NEW ZEALAND GEOTECHNICAL SOCIETY INC

DECEMBER 2025 Issue 109

NZ GEOMECHANICS NEWS

Bulletin of the New Zealand Geotechnical Society Inc.

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NZGS AND YGP SYMPOSIUM HIGHLIGHTS

MEET OUR 2025 PHOTO COMPETITION WINNERS

- ADVANCES IN LIQUEFACTION BASED ANALYSIS AND RESEARCH
- COMPLEX TUNNEL PORTAL RETAINING WALL DESIGN AND MONITORING DURING EXCAVATION
- GEOGRID REINFORCED LOAD BEARING BRIDGE ABUTMENTS DESIGN AND CONSTRUCTION
- PGA ADJUSTMENT FACTORS TO TS1170
- MAPPING TRAINING IN FIJI

RETAINING YOUR BUSINESS IS OUR BUSINESS

Design

Construct

Perform

Improve

GEOTECHNICAL ENGINEER VACANCIES

ABOUT EGL

EGL are based in Albany, Auckland. Since 1988 we have provided specialist geotechnical, earthquake and dam engineering consultancy services throughout New Zealand, Australia and the wider Asia-Pacific region.

The core activities of the internationally recognised EGL team are:

- **Geotechnical investigation, engineering design and construction support services for a variety of building types and retaining wall constructions.**
- **Dam and civil engineering for irrigation dams, water supply dams, flood control works, mine waste and tailings disposal.**
- **Seismic hazard and earthquake engineering.**

EGL are looking for two talented Geotechnical Engineers to join their Water Resources Team in Auckland and focus on audit, design and construction support services for water storage and tailings dam projects.

AS A MINIMUM, THE IDEAL CANDIDATES FOR THIS POSITION WOULD HAVE:

- A Bachelor of Civil Engineering Degree from a Washington Accord University.
- 5 to 15 years experience in the fields of geotechnical and civil engineering.
- Previous experience working on dam or water resource projects. Alternatively, experience working on large infrastructure projects would be of significant relevance.
- Attained or is close to attaining Chartered Professional Engineer status.
- Full New Zealand Drivers License.
- New Zealand resident status.

WHY YOU SHOULD CONSIDER JOINING EGL:

EGL has a focus on working on interesting and professionally rewarding projects. The company leadership strives to foster a supportive and collaborative working environment which allows staff to learn and grow professionally.

The key benefits of working at EGL are:

- A high ratio of senior to junior staff to enable close mentoring relationships.
- A competitive salary, that will be commensurate to the candidates experience.
- A small, friendly team of very clever people.
- Opportunity to learn from some of New Zealand's leading geotechnical and dam design specialists.
- Free parking available around the office location.
- The company is led and managed by Geotechnical Engineers and Geologists and has a focus on technical excellence.
- Opportunities to work on interesting projects throughout New Zealand and overseas.

HOW TO APPLY: **Please direct all submissions of interest for this vacancy to the EGL Managing Director, Mr Tony Fairclough E tony.fairclough@egl.co.nz PH +64 9 486 2546.**

Applications for this vacancy close 17/01/2026

EGL
Geotechnical, Earthquake and Dam Engineers

CONTENTS DECEMBER 2025

COVER IMAGE: Tight spaces, big views – ground investigation on a 15-metre-high scaffolding platform on the banks of the Weiti River for the Ō Mahurangi Penlink Project, by Matt Cook.

"There's no i in Team"

THERE IS A saying in our local Ground Engineering Team that "Engineering is a Team Sport" and this edition of *NZ Geomechanics News* showcases that collaboration between researchers, consultants, contractors and clients in New Zealand is going from strength to strength. The December issue of *NZ Geomechanics News* contains a great snapshot of an industry that's constantly evolving, tackling new challenges, and pushing innovation forward together. The stories and research featured here reflect an industry built not only on technical expertise but also on strong partnerships, shared knowledge, and collective problem-solving.

Among the highlights, the NZGS 2025 Symposium held in Auckland offered a dynamic program with keynote presentations covering earthquake and climate change-related hazards, sustainable engineering approaches, liquefaction testing, and managing uncertainty in design. The event brought together nearly 500 participants who explored a diverse range of technical topics through presentations, posters, and field trips navigating Auckland's volcanic and landslide geology. You will find all the winning papers and poster from the Symposium in this edition of *NZ Geomechanics News*.

Many of the articles in this edition of the magazine, show how collaborative efforts lead to better understanding and innovative solutions. Advanced research on liquefaction resistance in gravelly soils, for instance, draws on combined expertise from laboratory innovation and field data, showing the power of multidisciplinary teamwork. Similarly, predictive models for liquefaction ejecta and sophisticated soil reinforcement techniques are products of ongoing dialogue between researchers, engineers, and practitioners. The various projects covered, including complex retaining walls and bridge abutments designed for seismic resilience, demonstrate how practical engineering thrives through coordinated efforts amongst researchers, design teams, contractors, and other specialists. These successes underline that strong teamwork transforms challenging environments into opportunities for innovation and excellence.

In research, important strides have been made in seismic hazard modelling and liquefaction assessment. Recent studies have identified that traditional seismic hazard models tend to overestimate peak ground accelerations on soft soils, leading to the development of adjustment factors tailored to New Zealand's upcoming seismic standard TS1170.5:2024. These refinements promise more accurate seismic design parameters, especially for critical infrastructure built on soft ground. At the same time, advances in understanding liquefaction behaviour of gravelly soils have been achieved through novel specimen preparation methods that better replicate natural soil fabric. Such research is vital for improving liquefaction hazard assessments, as it demonstrates the significant influence of soil fabric, density, and gravel content on liquefaction resistance. This December issue also features a detailed technical paper on geogrid reinforced soil (GRS) bridge abutments using extensive geogrid elements previously not adopted on the NZTA network, highlighting modern design and construction techniques including novel preloading techniques utilising prestressing anchors, subsoil drainage, and performance monitoring to confirm design assumptions. The case study emphasizes how good collaboration between innovative engineering and willing clients can deliver robust, cost-effective bridge abutments capable of withstanding seismic demands common in New Zealand.

Branch activities and international collaborations remain integral to the vitality of our profession, providing valuable opportunities for continuous learning and networking. Workshops, training courses, and symposiums contribute to the ongoing development of geotechnical expertise, ensuring we remain at the forefront of global best practices, and we have reports of many of the activities that have been happening across the country in this edition of the magazine.

Lastly, for those of you interested in publishing papers, Robert and I were pleased to be asked to guest edit an edition of the Australian Geomechanics Society Journal, this will be a special New Zealand Themed edition, and we are still on the hunt for more papers. You will see the deadline for abstracts has been extended so please help us to show those Australians the broad range of exceptional geoprofessionals that New Zealand has! And thank you to those that have already submitted an abstract, it is already looking to be a great edition based on the calibre of abstracts and authors. If you have a paper you would like to publish that is applicable to Australia but based on a New Zealand site / research / topic, do not be put off by the peer review process, please go ahead and submit. You will find details in the advert included in this edition of the magazine. Any questions please email editor@nzgs.org

As we head into the holiday season, we would like to thank all the society members who have contributed to this edition of the magazine, we wish you all a happy, safe, and restful Christmas break.

Camilla Gibbons & Robert Kamuhangire
NZ Geomechanics News Co-editors

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

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

From the Chair

Philip Robins is a Principal Geotechnical Engineer and Technical Director at Beca. Philip is an acknowledged specialist in geotechnical engineering, high-seismicity engineering and design development, and is recognised by his peers as a Fellow of Engineering New Zealand. Trained as a civil engineer with a broad range of experience, locally and internationally, Philip brings outstanding technical expertise in geotechnical engineering leadership that spans all sectors of civil infrastructure. Over the past 30 years, he has consistently shown his ability to lead geotechnical design and the development of geotechnical designs for numerous projects while developing key client relationships. Philip is a Nominated Member of the ISSMGE Technical Committee (TC104) - Physical Modelling in Geotechnics and (T220) - Field Monitoring in Geomechanics and was the ISSMGE Vice President – Australasia 2019 - 2021. Philip served on the NZGS Management Committee in 2009 and 2010 and was on the organizing committee for the NZGS Symposium in Dunedin, March 2021. Philip is now based in Palmerston North, where he moved with his family at the end of 2021.

Phil Robins
Chair, Management Committee

Kia ora koutou

As the year draws to a close, I find myself reflecting on my tenure as Chair of the New Zealand Geotechnical Society (NZGS). The past two years have been a journey of collaboration, innovation, and growth—one that has left me inspired by the passion and talent within our geo-professional community.

CELEBRATING OUR PEOPLE

Returning from the Young Geotechnical Professionals (YGP) Breakfast Series at the NZGS Symposium 2025 in Auckland, I am more confident than ever in the future of our craft. The energy and commitment of our members, especially those stepping up for the 2026/28 Management Committee, assure me that the future of the NZGS is in excellent hands.

ACHIEVEMENTS AND PARTNERSHIPS

During my term, NZGS focused on three strategic themes, and I'm proud to say we've met most of our ambitious goals. We've advanced key projects, developed new guidance documents, and fostered collaboration with technical societies such as SESOC, NZSEE, CETANZ, and Engineering New Zealand. Our partnership with MBIE has been instrumental in updating the NZ Geotechnical Database and developing vital documents. Notably, our NHC-funded slope stability guidelines have reached an international audience.

INTERNATIONAL CONNECTIONS

NZGS's global engagement continues to grow. Through our representatives in ISRM, ISSMGE, and IAEG, we've strengthened overseas ties. Hosting distinguished guests at our Management Committee meetings and successfully bidding for the First International Joint Workshop of Joint Technical Committee 1 and 3 on Landslide Risk Assessment, Communication, and Geo-education are highlights. Next year's LaRGE event in Queenstown promises to unite leading experts and drive real change in landslide risk assessment and education.

SHAPING THE FUTURE OF ENGINEERING

NZGS was invited to join the steering group for Engineering New Zealand's new strategy project, addressing integration and operational challenges among engineering groups. We also signed a collaboration agreement for the Building Resilience in Design Guidance and Engineering (BRiDGE) Initiative, positioning NZGS at the forefront of research and practical application. Our proposal to develop guidelines for ground-governed structures and non-elastic design is under consideration.

GOVERNANCE AND STANDARDS

At our Special General Meeting, members approved NZGS's re-registration under the Incorporated Societies Act 2022 and Regulations 2023, ensuring our continued charitable status. I also chaired the Coordination Group (TCG-05) established by Standards New Zealand to provide strategic advice on geotechnical standards, a role commissioned by MBIE's Building System Performance branch.

ONGOING PROJECTS AND ADVOCACY

Our members remain active in initiatives such as TS1170.5, JC-SAR, C4 Geotechnical Considerations, Low Damage Seismic Design, and updates to VM4. We continue to advocate for geo-professionals, especially regarding proposed changes to CPEng Rules and the recognition of PEngGeol post-nominal.

LOOKING AHEAD

We're bidding to host the ISRM Congress in Christchurch in 2031, with strong support from Tourism New Zealand and Christchurch City Council. Our team presented the bid at EuroRock 2025 in Norway, and we await the outcome at ARMS2026 in Japan.

While our regional YGP mini symposia have thrived, we recognize the need to expand training and CPD opportunities. Planning is underway for 2026, with more presentations, webinars, workshops, and international guest series on the horizon.

GRATITUDE

It has been an honour to work alongside such a dedicated and hardworking Management Committee. Together, we have exceeded expectations and set a strong foundation for the future.

Noho ora mai,
Philip Robins
Chair 2024-2025

NZGS Management Committee Updates

IOANNIS ANTONOPOULOS
VICE CHAIR

Ioannis is a Chartered Geotechnical Engineer specializing in large infrastructure and development projects. He enjoys working with interdisciplinary teams on both design and construction, focusing on geotechnical earthquake engineering, water reservoirs, roading, ports, seawalls, foundations, cut-and-cover structures, tunnelling, slope stability, hydrogeology, and water resource management. As a volunteer for Engineering NZ, Ioannis serves as a Practice Area Assessor and frequently contributes to conferences as a presenter and reviewer. His expertise includes earthquake geotechnical engineering, soil-foundation-structure interaction (SFSI), numerical analysis and modelling, retaining structures, geotechnical design of soft soils, and geotechnical material characterization. Ioannis began his career in Greece, working on projects like the Athens Metro, the new Athens Conference Centre – Alexandra Trianti Hall, several highways, the Costa Navarino resorts, and commercial high-rises. Since 2012, he has been in New Zealand, contributing to major transport and water-related infrastructure projects, including Waka Kotahi NZ Transport Agency's highways, ports, dams, and levees.

