resilient New Zealand.

With the publication of the Royal Commission in November 2012 came demands for improvements in regulations and increased professional guidance. Following his election to Chair in 2013 Gavin was the main driver behind NZGS efforts to respond to these demands. In his endeavour to ensure that geotechnical perspectives were properly represented to government he put extensive effort in to responding to MBIE's consultation requests, and devoted much time to liaising with counterparts in MBIE and sister technical societies such as SESOC and NZSEE. His representation of NZGS in this respect has been an inspiration to others.

During Gavin's tenure as Chair, the NZGS was awarded the Outstanding Member Society of the International Society for Soil Mechanics and Geotechnical Engineering in recognition of the impact that the society was having under his stewardship.

Gavin was an enthusiastic champion of the development of the NZGS/MBIE Earthquake Geotechnical Engineering Practice Modules which have played a critical role in establishing the importance of a consistent and rigorous approach to geotechnical earthquake engineering in New Zealand. He managed their delivery and was heavily involved in the strategic direction, resulting in a very significant improvement in performance across the geotechnical industry.

Gavin also spent a very significant amount of time organising professional events which brought international best practice to New Zealand. These included four very successful conferences:

Gavin receiving his life membership of NZGS at his home in Riverhead.

Gavin greeting sponsors at the 20th NZGS Symposium, where he led the technical programme.

- 19th NZGS Symposium, Hanging by a thread. Lifelines, infrastructure and natural disasters (Queenstown, 2013)
- 12th Australia New Zealand Conference on Geomechanics, Changing the face of the earth (Wellington, 2015)
- 6th International Conference on Earthquake Geotechnical Engineering (Christchurch, 2015), and
- 20th NZGS Symposium, What In Earth Is Going On. Balancing Risk, Reward, Regulation and Reality (Napier, 2017)

In 2017 Gavin was appointed to the role of Australasian Vice-president of the International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE), representing the region in the world's leading geotechnical professional society until his untimely illness. The role required his attendance at meetings around the world and he became a well-known figure in the international arena.

During his time on the NZGS management committee Gavin focussed on making sure that it was an active society that was sought out for and presents views on current events, provides industry guidance and supports the education and growth of its members. Through the period of Gavin's governance the Society grew hugely in its status and contribution to society.

EDUCATION AND EARLY CAREER

Gavin's early start at engineering was as an architectural student and then a land surveyor, but his true calling of geotechnical engineering was already beckoning. As an ardent trailbike / motocross rider he was a founding member of the Dead Toad Racing Team, a regularly arranged high shear rate testing of loose sand and liquefaction trails at Woodhill. It has been suggested that this became core training on how to assess and plan the geotechnical aspects of transport projects.

Forsaking a career in architecture he obtained a New Zealand Certificate in Land Surveying before expanding his interest in engineering materials as a surveyor for the New Zealand Forest Service in Rotorua in 1979. There he was lucky enough to meet Fiona, the love of his life, who just two months later, agreed to become his wife. His steady progression towards engineering saw them move to Tauranga where Gavin worked as a Technician Surveyor with Shrimpton and Lipinski before moving to Auckland to join Babbage in 1981 in a similar role and commence his New Zealand Certificate in Engineering (NZCE) studies. Working for Babbage he was co-located with their geotechnical engineering team and recognised his true calling. While studying to obtain his NZCE he worked fulltime as a field technician learning the practical difficulties of soil logging, testing and the communication of field observations in a readily digestible and demonstrably verifiable way. He was hooked on geotechnical engineering and completed his training at the University of Auckland School of Engineering .

Flying through the engineering course at Auckland with an excellent degree in 1985 he took on the role

Gavin mingling with guests at the 12th Australia New Zealand Conference on Geomechanics while he was chair of the NZGS.

Gavin was a keen trail bike rider

of Engineer's Representative for Babbage based in Taumarunui to work on two sections of the North Island Main Trunk electrification scheme. More realignment than tunnelling was required but that was perhaps his first exposure to the interaction of water, pumiceous soil and repetitive loading - albeit largely due to rail transport rather than seismic.

The lure of overseas travel then saw him head to Australia and on to the UK in 1987 where he worked for Arup Geotechnics at Canary Wharf on the Canada Water project encountering the Thames gravels, the Woolwich and Reading Beds and Thanet Sands. He worked on large scale dewatering projects and deep base grouted piles while juggling excursions to Europe. During his time in Sydney with Arup he secured a Peter Dunican Scholarship (the first time that had been awarded outside the UK) to study for his Master's Degree at Imperial College. There he pursued his interest in geotechnical earthquake engineering, staying on in the UK to work for Arup in London, Edinburgh and Newcastle before returning to New Zealand to join Beca in 1993 as a senior geotechnical engineer.

BECA

As a practice leader, Principal and Technical Fellow in Beca's Geotechnical Engineering group for more than 20 years, Gavin led single and multi-discipline teams on many large-scale, high profile and complex projects, both across New Zealand and offshore (Australia, Indonesia, Singapore, Malaysia, Papua New Guinea, Thailand, New Caledonia and elsewhere in the Pacific).

Gavin was always ready for new and interesting challenges, and in 2003 he took up the Christchurch based role of Materials Team leader for Project Aqua, a hydroelectric power project proposal for the Waitaki valley. The project was not to proceed, and so in 2004 he returned to his role in Auckland, but not before he had become a popular member of the wider group of international engineers who had relocated to New Zealand for the project, and who now remember him from across the globe.