AS 2025 DRAWS to a close, it is a pleasure to share highlights from a year of progress and collaboration within the New Zealand Geotechnical Society (NZGS). This year has been defined by strong governance, technical leadership, and a commitment to enhancing member experience.

STRENGTHENING GOVERNANCE AND TRANSPARENCY

The Management Committee convened three times—in Christchurch, online, and Auckland—focusing on strategic priorities and stakeholder engagement. A major milestone was the completion of the Society Rules revision in September, ensuring compliance with the Incorporated Societies Act and improving clarity for members. During the 2025–2026 election cycle, the committee acted decisively to correct candidate information and reissue ballots, reinforcing confidence in our democratic processes.

DRIVING TECHNICAL EXCELLENCE

NZGS continued to lead on technical standards. Representation in the BRiDGE initiative introduced an emerging professional "Seconder" role, creating pathways for future leaders. Work progressed on aligning Module 6 with AS/NZS 1170.0, including an interim clarification process and plans for formal revisions. Additionally, a concept brief for a new Code of Practice for seismic design of ground-governed structures was developed and submitted to BRiDGE, laying the foundation for a coherent compliance framework that will benefit practitioners and infrastructure agencies alike.

Enhancing Member Engagement Member communications were strengthened through timely newsletter reviews and clear updates on membership fee adjustments effective October. The NZGS Symposium 2025 provided an excellent platform for knowledge sharing, with technical presentations and vibrant discussions. Award processes were streamlined, ensuring fair recognition of excellence across our community.

LOOKING AHEAD

Priorities for 2026 include advancing focus on geotechnical engineering, testing and design, continuing to work on Module 6 updates, advancing the Code of Practice from concept to draft, and embedding the BRiDGE Seconder role to sustain leadership development within NZGS.

Thank you to all members and partners for your contributions throughout the year. Together, we continue to build a resilient and forward-looking geotechnical community.

EMILIA STOCKS
TREASURER

Emilia is a Chartered Principal Geotechnical Engineer and Risk and Claims Adviser with Tonkin + Taylor Ltd, based in the Wellington office. She has over 16 years of experience across a wide range of geotechnical and civil engineering projects. Emilia has led several major geotechnical initiatives, including developments on the Wellington waterfront, new landfill projects, and the design and construction of retaining walls for roads and stopbanks. She is a Board Member for CEAS, a Director of I&G Insurance, and a Member of the Institute of Directors (IoD). Her expertise includes geotechnical investigations, liquefaction damage assessment, evaluation and design of liquefaction mitigation measures, and foundation design. Emilia is recognised for her commitment to continuous improvement, quality assurance, and proactive risk mitigation, all of which contribute to consistently strong project outcomes. Outside of work, Emilia volunteers with the Wellington Emergency Response Team (NZRT8)
treasurer@nzgs.org

WE CONTINUE TO monitor our finances closely to ensure sustainable operations that support research, events, and initiatives for our members and the wider geotechnical community. The budget for the current year has now been approved and includes allocations for ongoing guidance development and educational programmes.

As previously advised, the revised membership fees took effect on 1 October 2025. This adjustment, following a detailed financial review by the NZGS Management Committee, provides additional funds to help sustain and enhance the services, resources, and advocacy we offer our members.

JESSE BEETHAM
NATIONAL BRANCH COORDINATOR

Jesse Beetham is an Engineering Geologist (PEngGeol) with Tonkin & Taylor, based in the Tauranga office. He has been based in Tauranga for all of his career however, he has worked on projects all across the country. Jesse considers himself a true-blue Engineering Geologist with a strong background in the Earth Science field.

AS WE HEAD toward the end of 2025, it's fair to say that activity across most NZGS branches has been relatively quiet. While some regions have managed to host events and welcome new representatives, many branches are finding it challenging to maintain momentum and to gather crowds.

Despite this, we want to sincerely thank all our volunteers, past and present, who continue to give their time and energy to support the geotechnical community. Your contributions are valued, even during slower periods. Big shout out to the Tauranga NZGS Branch (Kim de Graaf, Rhiannon Robinson, and Matt Packard) for being the most active branch in 2025! We really appreciate the commitment to running a very successful branch!

If you have an idea for a branch event, presentation, or site visit, please don't hesitate to reach out to your local branch reps or the NZGS Secretary. A small spark can go a long way in reigniting engagement.

MARTIN LARISCH
CHAIR OF NZGS CLIMATE CHANGE RESILIENCE & ADAPTATION GROUP

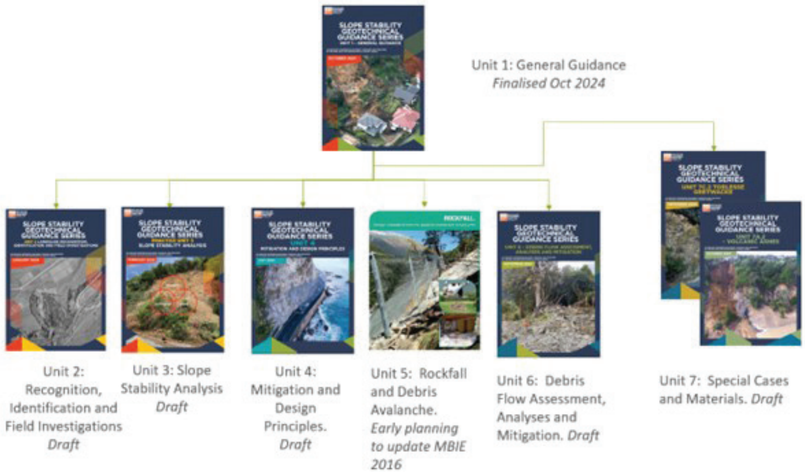
Martin Larisch is a Geotechnical Engineer with more than 25 years of international design and construction experience. He is based in Waikanae (Kapiti Coast), where he works as an Independent Consultant and Expert Witness on various geotechnical, piling, ground improvement and retaining wall projects across New Zealand and the Asia Pacific Region.

Since 2020, he is a member of the expert panel to revise the NZGS/SESOC Piling Specifications and he is also the current Chair of the NZGS Climate Change Resilience and Adaptation Group.

THE WORKING GROUP has not met in the last 6 months.

The guidance document *Climate Change & Resilience Advisory Note 1 - Climate Change Considerations for Geo Professionals* was finalised and is currently with the NZGS editors for final touches before it will be published soon.

If you are interested in actively shaping our future and profession and consider joining our group, please send an enquiry to secretary@nzgs.org with your short bio and some background why you would like to join the group.



RICHARD JUSTICE

PROJECT LEAD – SLOPE STABILITY GUIDANCE

I'm a Principal Engineering Geologist with ENGEO based in Christchurch. I graduated from the University of Canterbury in 1995. I was initially employed with Pells Sullivan Meynink, based in Sydney. After six years, I moved to URS, also in Sydney before moving to Wellington to be with Tonkin + Taylor. In 2008 I made the move to KiwiRail, to experience life on the client side for a while.

In 2012, I helped set up the Wellington office of Geoscience NZ (now ENGEO), before moving to Christchurch in 2014 and have been there since, apart from a four-and-a-half-year stint working on the North Canterbury Transport and Infrastructure Recovery (NCTIR) project. My work passion is engineering geological models - making sure that we don't forget the geological part of our assessments.

WORK ON THE Slope Stability Guidance under our current contract with NHC is now complete, or at least is very close to completion. All of the following units are now available on the NZGS website:

Unit 1 – General Guidance lays the foundation for upcoming technical units (Units 2–7) and covers slope movement types, landslide identification, investigation methods, geological modelling, risk assessment, stability analysis, hazard mitigation, design principles, and emergency response.

Unit 2 – Landslide Recognition, Identification, and Field Investigations builds on Unit 1, detailing how to identify and assess various landslide types using geological and geomorphological features. It also reviews resources and techniques for effective investigations, summarizing their strengths and weaknesses to assist future units.

Unit 3 – Slope Stability Analysis focuses on evaluating landslide triggers in soil and rock slopes using limit equilibrium methods (LEM), widely used in New Zealand and internationally. It highlights practical LEM applications, mechanics, and common issues in complex scenarios.

Unit 4 – Mitigation and Design Principles provides strategies for stabilising slopes, including engineered and non-engineered solutions. It discusses risk-based approaches, performance criteria, engineered and bioengineering measures, sustainability integration, safety standards, and case studies.

Unit 6 – Debris Flow Assessment, Analyses, and Mitigation offers guidance tailored to New Zealand's environment for assessing and managing debris flow hazards, ensuring best practices for engineers and related professionals.

Unit 7 – Special Cases and Materials tackles challenges from New Zealand's distinct geology, focusing on retaining local expertise by documenting knowledge for consistent engineering practices; its first section is now available. Three out of an eventually planned eight units have currently been developed and are:

Unit 7A.2 – Volcanic Ashes
Unit 7B.1 – Auckland
Unit 7C.2 – Torlesse Greywacke; with a focus on the Wellington area

Thank you to all the authors, members of the steering committee, peer reviewers and NZGS members who have taken the time to provide comment. It has been a privilege to have been involved with the development of these now internationally recognised documents, and we should all be proud! There is still more to come, with Unit 5 – Rockfall and Debris Avalanche not currently in development. While we are in discussion with funding partners work is unlikely to begin during NZGS's 2025/2026 financial year. But we also have five documents under Unit 7 still to go, so stay tuned for information on Otago Schist, Northland Allochthon and Loess Soils, amongst others!

AUSTRALIAN GEOMECHANICS
SPECIAL EDITION MARCH 2027

New Zealand Geomechanics Research and Practice Call for Papers

AUSTRALIAN GEOMECHANICS SOCIETY

NEW ZEALAND GEOTECHNICAL SOCIETY INC

Call for Submissions: Special Edition of *Australian Geomechanics*, journal of the Australian Geomechanics Society, featuring New Zealand contributions

The Australian Geomechanics Society invites submissions for a Special Edition of its Journal — *Australian Geomechanics* — that showcase the vibrant geotechnical engineering and engineering geology landscape of New Zealand. This exciting initiative underscores the strong partnership between our two nations and aims to highlight the shared challenges and recent advancements in Geotechnical Engineering and Engineering Geology.

THEMES

We encourage researchers, practitioners, and students to contribute papers on a diverse array of New Zealand-themed topics that would resonate with an Australian audience. Suggested themes may include:

- Innovative geotechnical methods and technologies
- Environmental geotechnics and sustainable practices
- Ground behaviour and site characterisation
- Landslide risk assessment and management
- Soil-structure interactions and foundation engineering
- Geohazards and natural disaster management
- Case studies of significant engineering projects

This is a unique opportunity to publish your work in a respected Australian journal, thereby broadening your reach and influence in the geotechnical community.

Please submit your abstracts for consideration by 3rd October 2025. We look forward to your contributions that celebrate the synergy between New Zealand and Australia in the field of geomechanics.

For further details, please contact editor@nzgs.org. Let's showcase the best of New Zealand's geotechnical expertise!

DEADLINE EXTENDED UNTIL 15TH JANUARY 2026

PAPER SUBMISSION

For further guidance on the preparation of papers, editorial policy and how to submit an abstract for consideration please refer to the *Australian Geomechanics* journal webpage:

https://australiangeomechanics.org/journals/

Abstracts of no more than 300-words should be submitted via Scholastica by 3rd October 2025 for consideration by the Guest Editors. We encourage submitting an abstract first to receive confirmation from the Guest Editors before completing and submitting a full paper.

Papers selected for publication will be based on their quality and relevance. Final paper to be submitted by 1st June 2026.

Papers for publication in this themed issue will be based on their quality and relevance to the topic. We encourage submissions from the geotechnical profession, other geoscience practitioners, the quarry industry, as well as academia.

All papers are peer reviewed.

LIAM WOTHERSPOON
TRAINING & SHORT COURSES, TECHNICAL WORKING GROUP LIAISON

Liam is a Professor in the Department of Civil and Environmental Engineering at the University of Auckland. He has held an academic position in the department since 2009 and has been involved in the teaching of a wide variety of geotechnical engineering courses. His research also covers a range of geotechnical engineering areas and extends into structural and infrastructure engineering. He has worked with a number of professional organisations to translate the outputs of his research into practice and support the evolution of best practice.

AS PART OF the Technical Working Group Liaison role, Liam has supported the connection and coordination across the working groups of NZGS and those led by other professional societies and regulatory orga isations. He co-chairs the NZGS Technical Coordination Group, providing coordination for geotechnical aspects of seismic design across different design documents that are currently in development. He supported work to align the retaining wall design guidance from SESOC and NZGS, He is involved in the development of new standards focussing on natural hazards risk as a representative of NZGS.

Liam supported the development of the In-situ Testing Practical Workshop as part of the NZGS Symposium, focussing on geophysical methods for engineering application. He was involved in the development of the Mick Pender Memorial Session and the Geo-education Session for the Symposium.

As part of the Geo-education subcommittee he has been developing initiatives related to the development and expansion of the profession. This has included the approval of ethics for a set of interviews that will be undertaken over the next few months.

LaRGE 2026

Landslide Risk & Geo-Education

27 APRIL - 3 MAY 2026

An International Workshop of JTC1 & JTC3

QUEENSTOWN, NEW ZEALAND

NAU MAI, HAERE MAI – WELCOME!

New Zealand invites you to a landmark internation event - the 1st International Joint Workshop of Joint Technical Committee 1 and Joint Technical Committee 3. We will share the latest research and develop best practice guidelines in the stunning city of Queenstown. Our theme *"Landslide Risk & Geo-Education"* unifies the full lifecycle of landslide risk management. It encompasses the need to educate the next generation of landslide risk managers, to understand landslide risk, and to communicate that risk to the public and decision makers so that real change is implemented. Bringing together JTC1 and JTC3 on key aspects of landslide risk – assessment, education, communication, and outreach – will drive strategic improvements in managing landslide risk. You'll hear from international experts including:

David Petley is recognised widely as a world leader in the study and management of landslides and for his popular blog on landslides which receives over 500,000 individual visits per year.

Jean Hutchinson is a Professor Emerita of Geological Engineering at Queen's University, Alberta Canada, and the Vice President of Innovative Geomechanics Inc.

Gonghui Wang is a professor at the Disaster Prevention Research Institute (DPRI), Kyoto University Japan, and serves as the head of the Research Center for Landslide Risk Cognition and Reduction at DPRI

Jo Horrocks is Chief Resilience and Research Officer at the Natural Hazards Commission, leading their science, data, and modelling to improve understanding of natural hazard risks and how to reduce them.

Reginald Hermanns is Professor at Norwegian University of Science and Technology. Research includes rock-slope stability, and the technical and societal response to landslide threats.

Lori Peek is director of the Natural Hazards Center and professor in the Department of Sociology at the University of Colorado Boulder. She has written award-winning books on the social impact of disasters.

Nicola Casagli is professor of Engineering Geology at the University of Florence, immediate past President of the International Consortium of Landslides, and President of the 6th World Landslide Forum.

Tim Davies is a former member of JTC1, convenor of the conference series on Debris-Flow Hazard Mitigation, and former Editor of Journal of Hydrology (NZ). He has held visiting fellowships at Durham University, UK and ETH-Zürich.

Ann Williams is Past Chair and Life Member of NZGS, past Vice President and Honorary Member of the IAEG and has worked internationally on landslide risk assessment and reduction.

Janusz Wasowski is the Editor-in-Chief of Engineering Geology. His research includes landslide assessment, collateral seismic hazards, and air/space-borne remote sensing.

This international workshop conference is hosted by the New Zealand Geotechnical Society and is endorsed by the member societies of the Federation of International Geo-Engineering Societies:

New Zealand Geotechnical Society

FedIGS

IAEG ENGEOL

SIMSG ISSMGE

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Field trips

Clyde Dam Landslide Stabilisation

Known landslides were monitored during the construction phase, and it was discovered that some 'dormant' slides in the Cromwell gorge were slowly moving downhill. Exploratory drilling for a new highway led to the discovery of a complex, high-pressure groundwater system, and this led on to an extensive drilling programme on other landslides. A strategy was developed for a fast-track stabilisation program, based primarily on the use of tunnels for both investigation and drainage.

Glenorchy Resilience Project

With a focus on education, natural hazard communication, and community resilience, this trip will visit the stunning village of Glenorchy. Directly exposed to multi-hazards from flooding, earthquakes and liquefaction, it is vulnerable to being cut off by landslides. A natural hazards adaptation strategy was developed in partnership with the local community. This tour will investigate how the strategy was developed and is being implemented with the community.

Milford Sound Cruise

Deep within Fiordland National Park lies Milford Sound, New Zealand's most stunning natural attraction. A million people a year visit Milford Sound. The nearby Alpine Fault ruptures, on average, every 330 years with a magnitude 8 earthquake, and this would likely cause a very significant rockslide. A landslide-triggered tsunami may leave no survivors, with as many as 3500 dying. This field trip will explore the decision-making process required to balance the public interest in visiting this natural wonder with the potential risk it poses.

Registration open now at landsliderisk.nz

Why attend? This landmark international event unites JTC1 and JTC3 to advance landslide risk assessment, education, communication, and outreach – creating a unique opportunity for diverse impacts, and will be attended by leading experts from around the world.

The workshop is structured around specific projects through interactive sessions. Beyond disseminating knowledge, we will generate new ideas, develop ongoing projects, and create tangible outputs including guidelines and research direction.

LaRGE2026 also delivers great training courses, keynote speeches, presentations, poster sessions, and field trips. The training courses will span landslide risk assessment, emergency response, science communication, and landslide geoeducation.

Sponsorship opportunities available now!

Why sponsor? By sponsoring, your organisation will have the opportunity to profile your ideas and solutions on the global stage as well as connect with global experts, local practitioners, government representatives and key decision makers from around the world. Your name will be associated with real deliverables that will outlive the event and drive meaningful change in New Zealand and around the world. We look forward to your participation in making LaRGE2026 a success, and to being permanently associated with the great outcomes of the workshop!

Programme – Tue 28 April to Sun 3 May 2026

Mon — **New Zealand Public Holiday** - IAEG Executive meeting and reserve day for training and fieldtrips.

Tues — **Field trips & exercises** - Three field trips carefully aligned with the objectives of the workshops.

Wed — **Training** - Learn from industry experts in the field of landslide risk management and science communication. Offerings include land use planning for landslide risk reduction, media training, rapid building assessment, slope stability guidelines and more.

Thur — **Workshop Day 1 - Susceptibility, Data & Risk**. Presentations and workshops on advanced monitoring techniques.

Fri — **Workshop Day 2 - Risk to Policy**. Presentations and workshops on landslide risk assessment techniques.

Sat — **Workshop Day 3 - Outreach & Education.** Presentations and workshops on geoeducation and risk communication.

Sun — **Additional Field Trips & Exercises** including an informal wine-tasting landslide focused trip.

2025 NZGS PHOTO COMPETITION WINNERS

2ND PLACE

WE HAD A great range of photos for this year's competition which made the judging rather more tricky. Congratulations to our first second and third placed photos which are some great interpretations on the theme of "AMAZING SPACES: Tight and Cramped, or Picturesque"

FIRST PLACE

Tight spaces, big views — ground investigation on a 15-metre-high scaffolding platform on the banks of the Weiti River for the Ō Mahurangi Penlink Project

by Matt Cook

SECOND PLACE

Sandwiched between SH1 and neighbouring properties – P2D Driven UC Pile Ground Improvements

by Robert Pirrie

THIRD PLACE

Mount Tongariro on film

by Jono Sorley

3RD PLACE

KiwiRail Update

December 2025

KiwiRail Geotechnical Team

OCTOBER ONCE AGAIN brought severe spring storms, testing the resilience of the rail network. Heavy rainfall triggered slips and flooding across the Central North Island between Taumarunui and Te Kuiti, a severe wind event impacted the South Island and heavy rain occurred on the West Coast near Greymouth. These events echo last year's major landslides along the Main South Line, reinforcing the need for proactive geotechnical monitoring and robust asset strategies.

To meet these challenges, KiwiRail is leveraging technology to improve asset intelligence. The team has deployed UAV-mounted LiDAR systems using the Matrice 350 RTK paired with the L2 unit. This capability enables high-resolution corridor mapping and temporal change detection in ArcGIS Pro, even through dense vegetation. Unlike publicly available LiDAR, which lacks the precision for narrow rail corridors, KiwiRail's approach supports a more refined data capture —critical for landslide and geotechnical asset management.

Meanwhile, construction works along the Palmerston North to Gisborne Line (PNGL) are winding down as the TREC Alliance continues to close out minor works across multiple sites. Physical repairs to damage sites between Palmerston North and Hastings have been completed, restoring key sections of the corridor. Additional improvement funding has enabled the replacement of several culverts, significantly enhancing drainage and overall resilience in the area. Ballast replacement works have also been progressing and are now near completion, delivered through a joint

effort between KiwiRail's regional team and TREC contractors. North of Napier, minor risk mitigation works are underway to address localised vulnerabilities, ensuring the corridor remains robust against future weather events. KiwiRail's Civil Engineering team is overseeing the handover process, to ensure assets are safely returned to service.

Finally, planning for the future is well underway. Development of the Rail Network Investment Programme (RNIP 3) for FY28–FY30 has commenced, with the Engineering and Asset Management team adopting a risk-based approach to renewals. This standardised methodology across asset types will help prioritise investment where it matters most, building resilience into the network for years to come.

What's On at NZ Transport Agency Waka Kotahi

Luke Storie, Lead Advisor Geotechnical Engineering, Office of the Chief Engineer, Transport Services, NZ Transport Agency Waka Kotahi

Luke Storie

NEW BEGINNINGS

I started as the Lead Advisor Geotechnical Engineering in June 2025, attempting to fill the big shoes left by Stuart Finlan after his retirement. It has been an exciting and busy first few months with the agency, with new initiatives to sink my teeth into almost every week and the tough task of prioritizing the work that needs to be done across new projects, asset management, collaborating with the wider Sector, and everything in-between. Thank you to everyone who has reached out to me and welcomed me into the role. I am really looking forward to being part of solving some of the geotechnical challenges that we face.

For those that don't know much about me, I have a passion for geotechnical earthquake engineering and in particular soil-foundation-structure interaction (SFSI) and liquefaction and lateral spreading. I completed a PhD in SFSI, looking at rocking foundations in the Christchurch Earthquakes, and have carried out extensive work in liquefaction assessment and mitigation during my prior 9+ years at Tonkin + Taylor. I am also passionate about understanding and mitigating against natural hazards in general, in particular rainfall induced landslides having worked through a number of events with the Natural Hazard Commission (NHC) in my previous role. Collaboration is of high importance to me, particularly with our structural engineering colleagues, and I think there is always room for improvement in that space.

My goal/aim at NZTA is to promote cross-discipline interaction and drive the latest advancements in practice, strongly advocating for putting research into practice to achieve pragmatic and efficient engineering design. I'm looking forward to the opportunity to shape policy and planning, work with industry and subject matter experts, integrate risk-based approaches and provide national technical leadership.

SEISMIC DESIGN OF TRANSPORT INFRASTRUCTURE

With my colleague Moustafa Al-Ani, Lead Advisor Structures, we are embarking on a project to revamp seismic design of transport infrastructure at NZTA. Seismic hazard for design tends to be derived from a focus on life safety associated with buildings and occupancy. For horizontal infrastructure such as the state highway network, consideration of other metrics such as route criticality and resilience, alongside life safety, is important in seismic design. We are exploring how to integrate resilience metrics into setting seismic design hazard and help streamline decision-making for the life cycle of projects, both small and large.