Gavin could not keep away from a technical challenge; he had the roles of designer, technical reviewer, peer reviewer, Board member, on a wide range of projects. Career highlights included his work for Kedah Cement in Malaysia, a number of projects for PT Vale Indonesia (he mobilised his family to Soroako for 3 years) including upgrade of the slag dump; wharf extensions for the 36th America's Cup; Centre Port Wellington, Maungaharuru, Taihape, Waitahora and Awhitu windfarms, Kawerau and Te Mihi power stations, Britomart station reviews, North Shore Busway, Nicoll Highway Collapse, East West Link, Hamilton section of the Waikato Expressway, SH1 McKays to PekaPeka expressway, Puhoi to Warkworth Motorway, SCIRT (Stronger Christchurch Infrastructure Rebuild Team), Victoria Park Tunnel, SH20 Waterview Connection – to name just a few!

PRIVATE LIFE

Gavin has always been a keen motorcycle enthusiast and took part in the first New Zealand Enduros, described as a motocross "test of man and relatively unsophisticated machinery against terrain". Gavin demonstrated his skill at the 1975 Maramarua Forest and Riverhead Virgin Swamps Enduros, finishing in the top ten in both on his Suzuki TS400.

He was a kiwifruit farmer and loved to drive his ride-on between the rows and trim vines at the weekend.

He is survived by his beloved wife Fiona and his twin children, Jack (married to Rachel), and Harry (engaged to Cindy).

Gavin will be missed by many of us but he will also continue to be a role model both as a technical expert and a kind and gentle man who was thoroughly professional in everything he did.

PARAWEB MSE ABUTMENTS
TARAMAKAU RIVER, WESTLAND

Professor John Beauchamp BERRILL (Retired)

1941 - 9 Oct 2020

Passed away in Christchurch on Friday, October 9, 2020, in his 79th year.

JOHN RECEIVED HIS BE from the University of Canterbury in 1964. Subsequently he obtained a Masters degree from the University of Colorado working with Professors Larry Feeser and Hon Yim Ko and from there a PhD from Caltech where he studied under Prof Ronald Scott. For his PhD John worked on attenuation and directivity effects in strong ground motion as well as on the evaluation of site response as indicated in recorded ground motions.

On completion of his PhD John took an academic appointment at the University of Newcastle in 1975. In 1977 he moved to the University of Canterbury, Department of Civil Engineering.

John's research and professional activities have contributed to the NZ understanding of earthquake geotechnical engineering in a number of areas which were detailed in the Citation accompanying his Life Membership award by NZSEE in 2011.

In summary:

The CUSP seismograph, which was selected from a handful of international contenders for regional strong motion recording in the GeoNet Project, exploited the mass-produced accelerometers used to trigger automotive airbags. John subsequently devised the layout of a Canterbury Regional Network, which is now operated as an integral part of the GeoNet system, and installed the network before the 2010 Darfield Earthquake, thus ensuring that the Canterbury Earthquake Sequence became one of the best-recorded events in the world.

Some of John's work on retaining structures was funded by the Structures Committee of the former Road Research Committee of the National Roads Board. John contributed to the So ciety's Study Group on the Seismic Design of Bridges leading the discussion and reporting on design earthquake loading. He was also involved with the discussions on earth retaining structures.

An analytical study of the conditions under which fault rupture is diverted around buildings was then extended to the effect of ridges and hillsides in displacing the surface expression of faults, a result which has been adopted into common geological interpretation practice.

Perhaps the most significant contribution made by John relates to the prediction of liquefaction, in conjunction with Professor Rob Davis, his colleague from the University of Canterbury. Their work was based on the concept of the liquefaction being related to dissipation of energy in the soil. Parameters for the energy dissipation had to be linked with engineering seismology ideas about the spread of energy from the earthquake source. This thinking was developed at a time when the approach to liquefaction prediction was almost totally based on empirical observation and the use of the Standard Penetration Test results.

A further aspect of John and Robs liquefaction work was EQC funded research on the liquefaction risk in Christchurch – found to be significant, years before the 2010 Darfield Earthquake.

Part of his interest in liquefaction involved work searching for past evidence of liquefaction in NZ. Investigations of sites in the Buller region which were known to have liquefied during the Murchison and Inangahua earthquakes provided valuable data on NZ conditions as did his work on the Landing Road bridge near Whakatane after the Edgecumbe earthquake.

John will also be remembered for his untiring student mentoring work and his constant efforts to find financial support for students.

Peter George Fookes

(31 March 1933 - September 2020)

PETER FOOKES' CAREER began in the 1950s. When he began there was a loosely coined 'geology for engineers' but no formal career structure in engineering geology.

After graduating from Queen Mary College, London he entered the world of civil engineering as a young geologist. Coming under Professor Skempton's influence, he studied for an external PhD at Queen Mary College, London which led to a position as a lecturer at Imperial College in the developing engineering geology group.

From there he never looked back, helping with his commercial experience to build up the world's first MSc in Engineering Geology, before moving on to develop his consultancy in 1971.

Over the following years he was a pioneer in the application of geology to civil engineering and, using his initial chemistry background, in the influence of desert materials on concrete durability. He was affectionately called the "father of engineering geomorphology" as he was a great supporter of the use of geomorphology on engineering projects.

Alongside his busy commercial consultancy, he never lost his links to academia continuing to lecture, lead field courses and initiate ground-breaking research while playing a leading role in the Engineering Group of the Geological Society.

Fookes was a prolific writer publishing some 200 papers and 10 books, still writing at the time of his death. Many of these have been seminal works. His writing was hallmarked by its pragmatic approach allied to the use of easy to understand graphics.

He helped to set up and was a chair of the Engineering Group of the Geological Society and chaired many of their working parties. His contributions were over the full range of geological application and resulted in awards from many disciplines, including Glossop Medal (engineering geology), Honorary Fellow of the Royal Geographical Society (engineering geomorphology), Fellow of the Royal Academy of Engineering (civil engineering) and Honorary Fellow of the Institute of Concrete Technology (concrete).

He also held several visiting professorships, was awarded Doctor of Science (Engineering) at Imperial College and was a recipient of the William Smith Medal of the Geological Society.