GEOSYNTHETIC SOIL REINFORCEMENT APPROVALS PROCESS

NZTA have been managing the approval process of geotechnical soil reinforcement products for over a decade, undertaking review of the documentation provided by suppliers and providing certification of the products. However, the scale, complexity, and subsequent cost of managing the approval of these products has become unsustainable and we are exploring options to develop a new approvals system for geosynthetic soil reinforcement products. The intent is for the new system to be more efficient and sector-led, with product approvals funded by suppliers and obtained directly from accredited certifiers, in-line with other established product approvals systems.

We have engaged with the National Transport Research Organisation (NTRO) in Australia to review our existing framework, benchmark against international best practice, and propose a framework for approval of these products in NZ that aligns with the National Harmonisation Framework for Australia-NZ transport infrastructure. We have also engaged BRANZ to understand our existing process with a goal of providing independent approval of geosynthetic soil reinforcement products in NZ.

GEOGRID REINFORCED BRIDGE ABUTMENTS

Section 6.6.8 of the Bridge Manual stipulates when inextensible (usually steel) and extensible (usually geogrid) reinforcement can be used for bridge abutments. We have been exploring requests to allow extensible reinforcement where the abutment is not piled, provided the bridge design can account for expected deformations. International research has shown that these geogrid reinforced bridge abutments may have wider applications where there is careful consideration of design details, such as close geogrid spacing and suitably sized backfill material. However, there remain uncertainties in the appropriate application in the NZ and NZTA context. We have undertaken a pilot on the O Mahurangi Penlink project where a range of instrumentation has been implemented. The initial results of this pilot are presented in a paper by Dr Jan Kupec et al. in this issue of the Geomechanics News and we are working on an appropriate mechanism to integrate these results into practice.

GEOTECHNICAL ASSET MANAGEMENT

Following on from Stuart Finlan's update in the September 2025 issue of the *NZ Geomechanics News*, we have been building on the foundation of our internally published Geotechnical Asset Management Framework (GAMF) and evolving our NZTA asset management practices. Structures on the NZTA network are managed regionally by our Structures Management Consultants (SMCs) and there has been an extension of SMC responsibilities to deliver geotechnical asset management across the regions. At the annual SMC Workshop, we discussed the development of our national geotechnical asset database and key geotechnical hazards for the network across the motu.

LANDSLIDE HAZARD RATING SYSTEM

The Landslide Hazard Rating System (LHRS) was approved and issued in September 2025 (Landslide Hazard Rating System (LHRS) | NZ Transport Agency Waka Kotahi), supplementing the existing Rockfall Hazard Rating System (RHRS) for consistent initial screening of rockfall and landslide hazards on the state highway network. Field applications have been developed for gathering assessment data and allowing for prioritisation of geotechnical assets for more detailed Assessed Risk Level (ARL) assessments in accordance with the New Zealand Country Amendment to NSW RMS Guide to Slope Risk Analysis.

We are also looking to plan for the next ARL training course. Watch this space!

BRIDGE & GEOTECHNICAL CONFERENCE

Planning for the next Bridge & Geotechnical conference is underway after the success of the conference last year. A date has been set for 24-25 August 2026 in Auckland. This conference will be hosted by the Bridge Engineering Technical Society (BETS) in collaboration with NZGS, with support from NZTA. Refer to advertisements for the conference in this edition of the NZ *Geomechanics News*. More details to come soon!

Report On New Zealand Geotechnical Society-Australian Geomechanics Society Geotechnical Mapping and Logging Training, Fiji

Martin Brook, School of Environment, University of Auckland;
Anthony Bowden, Bowden Geological Pty Ltd, Sydney, New South Wales;
Stephen Fityus, Douglas Partners, Warabrook, New South Wales

ABSTRACT

We outline a two-day geotechnical logging and engineering geological mapping course in Fiji. This was run for the Fiji Government's Mineral Resources Department's (MRD) geology staff. The program structure and content was developed in conjunction with the MRD staff and between members of the New Zealand Geotechnical Society and the Australian Geomechanics Society over a series of online meetings. The course was delivered in July 2025, based at the MRD offices in Suva, with field teaching at the Kasavu Landslide near Nausori, and an outcrop of Suva Marl at Maqbool Road, Suva. The first day of the course was a combination of classroom theory and experiential learning, the latter via core logging. This underpinned the second day, which was fully field-based. The field day included engineering geological mapping of the Kasavu Landslide, and geotechnical logging of the Suva Marl. This report describes the key components of the training course, and some important observations.

1. INTRODUCTION

Since late 2024, requests from Fiji-based government geologists were made to New Zealand-based engineering geologists for some training in engineering geology skills. Currently, Fiji-based geologists typically have strong core geological skills (structural geology, mineralogy, sedimentology), as taught within the University of the South Pacific (USP) BSc degree. However, the USP degree currently does not offer engineering geological courses, so many of the Fiji Government's Mineral Resources Department's (MRD) geologists do not have formal training in engineering geology, which would help them mitigate, and respond to, natural hazard and land instability issues. A series of online meetings subsequently took place in late 2024/early 2025 between New Zealand Geotechnical Society members and their Australian counterparts, with staff (led by Agnes Peter) from the MRD, Fiji. The MRD is one of the two Departments administered through the Ministry of Lands and Mineral Resources. Online meetings also took place with geologists (Gary Lee and colleagues) from a major NGO, the Pacific Community (formerly the South Pacific Commission, SPC). The latter is an international development organization governed by 27 members, including 22 Pacific island countries and territories around the Pacific Ocean (https://www.spc.int/).

Ultimately, a program was developed including classroom-based theory and experiential learning activities (including fieldwork), and delivered over two days (3-4 July 2025) in Suva. The training course was designed principally by Anthony Bowden, based on courses run through the Australian Geomechanics Society. It was delivered largely by Anthony and Stephen Fityus, with on-site assistance from Martin Brook. The field training exercises took place at Kasavu Landslide near Nausori, and an outcrop of Suva Marl at Maqbool Road, Suva, at a new residential subdivision. The training was supported by the New Zealand Geotechnical Society, the Australian Geomechanics Society, and the University of Auckland. This short paper outlines the training that was deliv ered, and some of the key outcomes and learnings.

2. CLASSROOM THEORY AND CORE LOGGING

THURSDAY 3 JULY

The course commenced with some classroom theory in the MRD Geological Survey Division's annex building (Figure 1). As outlined to the students, the goal of this training course is to provide them with the knowledge, skills and experience to allow them to successfully observe, measure and record geotechnically significant information in cores or at a site. The classroom training was given from the perspective of AS1726-2017 Geotechnical Site Investigations standards, augmented by photographs and annotated diagrams and tables to emphasize specific points. Occasionally, brief comparisons were made with the New Zealand Geotechnical Society Guidelines (NZGS, 2005), including the simplified plasticity terms in NZGS (2005), for example, which contrasts markedly with AS1726-2017. It was also outlined to the class that the NZGS (2005) is a guideline, rather than a standard, but is used extensively in New Zealand. The students were provided with a handbook that included detailed information regarding rock and soil logging and mapping symbology, a list of references to follow-up if required. They were also provided with laminated field sheets based on AS1726-2017 for field descriptions and classifications, and mapping symbology.

Of course, experiential learning is

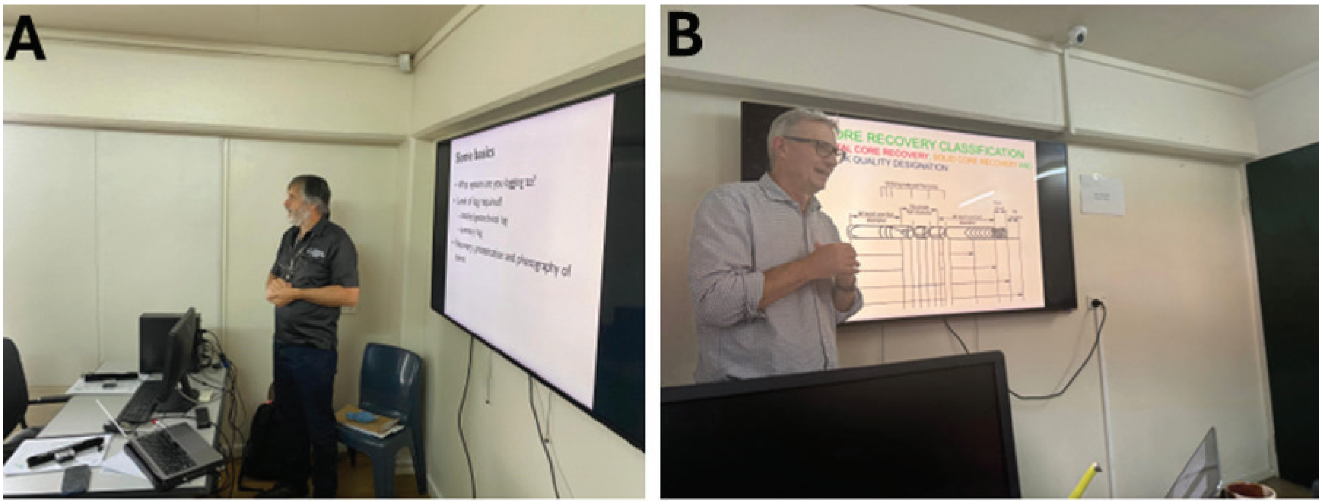

Figure 1: (A) Stephen Fityus covering soils and AS1726-2017; (B) Anthony Bowden focusing on rock properties.

a key component of geoscience, and the classroom theory was punctuated by core logging in the yard next to the Geological Survey Division's annex. This was from a range of sample core acquired from the Suva area. The students applied some of the knowledge acquired in the classroom, to the core samples, with guidance from the teaching team.

3. FIELD DAY
FRIDAY 4 JULY

The field day had two components: engineering geological mapping at Kasavu Landslide in the morning, and geotechnical logging of Suva Marl outcrop at Maqbool Road, Suva, in the afternoon (Figure 2). These two sites were chosen based on accessibility, as well as familiarity. Kasavu Landslide is an 'underslip' below the Kings Road, ~6.5 km north of Nausori. The site lies on the eastern flank of a low-lying ridge, which marks the interfluve between two adjacent catchments that are both tributaries of the Rewa River. The ridgeline is formed of weathered Upper Miocene to Lower Pliocene Waidina Sandstone, and landslides occurred in this area in 2014 and 2016.

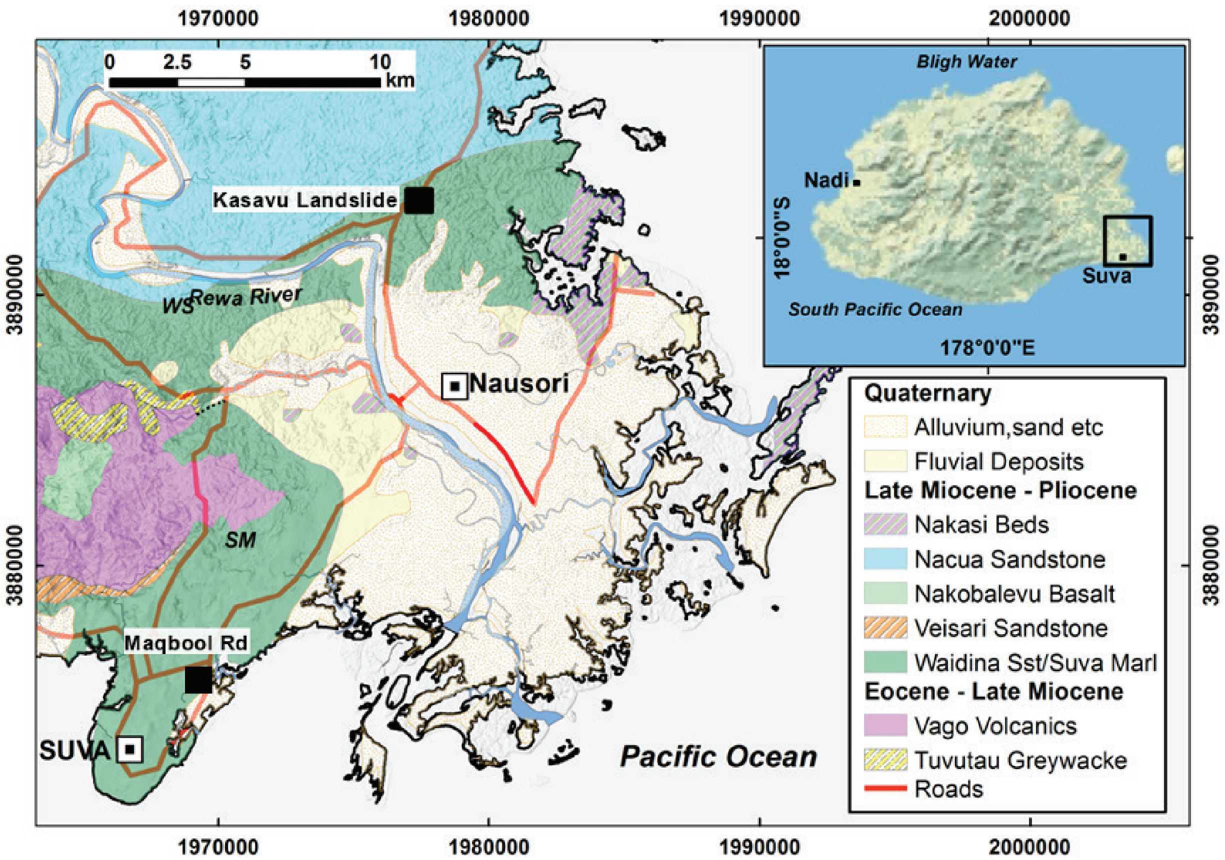

Figure 2: Geological map of southeast Viti Levu, and the field sites at Kasavu Landslide and Maqbool Road, Suva.

Figure 3: Kasavu Landslide: (A) view upslope from above toe area; (B) gabion wall under re-instated road in headscarp area; (C) break of slope below headscarp; (D) weathered Waidina Sandstone residual materials.

Investigations by Ram et al. (2019a,b) characterized the 2016 landslide as a rotational slump. The road was subsequently re-instated by the Fiji Roads Authority (FRA), and a stacked gabion wall with drainage was constructed (Figure 3). The aim of the field exercise at Kasavu Landslide was to undertake an engineering geological map of the landslide area, and to produce an annotated long-section. For this, the students were provided with a basemap and applied some of the symbology (from the Australian Standard) outlined the previous day. The 'mapping mantra' of Observe, Measure and Record provides a good framework for a methodical approach, and was reiterated to the students (Figure 3A). Additionally, the site exhibited a nice sequence of convex and concave slope profiles (Figure 3C), as well as interesting groundwater conditions and some classic landslide geomorphology (see Ram et al., 2019a). In some locations, weathered residual materials were present, which provided a further practical opportunity to investigate soil properties and understand slope-forming processes (Figure 3D).

During the mapping of the Kasavu Landslide, students had the opportunity to:

- observe the morphology of a recent landslide to gain familiarity with characteristic landslide features;
- choose a suitable map scale to facilitate the drawing of a useful map;
- practice the mapping of geomorphological features, by pacing out and recording distances between obvious breaks of slope, which is an important skill for remote field mapping.
- estimate and record local ground slopes across the site;
- record significant features such as exposed weathered bedrock, drainage lines, soft/wet ground and seepages;
- produce an approximate cross-section from a field map, to facilitate an interpretation of possible mechanisms.

In the afternoon, having briefly visited a local basalt quarry, the group undertook rock defect mapping at outcrop scale at a Suva Marl exposure in a new residential subdivision at Maqbool Road, Suva (Figure 4).

The Suva Marl is a sequence of Lower Pliocene siltstone to fine sandstone that contains 40% to 60% carbonate, and is exposed in cuttings throughout Suva (Clement et al., 1998). It is nearly

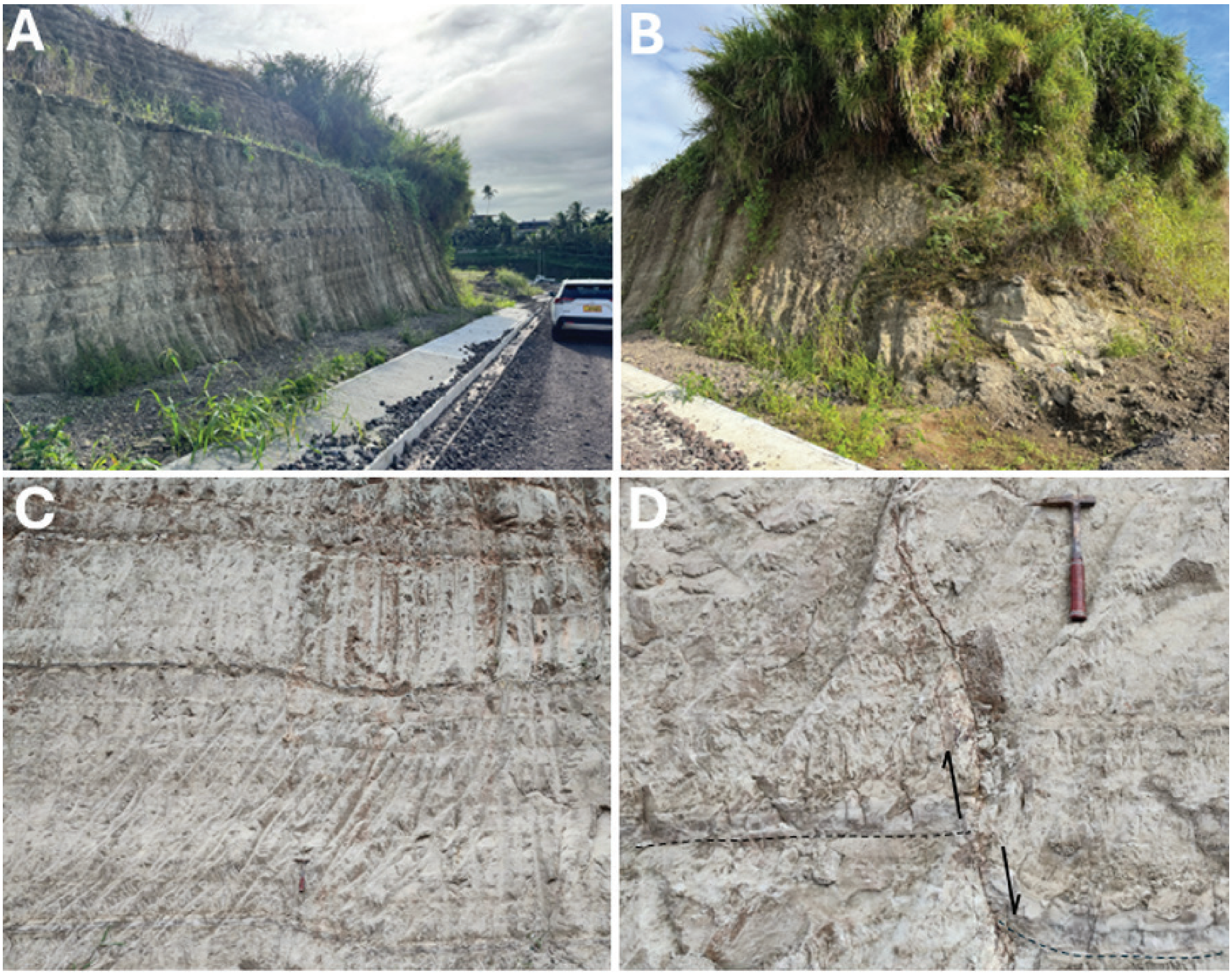

Figure 4: Maqbool Road Suva Marl: (A) the exposure used for the field class, note horizontal beds and tuff layers; (B) Suva Marl outcrop at Maqbool Road showing defects; (C) monocline and (D) normal fault (hammer for scale in each).

flat-lying, with the beds typically dipping <8°, apart from locally within the monoclines (Clement et al., 1998). Several normal faults are also present, usually with <0.5 m vertical offset. Faults and monoclines (e.g. Figure 4C, D) are typically oriented parallel to the NE-trending Suva Harbor graben (e.g. Shorten, 1993). In a basic sense, the Suva Marl is friable, and highly weathered soapstone, so the outcrop provided participants with an ideal site to describe the outcrop and classify the materials using AS1726-2017.

During the inspection, students were able to:

- Practice orientation measurement of bedding surfaces, joint surfaces and fault surfaces using a geological compass;
- Observe and classify normal faults in exposure;
- Identify slickensided fault surfaces and fault breccia material associated with faults;
- Observe faults expressing in cut faces with varying orientation, allowing consideration of how the potential of faults to contribute to rock face instability is affected by the way they are intersected in an excavation;
- Observe expressions of weathering and groundwater seepage and how these relate to the presence of rock mass features.

As outlined to the students, the Australian Standard (AS1726-2017) defines a rock defect as "a discontinuity, fracture, break or void in the material across which there is little or no tensile strength". There was ample opportunity to consider this statement and undertake the rock characterisation exercise at the Maqbool Road site (Figure 4).

4. SUMMARY AND KEY LEARNINGS

This 2-day training course for Fiji government geologists highlighted several effective teaching methods that facilitate student comprehension of engineering geological concepts. The students appeared to enjoy the course and seemed fully engaged with it. The course staff considered that the level of engagement was generally greater than that observed when the same courses are delivered to cohorts in Australia. The good weather fortunately allowed the field components to be delivered effectively. The experiential learning approaches and methods (core logging, field mapping and outcrop logging) played a crucial role in helping students understand the course material and theory. Anecdotally, students indicated that the combined effect of classroom theory, then experiential activities, was a favorable approach for their engineering geology education.

In addition, some learnings and outcomes include (but are not limited to):

1. The Fiji government geologist attendees have very strong skills in "core" geological concepts, such as minerology, sedimentology, structural geology, use of a geological compass etc.
2. Attendees were largely schooled through the USP BSc Geoscience program, which currently lacks engineering geology papers/ courses.
3. Attendees currently utilize aspects of the NZGS (2005) guidelines, and indeed many carry a laminated copy of the 2-page field sheet in the field.
4. Delivering the course material with a focus on the more detailed AS1726-2017 and its underlying principles probably enhanced their appreciation of NZGS (2005); notwithstanding some of the differences in classifications between the NZGS (2005) and AS1726-2017, the more detail provided in AS1726-2017 has probably helped in their interpretation and application of some of the basic parameters in NZGS (2005), such as the two-fold plasticity classification.
5. Linkages were made to key reference documents such as IAEG's C25, and there is potential interest in starting a Fiji IAEG national chapter, which would be useful for both government and private sector geological practitioners.
6. Interest was shown in further training courses on landslide risk assessment (AGS, 2007a etc) and engineering geological models (C25; Baynes and Parry, 2024).
7. The course was limited to two days and so attendees were directed to the NZGS and AGS websites for the myriad of information about further courses, conferences and CPD opportunities.

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ISSMGE Technical Committees TC103 and TC207

Summary of 2025 Activities

Ioannis Antonopoulos, Vice Chair, NZGS

OVERVIEW

In 2025, the ISSMGE Technical Committees continued to advance geotechnical engineering knowledge through conferences, collaborative projects, and educational initiatives. Below is a concise update on the key activities of TC103 and TC207.

TC103 – NUMERICAL METHODS IN GEOMECHANICS

TC103 had a highly active year in 2025, focusing on advancing computational approaches in geotechnical engineering.

- **Conference Contributions**: The committee played a key role in organising the Mini-Symposium MS-04 on “Numerical Simulation in Geomechanics and Geodisasters” at COMPSAFE 2025 in Kobe, Japan. This event attracted global participation and highlighted cutting-edge applications of numerical modelling for geohazard mitigation and infrastructure resilience.
- **Educational Initiatives**: TC103 strengthened its collaboration with TC306 (Geo-engineering Education) to develop open-access teaching resources that integrate numerical modelling into geotechnical curricula. These resources aim to bridge the gap between academic theory and practical application, supporting both students and professionals.
- **Governance and Future Planning**: Recognising the importance of leadership continuity, TC103 launched the chair election process in late 2025. Candidates were invited to submit vision statements outlining strategies for promoting innovation, inclusivity, and knowledge-sharing within the committee. Voting is scheduled for November 2025, ensuring a smooth transition ahead of the 2026–2029 term.
- **Strategic Focus**: The committee emphasised interdisciplinary collaboration, particularly in coupling numerical methods with experimental and field data, to improve predictive capabilities for complex soil-structure interaction problems.