If anything sums up Peter's approach to the world of geology and engineering it is the ground model, so logically bringing the two disciplines together. His approach was to characterise a site or infrastructure route by considering its historical development from the environment of deposition of the original rocks and soils, through global tectonic changes to the geomorphological processes which have most recently shaped the near surface landscape.

This philosophy was brought together in the first Glossop Lecture he presented in 1997, the resulting publication virtually a manual in its own right with many illustrations becoming standard references for widespread use.

The economic expansion of the Middle East from the 1970s and the development that accompanied it provided a huge workload for European and American engineers that had little previous experience of engineering in hot dry climates.

Peter identified inadequacies in the aggregates used in the concrete, pioneered the concept of salt attack resulting from high rates of evaporation and was at the forefront of developing guidelines to good practice now embodied in regional standards.

The five articles comprising Concrete in the Middle East, published in Concrete in 1977 are still widely referenced.

One of his last papers was "The Engineering Geology of Concrete in Hot Dry Lands" was published in 2019.

This obituary is summarised from one published by Ground Engineering in October 2020 that was written by John Charman. A much more detailed account of his career was published in the Quarterly Journal of Engineering Geology and Hydrogeology in 2008.

International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE)

Vice President - Regional Report for Australasia

Philip Robins

Philip is a Technical Director – Geotechnical with Beca based in Wellington, with over 20 years' experience specialising in geotechnical analysis and design. He has been involved in the design and construction of major infrastructure projects in New Zealand, California, Hong Kong and Southern Africa. Philip was Chair of the NZGS Management Committee from 2009 to 2010.

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.

See the ISSMGE website – http://www.issmge.org/ for full details of all ISSMGE activities.

1. INTRODUCTION

On behalf of the ISSMGE President and Board, I would like to congratulate the NZGS on the 100th edition of the *NZ Geomechanics News*. I have a copy of the first edition on my desk (dated November 1970)! It makes for entertaining reading and apparently was "A Newsletter of the N.Z. National Society for Soil Mechanics and Foundation Engineering". Our learned society has come a long way in 50 years and our Bulletin is now a high-quality publication which graces many desks and something most of us wait for eagerly every 6 months. Well done team.

Like most of us, I am looking forward to closing the door on the Year 2020. While the Covid-19 pandemic will be an ongoing 'challenge' next year I am confident that we are better prepared. The following is a summary of recent highlights of ISSMGE activities regionally and internationally.

2. 2021 TERZAGHI ORATION – NEWS FROM THE ISSMGE PRESIDENT CHARLES NG

"I am very pleased to announce that Professor Antonio Gens from the Universitat Politècnica de Catalunya in Spain has been selected as the 2021 Terzaghi Orator. Professor Gens was selected from a pool of 16 outstanding nominations including 5 former Rankine Lecturers. After reviewing the abstracts of proposed case histories including photographs submitted by 6 finalists and consultation among some distinguished peers in our Society, I had the privilege to select Professor Gens to be our next Orator although the decision was extremely difficult since we had so many outstanding candidates. I am absolutely confident that Professor Gens will deliver an excellent lecture in Sydney in 2021. Please join me in congratulating Professor Gens."

3. ICSMGE 2021 SYDNEY 2021

Preparations for 20th International Conference on Soil Mechanics and Geotechnical Engineering (ICSMGE) in Sydney continue, with co-chairs, John Carter and Graham Scholey along with Scientific Program Chair, Mark Jaksa and their teams putting in many hours of hard voluntary work. The Technical Program Committee has announced their State-of-the-Art Lectures, each one led by experts in their field. Find out more on the ICSMGE 2021 Website (https://www.icsmge2021.org/).

I am also pleased to announce that our own Dr Brendon Bradley has been selected as one of the Bright Spark Lecturers. He will be giving a keynote lecture at the ICSMGE in Sydney.

ICSMGE 2021 has accepted 1,230 abstracts for the Conference. Sponsorship and exhibition opportunities are also available and the ICSMGE 2021 will "provide an excellent opportunity to promote your company, to network with international experts, share your latest research and technologies, hear about the latest developments from opinion leaders in the field and build collaborations". Mark your calendars for a trip to Sydney on 12 to 17 September 2021.

For the 7iYGEC, ISSMGE Member

Societies are invited to nominate up to four of their young members to attend the Conference. The NZGS has received 12 applications for nomination. The NZGS Management Committee has selected the top 4 abstracts to upload to the conference portal. Only the top 2 abstracts are guaranteed to be accepted to the conference, with up to 2 additional places offered if the conference quota is not filled. The NZGS is offering full funding for up to 4 applicants should they be invited to the 7iYGEC to be held just before the ICSMGE 2021 in Sydney.

4. NEW YGP ROLES

At the NZGS Management Committee meeting on 22 September 2020 in Christchurch, the committee unanimously agreed to the creation of three new YGP roles (outside the committee, reporting though the YGP representative and the international society VPs) including an ISSMGE YGP rep. The NZGS Management Committee is preparing a role description for the committee's agreement, after which expressions of interest will be sought from all NZGS young members.

5. GEOTECHNICAL DESIGN CHALLENGE

The CAPG's "Are we over designing?" Survey is gaining momentum with various ISSMGE groups (YMPG, CAPG, Corporate Associates, Member Societies) hosting virtual design challenges around the world. The AGS is hosting an Australia and New Zealand Geotechnical Design Challenge as part of an international survey intended to assess the consistency of calculation models and design methods for a variety of geotechnical structures and, where possible, to compare the results with full-scale tests and reliability analyses. The survey includes ten common geotechnical design problems, such as shallow and deep foundations, slope stability and retaining walls, in clay and sand soils.

Harry Poulos gave a briefing regarding the design challenge on 14 October 2020. The briefing is now publicly available to watch here: https://australiangeomechanics.org/meetings/australia-and-new-zealand-geotechnical-design-challenge/.