TC207 – SOIL-STRUCTURE INTERACTION AND RETAINING WALLS

While TC207’s activities were less publicly visible compared to TC103, the committee maintained a strong presence in technical and collaborative domains throughout 2025.

- **Technical Engagement**: TC207 continued to provide expertise on soil-structure interaction (SSI), a critical area for the design and performance of retaining structures, foundations, and underground systems. The committee contributed to discussions on advanced modelling techniques, including nonlinear soil behaviour and dynamic loading scenarios, which are increasingly relevant for seismic-prone regions like New Zealand.
- **Collaborative Projects**: Members of TC207 actively participated in joint initiatives with other ISSMGE committees, ensuring that SSI considerations are integrated into broader geotechnical frameworks. These collaborations often focus on harmonising design methodologies and sharing best practices across regions.
- **Knowledge Dissemination**: Although no major standalone events were reported in 2025, TC207 leveraged ISSMGE platforms to circulate technical papers, case studies, and guidelines on retaining wall performance and soil-structure interaction challenges.
- **Future Outlook**: The committee is expected to play a pivotal role in upcoming ISSMGE conferences, particularly in sessions addressing resilient infrastructure and performance-based design, aligning with global trends in sustainable and safe geotechnical engineering.

KEY THEMES ACROSS ALL THREE TCS

- Strong emphasis on **conference participation and knowledge dissemination** (COMPSAFE 2025, ICSMGE 2026).
- **Collaborative sessions** between committees to address interdisciplinary challenges.
- **Leadership renewal and governance** activities to ensure continuity and engagement.
- Continued focus on **education, research, and technical innovation** in geotechnical engineering.

Project Progress Report:

Update of the AGS (2007) Guidelines for Landslide Risk Management

THE JOINT AGS and NZGS project to update the AGS (2007) Guidelines for Landslide Risk Management continues to progress well, and our 4 Working Groups (WG), comprised of experts from across Australia and New Zealand, have achieved some important milestones. We have recently provided progress updates at the 1st Australian Engineering Geology Conference (AEGC) in Brisbane (July 2025) and at the NZGS Symposium in Auckland (October 2025), and a brief summary of the project progress is provided below.

DOCUMENT STRUCTURE

The updated guidelines will be consolidated into 3 documents: (i) Principles of Landslide Risk Management, (ii) Landslide Mapping, and (iii) Landslide Risk Assessment. Each of these documents has a specific target audience and aims, but with interrelated content. The updated guidelines are intended to be complementary to other existing documents, including the NZGS Slope Stability Guidance documents and the NSW RMS Guide to Slope Risk Analysis, which is used by some roading authorities in New Zealand and Australia.

WORKSHOPS

The Landslide Mapping and Risk Assessment WGs each held workshops in Melbourne in May 2025, and the Principles WG held a workshop in Sydney in September 2025. These workshops have supported content drafting and refinement, as well as coordination among authors. In addition, the Steering Committee and WG Chairs held a workshop in July 2025 in Brisbane, coinciding with the 1st Australian Conference on Engineering Geology, where participants reviewed progress and draft content, and discussed the next steps. These workshops have proven incredibly valuable in progressing the guideline development.

WORKING GROUP 1 – INTERNATIONAL PRACTICE

We have received 12 reports from international experts which summarise landslide risk management practice in their countries. These reports have been assigned to the WGs for review, feedback, and to support their updates. In addition, a separate publication is planned to summarise these international reports. As part of the international reviews, it was noted that AGS (2007) is still very well recognised internationally and is looked to by others for guidance on landslide risk management.

WORKING GROUP 2 – PRINCIPLES OF LANDSLIDE RISK MANAGEMENT

This WG is developing general guidance on landslide risk management which is primarily intended for a broad range of non-geotechnical stakeholders, including regulators, landowners and land managers. The document is well advanced - all sections have now been drafted and are currently undergoing internal WG review.

WORKING GROUP 3 – LANDSLIDE MAPPING

The Landslide Mapping WG is developing guidelines for geotechnical practitioners for preparing landslide inventories, susceptibility, hazard, and risk maps, along with advice on how to use them for planning and landslide risk management. Some of the key updates to the mapping document, compared to the existing AGS (2007) Guidelines, will include the consideration of uncertainty in mapping, a new flow chart to set out the methodology and a more data driven approach to developing these maps.

WORKING GROUP 4 – LANDSLIDE RISK ASSESSMENT

WG 4 is preparing updated guidelines for geotechnical practitioners setting out best practice for landslide risk assessment. While the AGS 2007c document provides the foundation for the draft, several new topics and previously omitted areas have been identified for inclusion in this revision. This includes a shift from the previous residential focus to a wider range of applications, as well as additional guidance on uncertainty and probability in landslide risk assessments. The landslide risk management flow chart is being updated to incorporate these updates, and worked examples will be included to support the reader's understanding and application of the guidelines.

NEXT STEPS

The drafting of the guidelines continues, and at this stage we are targeting to have a draft completed by April 2026 to coincide with LaRGE2026 workshop in Queenstown. Once the initial drafts are completed, we will seek input from our broader interest groups from Australia and New Zealand, as well as the nominated international peer reviewers.

The drafting of the guidelines is supported by regular WG meetings, focused on refining content and integrating feedback, as well as joint Steering Committee and WG chair meetings focused on progress of the guidelines and ensuring alignment between WGs. A further workshop for the Steering Committee and WG Chairs is planned for February 2026 in Sydney to consolidate the full draft of the guidelines, prepare for external peer review and ensure consistency across the three documents.

Once again, we want to thank all those who have contributed to the project so far. Contributions, comments and feedback is welcome at any time through our queries page on the AGS website: www.australiangeomechanics.org/2024/03/05/ags-technical-committee-for-landslide-risk-management/

Event report: Developing the Next Generation of Dam Engineers Forum Think Tank 2025 (Australian National Committee on Large Dams, ANCOLD)

Melbourne, Australia, 11-12 September 2025

Dr Kaley Crawford-Flett, Senior Research Fellow, University of Auckland

FOR MANY YEARS, NZGS and the New Zealand Society on Large Dams (NZSOLD) have fostered collaboration with their trans-Tasman counterparts, the Australian Geomechanics Society (AGS) and the Australian National Committee on Large Dams (ANCOLD). These relationships play an important role in sharing knowledge and supporting the professional development of engineering and geotechnical professionals in New Zealand.

Thanks to support from NZGS, I was able to accept an invitation to speak at the *ANCOLD Think Tank: Developing the Next Generation of Dam Engineers Forum*, hosted in Melbourne on 11-12 September 2025. The two-day event brought together a passionate group of 82 attendees from Australia and abroad. The delegates included experienced and emerging professionals representing owners, government agencies, consultancies and academia.

The opening keynote by economist Adrian Hart (Oxford Economics) set a compelling scene with forecasting of an Australian ‘infrastructure wave’. By the end of the decade, spending on water infrastructure is projected to exceed $12 billion per annum. This economic backdrop provided context for two days of conversation around workforce development and retention. The Australian dams sector is buoyant and growing, and reliable resourcing is crucial.

The forum programme was structured around three themes, including presentations and interactive discussions:

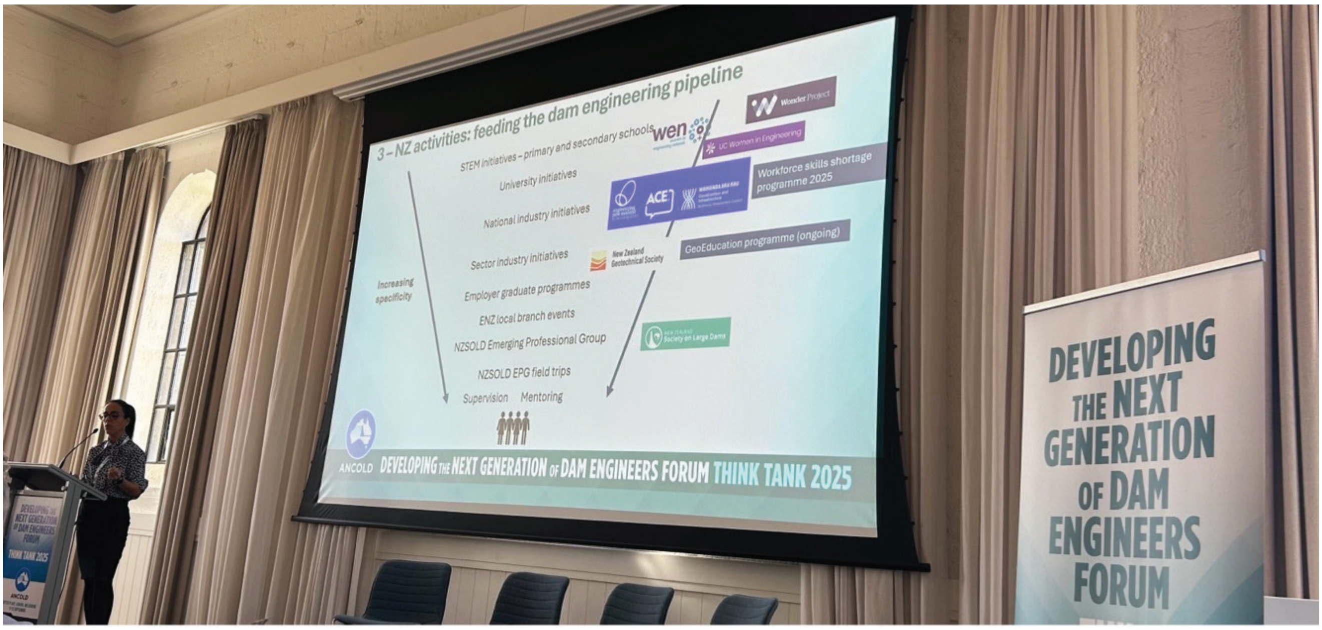
1 - ADDRESSING THE SHORTAGE OF DAM ENGINEERS

Presentations included data on industry signals on skills shortages, ANCOLD member survey results, pathways into the sector, and the role of university education. I presented on the NZGS GeoEducation action plan and NZSOLD Emerging Professional initiatives, in a broader context of the Engineering New Zealand Workforce Skills Shortage project and other New Zealand initiatives. This theme introduced the ‘workforce pipeline’, from school outreach to ongoing professional development.

2 - INTERNATIONAL AND CROSS-INDUSTRY INITIATIVES FOR CAPABILITY BUILDING AND NEXT GENERATION DEVELOPMENT

Keynotes included Natalie Currey (Australasian Railway Association) on diversity in rail, and Del Shannon (Knight Piésold, USA) on attracting the next generation of dam engineers. Kim Morrison (ATC Williams) shared reflections on a 30 year career in tailings engineering.

This theme covered global capacity-building efforts, including successes from Spain, the Americas, and Africa. Examples of successful industry-focused training and education programmes (e.g. Master’s degrees) were discussed, along with the challenges in their long-term viability. Sustaining these programmes requires ongoing investment, which proves challenging due to uncertainty in infrastructure forecasting (boom-bust cycles), demands of project lifecycles, and changing political drivers.



3 - HOW DO WE ENHANCE THE KNOWLEDGE AND SKILLS OF DAM ENGINEERS?

Bernadette Foley (Engineers Australia) provided an overview of the role of Engineers Australia in reporting on the demographics of the engineering profession, responding to diversity challenges, decoding career pathways, and strengthening the Australian engineering workforce. Presentations from early-, mid-, and late- career dam engineers provided examples of lived experience in the sector, along with reflections on changing technologies and employee expectations.