In addition to providing valuable insight into the practice of design across various countries in the world, the survey is also a great opportunity to benchmark your companies approach to design of relatively simple geotechnical problems. No details of the company are required nor would these details, even if provided, be released.

Full details of how to undertake and submit the survey can be found both on the ISSMGE website: https://www.issmge.org/news/are-we-overdesigning-a-survey-of-international-practice and in Geoworld (https://www.mygeoworld.com/file/139638/capg-overdesign-survey).

6. ISSMGE ONLINE LIBRARY

The ISSMGE Online library (https://www.issmge.org/publications/online-library) is in continuous development. Hugo Acosta-Martinez, Past Chair of the Australian Geomechanics Society continues to drive the upload of ANZ Conference Proceedings in this open access database. This month, Hugo uploaded 127 papers from the 9th ANZ Conference on Geomechanics (Auckland, 2004), which means that the full collection of papers from that ANZ event series is available online.

7. ISSMGE TIME CAPSULE

As a part of our continuing developments of the ISSMGE, a new project, ISSMGE Virtual Time Capsule, has been launched. The coming decades, encompassing the centenary of the ISSMGE (2036), are predicted to bring unprecedented changes in all walks of life, through the underlying shift in world view and attitudes towards digital advances and sustainability considerations, accelerated by the necessary changes in practice and expectations that COVID 19 has brought – the new normal. Our opportunity is here and now to bring forward the creation of an entirely Virtual Time Capsule that brings together and provides a common heritage to all geotechnical engineers.

The Virtual Time Capsule will initially be hosted on the ISSMGE web site, enabling interactive communications with the geotechnical community in the lead-up to the grand release, at the 20th ICSMGE in Sydney. We anticipate material will be progressively added on the ISSMGE website from early - mid 2021, to build up a ground swell of interest. For further information, or to offer your support, please contact Sukumar Pathmanandavel sukumar.pathmanandavel@gmail.com.

8. ISSMGE FOUNDATION

The ISSMGE Foundation was created to aid individual ISSMGE members throughout the world to enhance their geotechnical engineering knowledge and practice by providing financial support for participation in technical and professional activities approved by the ISSMGE Foundation. There is no prescriptive list of admissible events and all reasonable applications will be considered, considering the relevance to ISSMGE and active rather than passive participation by the applicant.

The next deadline for receipt of applications for awards from the ISSMGE Foundation is the 31st January 2021. For further information on the ISSMGE Foundation: https://www.issmge.org/issmge-foundation/application-form.

9. ISSMGE BOARD MEETING

As the COVID-19 pandemic continues to ravage most of the northern hemisphere, the next ISSMGE Board Meeting will be held online in March 2021.

Best regards,

Philip Robins
ISSMGE Vice President for Australasia
philip.robins@beca.com

International Society for Rock Mechanics and Rock Engineering (ISRM)

Report for New Zealand – December 2020

Paul Horrey

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

THE REPORT ADDRESSES ISRM matters since the June 2020 edition of Geomechanics News.

BOARD AND COUNCIL MEETINGS:

The latest ISRM Council meeting took place as a videoconference on 28 October. It was preceded by the Board meetings on 20, 21 and 22 October.

53 out of a total of 61 National Groups were represented at the online Council meeting. Running a large multinational meeting like this online is a significant challenge and it smooth running is a credit to both organisers and participants, notwithstanding the NZ country representative having to join in the middle of the night.

MEMBERSHIP UPDATE:

The ISRM has achieved a record of 8620 individual members and 184 corporate members. This represents an increase of 2% in the number of individual members since the last year. The National Group of Mongolia joined the ISRM in 2020.

ISRM AWARDS:

The following awards were announced at the Council meeting:

ROCHA MEDAL 2021.

Winner: Yasuhiro Yokota, Japan, for the thesis "Experimental and computational study on rock bolt modelling and its application on a new type of energy-absorbing rock bolt", presented at Nanyang Technological University, Singapore.
Runner-up: Wenzhuo Cao, China, for the thesis "Monitoring and modelling of microseismicity associated with rock burst and gas outburst hazards in coal mines",
Runner-up: Bing Li, Canada, for the thesis "Microseismic and real-time imaging of fractures and microfractures in Barre granite and opalinus clayshale",

JOHN HUDSON ROCK ENGINEERING AWARD 2020.

The ISRM Corporate member 3GSM GmbH, was selected for the achievement "Photogrammetric 3D models from drone imagery for improved rock mass characterization".

SCIENCE ACHIEVEMENT AWARD 2020.

This award, approved in 2019 to recognizes outstanding contributions to science and technology in the field of rock mechanics and rock engineering, was conferred to Prof. Jianping Zuo of China.

A new Young Engineers Award has been created to recognize achievements in rock engineering practice by young engineers, that will be implemented by the ISRM in 2021.

OTHER COUNCIL/ BOARD MATTERS:

- The Board approved the initiative to broaden the number and type of courses available on the website.
- Printing of the proceedings of ISRM sponsored conferences was discussed, namely concerning the possibility to print for personal use, the question of copyright and the access to the publications.
- The conference AusRock2022, in Melbourne, was approved as the 2022 ISRM Regional Symposium for Australasia.

INTERNATIONAL SYMPOSIA 2021 AND 2022

- The 2021 ISRM International Symposium will be held in Torino, Italy 20-25 September 2021.
- The National Groups of Finland and Paraguay presented proposals to host the 2022 International Symposium of the ISRM, in Helsinki and Asunción, respectively. The Council, by secret ballot, selected the application from Paraguay, which is also the IX Latin American Congress on Rock Mechanics, Rock Testing and Site Characterization.

A more detailed summary of the matters covered in the October Board and Council meetings may be found on the ISRM website https://www.isrm.net

COVID19 EFFECTS

The ongoing effects of the pandemic continue to result in a number of changes to both timing and format of upcoming events. Please refer to the ISRM website for updated information.

ISRM ON-LINE LECTURES

Two on-line lectures have been given since the last report:

- September 2020 – *Damage and Time-Dependent Behaviour of Rocks in Underground Construction*, Prof. Frederic Pellet
- June 2020 – *Contact Theory and Algorithms for Discontinuous Computations* Dr. Gen-Hua Shi

These have been recorded and are available on the ISRM website

COMMUNICATION

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

There is also Linkedin, Twitter or RSS access.

The latest ISRM newsletter continues to appear quarterly and is sent directly to all ISRM affiliated members. Issues for 2020 so far include:

- Newsletter No. 51 - September 2020 - Latest Issue
- Newsletter No. 50 – June 2020
- Newsletter No. 49 - March 2020

The ISRM Digital Library, which was launched in October 2010, is intended to make rock mechanics material available to the rock mechanics community, in particular papers published from ISRM Congresses and sponsored Symposia. It is part of OnePetro (https://www.onepetro.org), a large online library managed by the Society of Petroleum Engineers.

It includes proceedings from 56 ISRM sponsored conferences and ISRM individual members are allowed to download, at no cost, up to 100 papers per year from the ISRM conferences. To use this facility ISRM members must register each year, with further details given on the ISRM website (https://www.isrm.net/gca/?id=992).

OTHER ITEMS

On 26 March 2020, Prof. Manuel Romana Ruiz passed away. Throughout his long professional career, Professor Romana became one of the greatest exponents of Rock Mechanics both in Spain and internationally. He developed the so-called Slope Mass Rating (SMR), one of the most used geomechanical classifications focusing slope stability. He was elected president of the Spanish National Group of the ISRM (SEMR) in 1980 and was in office for more than 20 years. The ISRM and the rock mechanics community deeply regret his loss.

Paul Horrey
ISRM NZGS Liaison
12 November 2020

IAEG Report
December 2020

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.

THERE HAS BEEN very little activity for IAEG since last reporting. The programme of international conferences and meetings has been severely disrupted by Covid-19. The status of current IAEG meetings and conferences is available on the IAEG website and updated in the weekly connector newsletter. Uncertainty on the programme of events is likely or remain for at least Q1/Q2 of 2021.

National Groups have reported limited activity in 2021 and most counties have been equally disrupted by Covid-19. Member activity has been dependent on local and regional group activities. There has been some activity for new national groups for IAEG in South America, but this will not be verified until the next council meeting – deferred to 2021.

A new YEG Chair has been appointed - Stratis Karantanellis, from Greece. The YEG have started a programme of providing structured and predefined questions interviews to our YEG and IAEG members and then sharing their answers via IAGE / YEG social media and IAEG website. The aim is for Engineering Geologists to get to know other members, create networks, and give them the opportunity to gain profile for the work they are doing in this new world order.

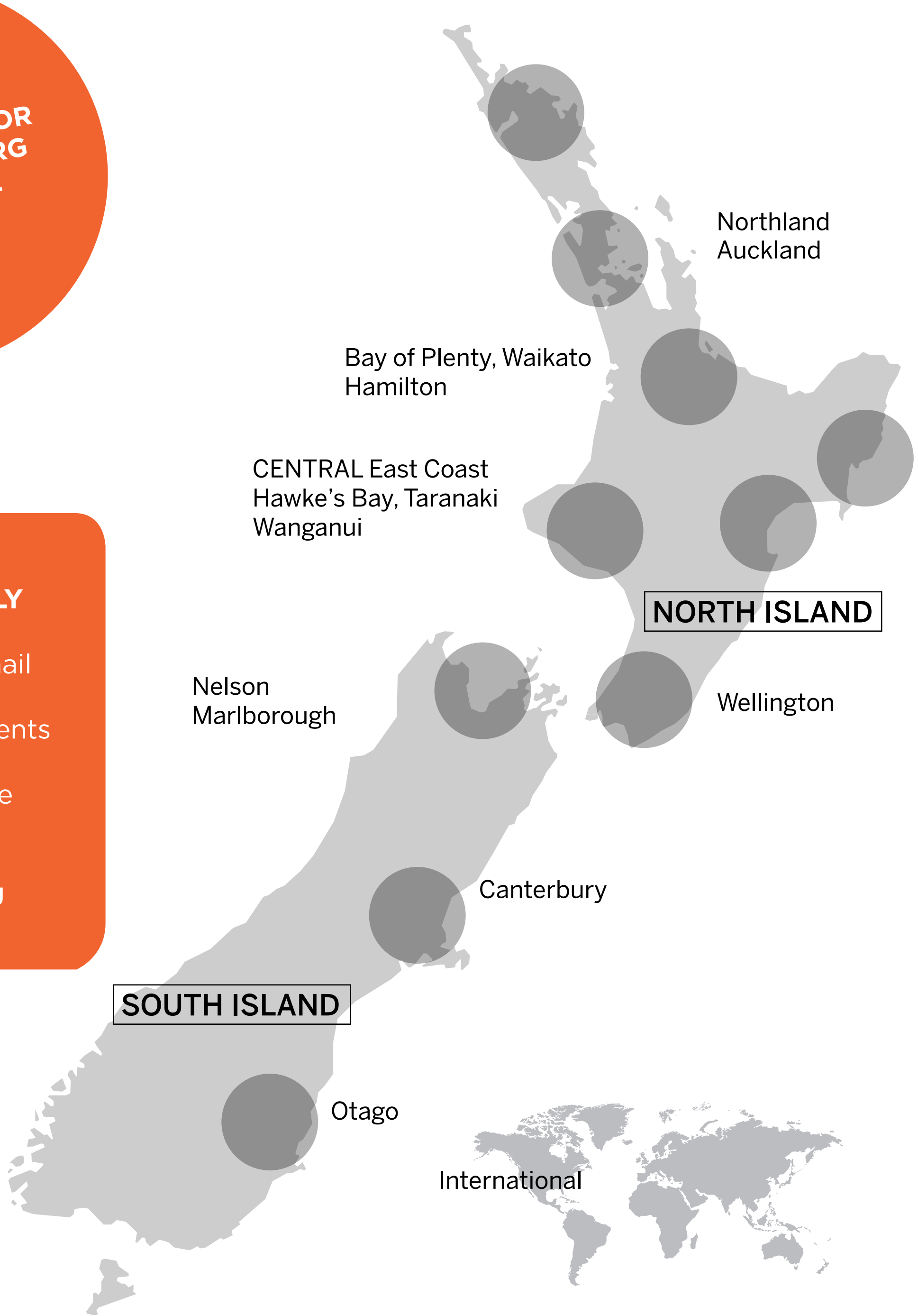
The review of the rules and statutes of IAEG remains on hold pending an Executive and Council meeting.