Despite clear differences between Australian and New Zealand infrastructure spending forecasts, forum outcomes were equally relevant to the New Zealand engineering and geoprofessional sectors:

- The forum highlighted ongoing challenges around boom-bust cycles, fragmented delivery of projects, outsourcing, knowledge transfer, and stewardship of both physical infrastructure and experience.
- There is a need to improve visibility of the sector across our society and education systems: from public awareness to STEM-specific (school/university) outreach.
- Throughout the world, we face challenges in sustaining industry-focused (postgraduate) training and education programs. Shifting industry demands often limit partners' ability to commit the long-term funding necessary to sustain training centres.
- We need action in both top-down and bottom-up directions. While individuals can't control the national infrastructure pipeline, we can all be better colleagues to those around us and invest time in junior colleagues.

Many presentations at the Forum focused on a specific 'stage' of the workforce pipeline. It was pleasing to reflect that the NZGS Geoeducation Action Plan covers all aspects of the workforce pipeline discussed at the Forum, from public awareness through to workforce training and mentoring.

The Forum highlighted action items that we can consider in New Zealand, at the individual, technical society, corporate, and university levels:

- **As individuals:** Never underestimate the individual impact you can have as an industry professional – and the 'snowball effect' of showing an interest in those around you. Time is our greatest asset, and even small investments matter. A coffee or lunch with a junior colleague can change a career trajectory.
- **Industry, universities, and technical societies:** Industry and technical groups must continue to foster relationships with universities. We should celebrate the involvement of academics on the NZGS management committee and working groups.
- **Consultancies and clients:** Should consider how projects are structured and incorporate training as a non-negotiable. Are there opportunities for secondments on projects to provide training opportunities?

ANCOLD is in the process of documenting key outcomes from the Forum and will use these outcomes to form a pathway to strengthen the future dam engineering workforce. We look forward to reading ANCOLD's post-forum report.

Eurock 2025, Trondheim, Norway

16 – 20 June 2025

Romy Ridl, KiwiRail, Eleni Gkeli, Stantec, Christoph Kraus, Beca

Figure 1. The Nidaros Cathedral in Trondheim

Figure 2. Organ concert at the Nidaros Cathedral in Trondheim

1 INTRODUCTION

From 16 to 20 June, the Norwegian Group for Rock Mechanics hosted the Eurock 2025 conference in the beautiful city of Trondheim, Norway. Eurock is the annual regional ISRM conference for Europe, and this year the Eurock conference was also the ISRM International Symposium for 2025. The conference was attended by about 400 delegates.

The ISRM Council meeting was also held as part of the conference. With the support of Tourism New Zealand, Christchurch NZ and the NZGS, Eleni Gkeli, Romy Ridl and Christoph Kraus attended the Council meeting to present the NZGS proposal to host the ISRM Congress in 2031. Consequently, the same team attended the conference to continue promoting New Zealand and the NZGS, and to run the NZGS exhibition booth. A summary of the conference is provided below.

2 COUNCIL MEETING AND BID TO HOST THE ISRM2031

The ISRM held its 2025 Council meeting on 16 June ahead of the Eurock2025 conference. 51 National Groups were represented at the Council. New Zealand was represented by Eleni Gkeli who is the NZGS ISRM liaison, Romy Ridl and Christoph Kraus attended as observers. The Council Meeting provided us with many useful insights to the initiatives and activities of the ISRM and other national societies, and we were able to make many connections with rock mechanics experts from around the world.

As part of the Council meeting the three of us presented the NZGS proposal to host the 17th International Congress of the ISRM in Christchurch

Figure 3. Access tunnels at the Leirfossene Hydro Power Plant

in September 2031, which was the main purpose of our attendance at the conference. To support our proposal to host the ISRM congress, we also had an NZGS booth in the exhibition area for the duration of the conference thanks to the support of Tourism NZ and Christchurch NZ.

For more details about the NZGS proposal to host the ISRM2031 Congress and our booth at the conference, please refer to the ISRM report in this issue of *Geomechanics News.*

3 CONFERENCE

The conference began with a welcome ceremony. The ceremony was followed by the Rocha Medal Lecture (awarded for an outstanding doctoral thesis) delivered by Dr Lucille Carbillet, and the Franklin Lecture (recognizing a mid-career ISRM member who has made a significant contribution to a specific area of rock mechanics and/or rock engineering) which was delivered by Dr Charalampos (Harry) Saroglou. Throughout the conference there were seven excellent keynote lectures, and lots of great presentations on a variety of topics including tunnelling, geohazards, rock mass monitoring, 3D modelling, laboratory testing and much more. The papers presented at the conference were of high quality and it was great to see the projects and research being completed in Europe and further abroad.

The conference also hosted the 10th Early Career Forum. As part of the forum, six young rock engineers from European and North African countries, who otherwise would not be able to attend the conference, were invited to attend the conference and present their work. It was great to see the ISRM supporting the development of young professionals from around the globe! The conference also included the traditional ISRM RockBowl competition, with eight teams of young professionals and students competing.

The main conference programme spanned over three days and on each of the two evenings, the conference organisers hosted social events to facilitate networking among delegates. On the Tuesday evening, there was an organ concert at the Nidaros Cathedral, the world's northernmost medieval cathedral and a famous landmark in Trondheim. In addition to the organ concert, the hosts also provided background on the history and construction of the cathedral. The banquet dinner followed on the Wednesday night and included a local cultural performance.

During the conference, awards were also presented to the Institute of Rock and Soil Mechanics, Chinese Academy of Sciences (Technological Innovation Award 2025), the French and South African ISRM national groups (awarded joint best ISRM National Groups for 2022–2024) and Ignacio Pérez Rey from the University of Vigo, Spain (Young Rock Engineer Award 2025).

4 CONFERENCE FIELDTRIPS

Two fieldtrips were offered following the conference, visiting either the traditional mining town of Røros (a UNESCO's list of cultural heritage site), or the local Leirfossene Hydro Power Plant on the outskirts of Trondheim.

The fieldtrip to the Leirfossene (which translates to clay waterfalls) Hydro Power Plant provided great insights to tunnelling practices and renewable electricity generation in Norway. Leirfossene is an underground plant located along the lower part of the Neavassdraget watercourse (Nidelva River), and its two turbines have a combined capacity of 45 MW and an annual average production of just under 150 GWh. Water enters the power plant via a shaft and tunnel from the intake reservoir water and is discharged again through a ~1.5 km long outlet tunnel to Lake Nedre Leirfoss. It is interesting to note that the two older plants, which the Leirfossene Plant was built to replace, were not demolished but were instead converted into smaller power plants, which utilize the smaller flow down the Nidelva River adjacent to the Leirfossene Power Plant tunnels. During the fieldtrip we were able to walk down one of the access tunnels and visit the cavern of the generator hall. The local Statkraft staff provided excellent explanations of the power plant, construction of the tunnels and power plant, as well as the local geology.

5 ACKNOWLEDGEMENTS

We are very grateful to Tourism New Zealand and Christchurch NZ for their amazing and ongoing support for the NZGS to host the ISRM2031 congress. We also want to acknowledge the Eurock 2025 conference organisers for hosting a fantastic and well-run conference, and the Statkraft staff for the insightful fieldtrip to the Leirfossene Power Plant.

NZGS 2025 Symposium

Geotechnical Horizons: Innovations & Challenges

Emilia Stocks – Symposium Convener

BUILDING A SAFER, stronger Aotearoa requires collaboration, innovation, and a shared vision. From 15 to 18 October 2025, that vision came to life at the 2025 New Zealand Geotechnical Society (NZGS) Symposium in Auckland. The event united over 500 delegates at the Aotea Centre for a dynamic exchange of ideas and insights, capturing the forward-looking spirit of a profession ready to tackle New Zealand's greatest geotechnical challenges head-on. The overwhelmingly positive feedback received from attendees has affirmed the event as a resounding success.

The main sessions of the Symposium took place in the basement of the Aotea Centre, a location that could hardly have been more fitting for a gathering of geotechnical professionals. This light-hearted fact framed a serious theme: "Geotechnical Horizons: Innovations & Challenges", an exploration of how the profession continues to adapt to change, integrate new technologies, and meet the demands of a complex and evolving natural environment.

The theme gained particular resonance in the context of recent national events. Following the 2023 Auckland storms and Cyclone Gabrielle, geotechnical engineers have been once again at the forefront of addressing the challenges posed by climate change and natural hazards.

In-Situ Testing Practical Workshop

Slope Stability in Practice Workshop

Tier 2 Rapid Building Assessment Training

Earthworks: Theory to Practice Workshop with Dr Burt Look

Welcome Function

PRE-SYMPOSIUM WORKSHOPS AND WELCOME FUNCTION

The Symposium began on Wednesday, 15 October, with a suite of Pre-Symposium Workshops run by experts from industry and academia. The workshops included:

- *Earthworks: Theory to Practice Workshop* – Dr Burt Look (Australia), with a NZ perspective provided by Ayoub Riman.
- *Slope Stability in Practice Workshop* – Richard Justice, Eleni Gkeli, Razel Ramilo, Alan Wightman, Tom Revell, and Naomi Norris, supported by the Natural Hazards Commission.
- *Tier 2 Rapid Building Assessment Training* – Rori Green and Jeremy Neven.
- *In-Situ Testing Practical Workshop* – Robin Power and Dr David Lacey (both from Australia), with field demonstrations held at the Parnell Cricket Club.

These workshops were oversubscribed, drawing approximately 115 participants and setting a tone of enthusiasm and technical depth that carried through the week.

That evening, the Welcome Function at Wynyard Pavilion offered a relaxed start to the event. A chance to reconnect over canapés and harbour views while applauding the generous support from our sponsors, without whom the event would not have been possible.

BELOW THE SURFACE: THE MAIN SYMPOSIUM UNFOLDS

Over the next two days, the Aotea Centre became the beating heart of New Zealand's geotechnical dialogue. A memorable moment came when the entire symposium took part in the New Zealand ShakeOut earthquake drill, a practical reminder of the country's seismic hazards.

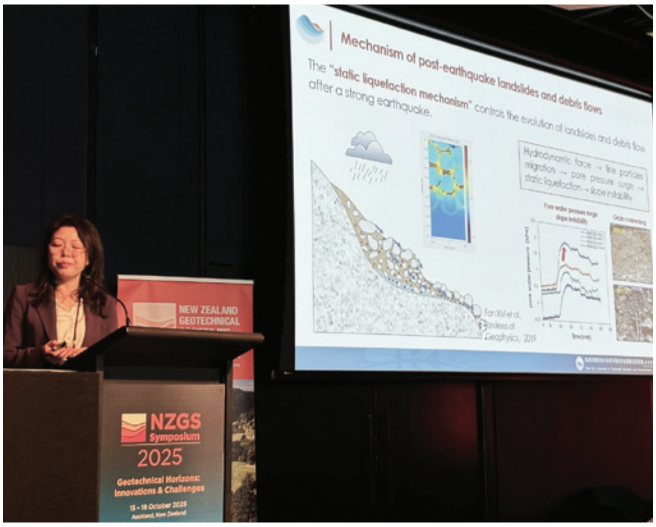
The programme featured four keynote presentations:

- Professor Xuanmei Fan – *Earthquake- and Climate Change-Induced Cascading Hazards: Mechanism and Prediction*
- Professor Jan Evans-Freeman – *Building a Sustainable and Resilient Future*
- Professor Kyle Rollins – *Liquefaction-Induced Downdrag and Dragload from Full-Scale Blast Liquefaction Testing*
- Dr Burt Look – *Managing Engineering Uncertainty*

Complementing these keynote presentations were nearly 80 oral and 31 poster presentations on topics ranging from landslide risk management to future-focused insights into machine learning.