Doug Johnson
IAEG Australiasian Vice President
DJohnson@tonkintaylor.co.nz

Branch Reports

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

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



Auckland Branch

The Auckland Branch have had a unique latter half of 2020, electing to hold presentations via the online format with the support of Engineering New Zealand. Although this format does not present the opportunity for local Auckland Branch members to network in person, it has presented other benefits. It has provided the opportunity to involve geographically distant speakers in each presentation and to host a much larger audience, gaining a wider reach (often well beyond the Auckland Branch).

Presentations occurring since the last branch report include information and discussion around the Redi-Rock Retaining Wall system, which outlined both technical aspects of the system and local examples where the product has been used to good effect. More recently, Morgan NeSmith from Berkel & Company (Atlanta, USA) presented on 'Drilled Displacement Piles/Elements for Liquefaction Mitigation', which was thought provoking for an audience who design in conditions which often include seismicity and associated liquefaction hazards.

At the time of writing, the Branch Coordinators are busily organising the Auckland Branch Christmas Mini-Symposium which will be followed by a dinner, providing a long-awaited opportunity for an 'in-person' event. This event is one of the highlights on the Calendar each year for local geotechnical professionals and the team are hoping this is another afternoon of high-quality presentations, discussion and networking.

The Auckland Branch Coordinator team will unfortunately be farewelling Chris Wright, who has elected to step away from the coordinator role after more than 3 years of service. Ben Francis and Jay Doddaballapur would like to express their gratitude to Chris on behalf of NZGS and the Branch. Chris has had significant input over the time he has been working in this role and his presence will be missed. We wish Chris all the best with his geotechnical endeavours and look forward to seeing him at the NZGS events going forward.

Ben Francis, Chris Wright & Jay Doddaballapur

Waikato Branch

On 5th August David Hepburn from Duracrete Products presented an introduction to the Redi-Rock retaining system which is now being supplied in NZ for land development and infrastructure applications. This included an introduction to the suite of design resources and free software analysis for geotechnical engineers. Some case studies of local and recent projects in NZ were featured, with the challenges of the applications and the variety of solutions for them highlighted. Feedback from clients, contractors and engineers who have used have been very positive.

FUTURE EVENTS

The next Waikato Event is the CFA Piling BPC & Hawkins Site Visit & Technical Presentation at the Project Hauata site. We have several other events in planning and will be notifying branch members as soon as we have details and dates confirmed.

Bay of Plenty Branch

Along with many other organisations and branches of NZGS, the Tauranga branch has had a quiet year due to the various lockdowns. We have been lucky recently with two presentations from Phaedra Upton (GNS) including her Hochstetter Lecture (2020). Although a smaller group than normal attended the lectures, it was great to see a number of familiar faces eager to be back networking and listening to some fascinating research. Our next presentation on 8 December will be from Marlena Prentice (University of Waikato PhD Candidate) and focus on the ignimbrite flows around the Bay. We're looking forward to getting 2021 underway and have a few people lined up to present. We would like to acknowledge the support from our local ENZ branch which has been invaluable.

Hawkes Bay Branch

We met on 4th August for two presentations by GNS Science. HBRC kindly hosted the group in the council chambers.

Julie Lee presented on the status of a long running project to update the geological map for the Napier-Hastings urban areas. The use of 3D software and LiDAR data allowed them to improve differentiation of Holocene units that form the Heretaunga Plains. Further maps are scheduled to be delivered over coming months.

Chris Massey described an earthquake-induced landslide (EIL) forecasting tool the GNS have developed, initially using the 2016 Kaikoura Earthquake landslide dataset, but subsequently working in other EIL datasets (e.g. 1929 Murchison and 1968 Inangahua). The tool has been applied in Hawke's Bay to understand the local rockfall hazard to tsunami evacuation paths on Bluff Hill, and where the regional landslide hazards are so this can be assessed against existing and planned asset development.

Wellington Branch

The highlight of this quarter for the Wellington branch is definitely the joint presentation with SESOC Emerging Structural Engineers on collaboration between Structural and Geotechnical Engineers. The event presenters were Alistair Cattanach, Director at Dunning Thornton Consultants and Stuart Palmer, Technical Director at Tonkin + Taylor. The event was initially scheduled for late March, then postponed to late August and eventually became an online-only event to accommodate restrictions imposed due to COVID pandemic. The online event was a success with 448 registered online-attendants and quality post-presentation discussion. Thanks SESOC and Geotechnics for kindly sponsor the online event. And thanks to Engineering New Zealand for the online conference coordination and support.

With the end of 2020 fast approaching, the Wellington branch has decided to postpone the 2nd Wellington YGP Symposium till mid-2021. More details will be sent out in NZGS newsletters in early 2021.

We would like to thank Jerry Spinks (Jacobs) for over three years of service to the Wellington branch. Jerry has stepped down as a branch coordinator earlier this year.

Please get in touch if you have any presentation or event ideas. As always we are looking forward to hearing from you.

Nelson Branch

The Nelson NZGS group has recently started presentations and organised events back up since the lockdown. In the last couple of weeks we have had the following:

- A combined event with the local Engineering NZ and NZIA branch to bring 4 local speakers to give presentations about 'What are the options for the top of the south as sea levels rise?'. The event was well attended by many disciplines from around the region and was a great networking and informative event.
- We had a visit from the Geo Fabrics team who along with Eric Ewe gave an insightful presentation about Passive Rockfall Protection Structures with NZ case studies.

We are excited for the next couple of months with events in conjugation with Engineering New Zealand coming up including a site visit to the SH60 Takaka Hill repairs site and the end of year function, so watch out for the emails to register.

Lastly we have sadly said good bye to a long standing NZGS rep Rebecca Ryder (formally from Beca), we would like to wish her well with her new adventures across the ditch, you will be missed within the region!

Christchurch Branch

The Christchurch branch had a lot of action in late 2019/2020, with evening talks scheduled for every month - largely thanks to Mark Stringer's (UoC) ability to secure top quality lecturers. Talks included Jordi Corominas, Jonathon Stewart, and Armin Stuedlein. Unfortunately a planned lecture by Tom O'Rourke was unable to go ahead before Covid-19 put a halt to things.

The local branch reps have some new faces. Sam Burgess came on board in late 2019 and Natalie Hyland passed the baton over to Sarah Bastin in mid-2020 before welcoming a new addition in her family, baby Lyra. Thanks to Natalie for the hard work she put into this role and congratulations on the new arrival!

Like most of the country, we hibernated for Covid-19 and kicked back off in October with a Building Compliance Technical Forum presented by the Christchurch City Council Engineering Services Team at CCC. This was a really good session, and a great chance for the local industry to get some detail on how the Engineering Services Team interprets the relevant standards and codes, and what they are looking for when reviewing consent applications. It was great to see some familiar faces again.

As we look ahead to 2021, we are hoping for a more settled year with (hopefully) more lectures in the pipeline. If you are local and would like to give a presentation, or have an idea for one, we are looking for speakers and would love to hear from you.

Sam, Duncan, and Sarah

Otago Branch

The Dunedin branch had a great recent event hosted by TerraMDC, which was very well attended. As with everything else in life at the moment, Covid-19 has put a dampener on branch activities this year (including the Symposium!). The branch coordinators are planning a couple events for early next year, so stay tuned after the holiday break. And, of course, we're all looking forward to seeing our peers and colleagues from around the country at the Symposium here in Dunedin in March!

Cheers, Matt and Nima

Branch Coordinators

NORTHLAND

PHILIP COOK

I am a Chartered Professional Engineer. I have an interest in risk assessment, landslides, Northland Allochthon geology, liquefaction, and seismic assessment for earthquake resistant foundations, foundation settlement.

Look forward to improving the geotechnical features of soils in Northland. Enjoy the coastal lifestyle of Northland

phil@coco.co.nz

AUCKLAND

JAY DODDABALLAPUR

Jay is a Chartered Principal Geotechnical Engineer with an MSc in Geotechnical Engineering from the University of Glasgow. He has worked in the UK, Middle East and New Zealand on buildings, infrastructure and marine projects. He has experience in design and management of temporary and permanent works with a particular focus on providing value engineered, sustainable and buildable solutions.

jay.doddaballapur@aecom.com

BEN FRANCIS

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

BFrancis@tonkintaylor.co.nz

CHRISTOPHER WRIGHT

Chris is a geotechnical engineer at Riley Consultants Ltd. He has bachelors degree 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 ENGEO 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.

KDeGraaf@engeo.co.nz

HAWKE'S BAY

TOM GRACE

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

tgrace@rdcl.co.nz

WELLINGTON

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@wsp.com

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

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**

SAFIA MONIZ

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

safia.moniz@holmesconsulting.co.nz

NELSON

REBECCA MCMAHON

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

Rebecca.McMahon@beca.com

CANTERBURY

Sarah Barett

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

sarah.barrett@beca.com

SAM BURGESS

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

SBurgess@tonkintaylor.co.nz

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.com

MATT FITZMAURICE

Matt is an engineering geologist in GHD's Dunedin office. He has 9 years' experience working in both the Western Australian mining industry (predominantly underground), and in New Zealand consultancies. Matt's areas of interest typically revolve around rock mechanics, and he loves to get out of the office and walk around the hills looking at rocks.

matthew.fitzmaurice@ghd.com

QUEENSTOWN

PAUL JAQUIN

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

Paul.Jaquin@wsp-opus.co.nz

NEW ZEALAND GEOTECHNICAL SOCIETY INC

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

The aims of the Society are:

a) To advance the education and application of soil mechanics, rock mechanics and engineering geology among engineers and scientists.
b) To advance the practice and application of these disciplines in engineering.
c) To implement the statutes of the respective international societies in so far as they are applicable in New Zealand.
d) To ensure that the learning achieved through the above objectives is passed on to the public as is appropriate.

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

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

WELCOME TO THE last issue of 2020, our 100th edition. It has been a challenging year with Covid-19 and we have lost a dear friend and colleague, our ex Chair and ISSMGE VP Gavin Alexander who passed away in November. I believe most of us are looking forward to waving goodbye to 2020 and making 2021 the year of positivity and success. A big thank you to the advertisers that continue to support the *NZ Geomechanics News*, our branch reps that have been inventive in this year's exceptional circumstances to bring you information and online webinars. I wish you all a very happy, safe summer break and look forward to assisting you in 2021.

Teresa Roetman

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

Management Committee 2019-2020

POSITION	NAME	EMAIL
Chair	Ross Roberts	chair@nzgs.org
Vice-Chair & Treasurer	Eleni Gkeli	treasurer@nzgs.org
Immediate Past Chair	Tony Fairclough	TFairclough@tonkin.co.nz
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Elected Member	Sally Hargraves	sally@tfel.co.nz
Elected Member	Rolando Orense	r.orense@auckland.ac.nz
Elected Member	Jen Smith	jsmith@tonkintaylor.co.nz
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ISRM Australiasian Vice President	Paul Horrey	Paul.Horrey@beca.com
ISSMGE Australasian Vice President	Phil Robins	Philip.Robins@beca.com

EDITORIAL POLICY

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

Readers are encouraged to submit articles for future editions of *NZ Geomechanics News*. Contributions typically comprise any of the following:

- technical papers which may, but need not necessarily be, of a standard which would be required by international journals and conferences
- technical notes of any length
- feedback on papers and articles published in *NZ Geomechanics News*
- news or technical descriptions of geotechnical projects
- letters to the NZ Geotechnical Society or the Editor
- reports of events and personalities
- industry news
- opinion pieces

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

Articles and papers are not normally refereed, although constructive post-publication feedback is welcomed. Authors and other contributors must be responsible for the integrity of their material and for permission to publish. Letters to the Editor about articles and papers will be forwarded to the author for a right of reply. The editors reserve the right to amend or abridge articles as required.

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

NEW ZEALAND GEOTECHNICAL SOCIETY INC

NZGS Membership SUBSCRIPTIONS

Annual subscriptions cost $135 per member. First time members will receive a 50% discount for their first year of membership; and student membership is free. Membership application forms can be found on the website http://www.nzgs.org/membership.htm or contact the NZGS Secretary on secretary@nzgs.org for more information.

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

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

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

Advertisers' Directory

ADVERTISING

NZ Geomechanics News is published twice a year and distributed to the Society's 1000 plus members throughout New Zealand and overseas. The magazine is issued to society members who comprise professional geotechnical and civil engineers and engineering geologists from a wide range of consulting, contracting and university organisations, as well as those involved in laboratory and instrumentation services. NZGS aims to break even on publication, and is grateful for the support of advertisers in making the publication possible.

TYPE	BLACK AND WHITE	COLOUR	SPECIAL PLACEMENTS		SIZE
			INSIDE FRONT OR BACK COVER	OPPOSITE CONTENTS PAGE	
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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
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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.
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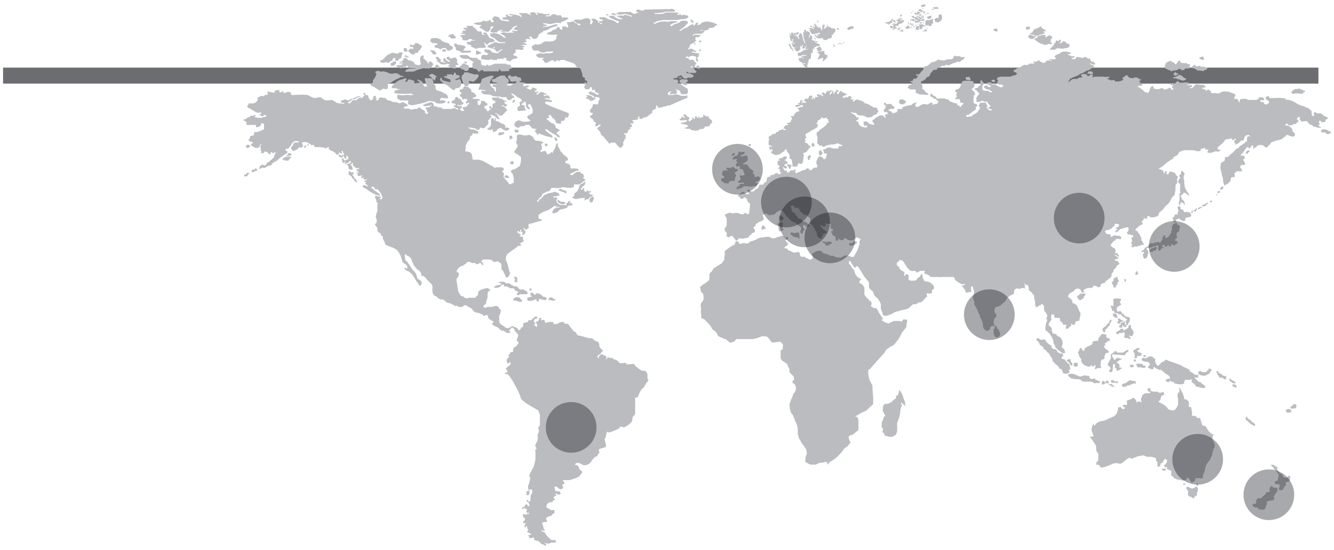
5. Advertisers are responsible for ensuring they have all appropriate permissions to publish. This includes the text, images, logos etc. Use of the NZGS logo in advertising material is not allowed without pre-approval of the NZGS committee.

National and International Events



2021

8-12 April 2021
Athens, Greece
3rd European Regional Conference of IAEG

NZGS SYMPOSIUM 2021
Good grounds for the future
24-26 March 2021 • Dunedin • New Zealand

24-26 MARCH 2021
Dunedin, New Zealand
NZGS Symposium 2021
"Good Grounds For The Future"

7-9 June 2021
Naples, Italy
Mediterranean Symposium on Landslides

28-30 June 2021
Cambridge, England
TC204 Geotechnical Aspects of Underground Construction in Soft Ground

9-10 September 2021
Fukuoka, Japan
5th International Workshop on Rock Mechanics and Engineering Geology in Volcanic Fields

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

21-25 October
Beijing, China
ARMS11 – 11th Asian Rock Mechanics Symposium – Challenges and Opportunities in Rock Mechanics

6-7 December
Colombo, Sri Lanka
3rd International Conference on Geotechnical Engineering

2022

15-18 May
Asuncion, Paraguay
LARMS2022 IX Latin American Congress on Rock Mechanics

26-28 June
Chania, Greece
9th International congress on Environmental Geotechnics

2023

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

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