ShakeOut earthquake drill

Ross Roberts' presentation on "Auckland 2023 Storm Response: Successes, Challenges, and the Road to a Better Recovery"

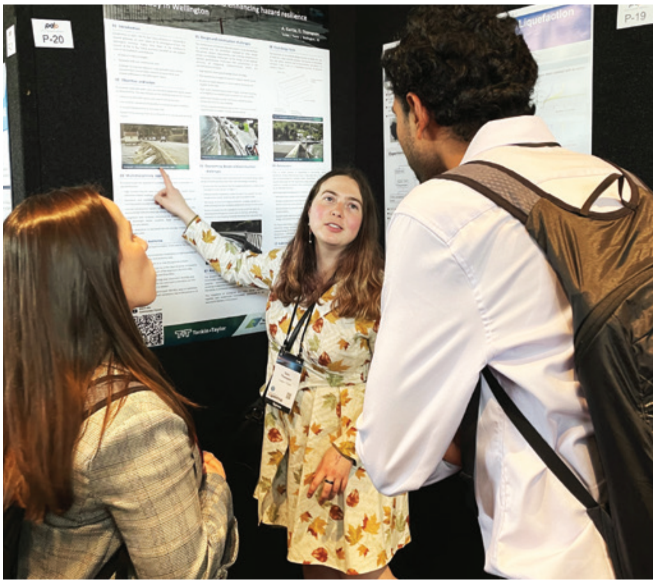


Keynote: Prof. Dr. Xuanmei Fan's presentation on Earthquake- and Climate Change-Induced Cascading Hazards: Mechanism and Prediction

Keynote: Prof. Kyle M. Rollins' presentation on Liquefaction-Induced Downdrag and Dragload from Full-Scale Blast Liquefaction Testing

Keynote: Dr Burt Look presentation on *Managing Engineering Uncertainty*

Panel discussion on Geotechnical Design and Compliance: Making sense of the NZ Building Code, TS1170.5, and other guidance



Poster presentation session

Oral presentation sessions

Chaired by Ann Williams, the first panel discussion delved into how to best equip engineers for a sustainable and resilient future. Panellists Professor Jan Evans-Freeman, Ross Roberts, and Nick Wharmby examined the ideal timing for sustainability education and debated the effectiveness of frameworks like the UN Sustainable Development Goals. The session was less about offering immediate solutions and more about establishing the critical mindset needed to navigate a sustainable future.

The second panel discussion, chaired by Dr Luke Storie, focused on the major shifts in New Zealand's seismic design regulations following the release of the 2022 National Seismic Hazard Model. The panel of experts with Dr Kaley Crawford-Flett, Professor Ken Elwood, Stuart Palmer, Kiran Saligame and Rick Wentz, covered key topics including updates to seismic design standard TS1170.5, amendments to the Earthquake Geotechnical Engineering Practice Series and the Earthquake-Prone Building framework.

CELEBRATING ACHIEVEMENT AND COMMUNITY

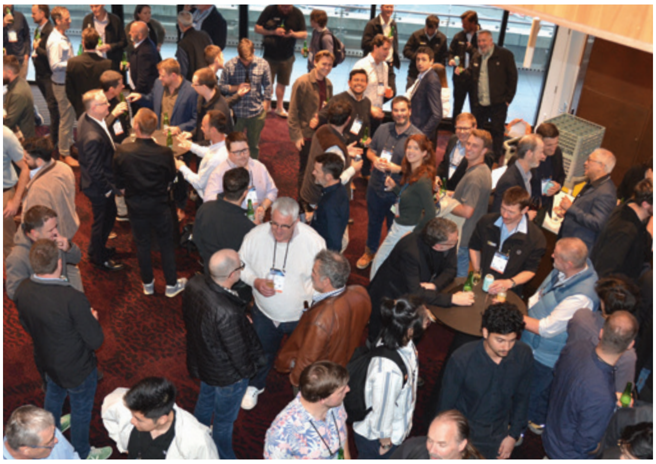
The Gala Dinner on Thursday evening was a full-house event featuring guest speaker Nu'uali'i Eteroa Lafaele, co-founder of Fibre Fale and 2025 Kiwibank Young New Zealander of the Year. Her talk, “Resilience Beyond the Blueprint,” drew on powerful themes of inclusion, empowerment, and leadership.

The evening also celebrated lifetime achievement and notable contributions to New Zealand geotechnical profession, with the presentation of the 2025 NZGS Geomechanics Lecture Award to Professor Rolando Orense.

That night also saw the presentation of the NZGS 2025 Best Paper Awards, recognising excellence across research, practice, and student contributions.

- **Best Research Paper:** *PGA Adjustment Factors for TS1170.5 to Account for Nonlinear Site Response on Soft Soils* by C.A. de la Torre, M. Cubrinovski, B.A. Bradley & S.S. Bora (University of Canterbury & GNS Science).
- **Best Practice Paper:** *Under the Mountain – City Rail Link, Mt Eden Tunnel Portal Temporary Retaining Structure Design and Construction Challenges* by S.A.B. Farquhar & Y.F. Thorp (Tonkin + Taylor Ltd).
- **Best Student Papers (Joint First Prize):**
  - *Liquefaction Characteristics of Gravelly Soils Prepared by Water Sedimentation Method* by L. Wang, G. Chiaro, S. Rees, C. Cappellaro & A. Pokhrel (University of Canterbury).
  - *Simplified CPT-Based Liquefaction Ejecta Severity Model Using Christchurch Data* by K.M. Azul, R.P. Orense & L.M. Wotherspoon (University of Auckland).
- **Best Student Poster:** *Liquefaction Characteristics of Gravelly Soils Prepared by Water Sedimentation Method* by L. Wang G. Chiaro, S. Rees, C. Cappellaro & A. Pokhrel (University of Canterbury).

A dedicated session on Friday afternoon honoured the life and legacy of the late Professor Mick Pender. Tributes and memories of his significant contributions to research and the profession were shared by Professor Liam Wotherspoon, Professor Rolando Orense, Arman Kamalzadeh, Dr Mark Stringer, and Dr Luke Storie. It was a touching and fitting tribute to a giant in the field.



Pre-dinner drinks and networking

Gala dinner and networking

Dinner invited speaker Nu'uali'i Eteroa Lafaele

Professor Rolando Orense 2025 NZGS Geomechanics Lecture award and Kristian Azul from University of Auckland Best Student Papers

FIELD TRIPS: LEARNING FROM THE LANDSCAPE

The symposium concluded on Saturday, 18 October, with two field trips rich in learning and spectacular New Zealand scenery.

The “Magmatic Mysteries: The Secrets of Rangitoto” field trip, led by Professor Jan Lindsay, explored the volcanic foundations of Auckland. Attendees trekked across rugged lava flows, inspected ancient scoria cones, and examined the city’s geologic youth through an up-close encounter with Auckland’s newest volcano. On Kepa Road, Professor Brook guided the group through the complex landslide zones of Ōrākei Basin, where deep-seated translational failures continue to shape the urban environment.

Running concurrently, the “Slip ’n’ Slide: The Chronicles of Ground Movement” field trip, led by Dr Bruce Hayward and Willy Roberts, took delegates on a tour with stops at Takapuna Beach, where fossilised kauri trunks encapsulated in lava testify to the region’s volcanic past, and Exhibition Drive, where recent landslide remediation works following the 2023 storms offered real-world insight into the challenges of stabilising steep urban terrain. At the Arataki Visitor Centre, Ross Roberts, Auckland Council’s Chief Engineer, shared stories from the city’s 2023 emergency response.

A PROFESSION ON STEADY GROUND

Once again, I would like to thank the organising team, speakers, sponsors, and attendees for the energy, curiosity, and good humour that made this symposium so memorable.

As I mentioned in my closing speech, while the challenges facing our profession are real, so too are the expertise and passion within it.

From the humour of its subterranean venue to the richness of its technical programme and the inspiration of its fieldwork, the 2025 NZGS Symposium was more than a symposium, it was a collective statement about the power of connection, collaboration, and shared purpose. This was clearly reflected in the post-event survey results, which showed that networking was considered the most valuable aspect of this Symposium.

Let’s keep pushing boundaries, collaborating across disciplines, and engineering a safer, stronger Aotearoa New Zealand.

Learning from Professor Martin Brook on Kepa Road landslide history and geomorphology.

Learning from Professor Jan Lindsay on Rangitoto Island's eruption history

Field trip attendees of 'Magmatic Mystery' at the summit of Rangitoto

"Slip 'n' Slide: The Chronicles of Ground Movement" field trip with the Rangitoto in the background

Dr Bruce Hayward talk at Campbells Bay Beach

Arataki Visitor Centre, Ross Roberts shared stories from the city's 2023 emergency response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Under the Mountain – City Rail Link, Mt Eden tunnel portal temporary retaining structure design and construction challenges

S.A.B. Farquhar & Y.F. Thorp
Tonkin + Taylor Ltd, New Zealand

Simon Farquhar, Tonkin + Taylor

NZGS SYMPOSIUM WINNER: Best Practice Paper

ABSTRACT

The City Rail Link (CRL) is the largest transport infrastructure project ever to be undertaken in New Zealand. It comprises a 3.45 km twin-tunnel underground rail link up to 42 m below the city centre. The Link Alliance are delivering the design and construction of two new stations, Te Waihorotiu and Karanga-a-Hape, redevelopment of the Maungawhau (formerly Mount Eden station) and bored twin tunnels between Maungawhau and Te Waihorotiu. This paper discusses the portal retaining wall at Maungawhau, where the tunnel boring machine started its journey to Te Waihorotiu station.

The portal wall is a complex reinforced concrete piled retaining structure up to 28 m high. In the order of 100 ground anchors provide stabilising tie back forces at four levels. 3D modelling was required to ensure no interaction between the bond lengths of the overlapping anchors, the tunnels, and the tightly constrained project boundaries. Three tunnels pass below the wall. The design was complicated by the presence of uncemented sandstone that was encountered at the mined tunnel face level, a 1.3 m diameter watermain that supplies a large area of Auckland inner city running just behind the wall, a street behind the wall that remained open for much of the construction period and a vibration sensitive television studio filming during construction across the road. This paper describes the design of the wall and monitoring results through excavation, tunnel mining and backfilling phases of the wall. Instrumentation includes inclinometers, surveyed surface prisms and ground anchor load cells.

1 INTRODUCTION

The $5.5 billion City Rail Link project is ambitious. When it is fully operational, 54,000 passengers an hour will use CRL stations at peak times. This is equivalent to 16 lanes of road or three Auckland Harbour Bridges. Auckland rail capacity will at least double when CRL is fully operational. The project involves construction of twin 3.5 km long tunnels linking the Waitematā Station downtown (formerly Britomart Station) with the Maungawhau station (formerly Mt Eden Station) on the outskirts of the CBD. Two new underground stations have been constructed, Te Waihorotiu Station (located

FIGURE 1: Mt Eden tunnel portal temporary retention structure with TBM Dame Whina Cooper in foreground.

near Aotea Square) and Karanga-a-Hape Station (located near Karangahape Road). The project also includes wider rail networks upgrades. The Link Alliance (City Rail Link Ltd, Vinci Construction Grands Projets S.A.S, Downer NZ Ltd, Soletanche Bachy International NZ Limited, WSP New Zealand Limited, AECOM New Zealand Limited and Tonkin + Taylor Limited) are delivering a large portion of the CRL works.

The Mt Eden tunnel portal temporary retention structure (hereby referred to as the portal wall) supported the excavation at the transition between cut and cover tunnels and tunnels constructed using tunnel boring machine (TBM) and mined construction methods. Design and construction of the portal sat on the project program critical path. Dame Whina Cooper, the Tunnel Boring Machine doing the bulk of the tunnelling work, was arriving in NZ in October 2020 and the portal wall and associated mined tunnels had to be ready for tunnelling to commence in April 2021. Detailed design began in May 2019 creating a tight design and construction programme. Covid-19 further added to the challenges faced by the project team as New Zealand entered a lockdown during early |piling works.

This paper outlines the geotechnical design and construction challenges faced by the project team and assesses the design performance through monitoring.

2 GEOLOGY

2.1 OVERVIEW

The geology in the Mt Eden Portal and Station area is dominated by the East Coast Bays Formation (ECBF) ridge to the north along Mt Eden Road and Maungawhau (Mt Eden) volcano to the southwest. The area has a mantle of volcanic ash in the north, valleys infilled by alluvium to the east and west with basalt tongues to the south underlain by ECBF. A thick (up to 14 m) layer of basaltic ash is located on the top of Newton Hill but thins on the side slopes and is entirely absent on the lower slopes. Alluvium infilled paleo valleys dip towards the west and south.

At the portal wall location, a surficial layer of fill up to 3m thick covers the site. The fill overlies a lens of about 1.5m thick volcanic ash (VA) which in turn overlies a thin (<1 m thick) layer of stiff Tauranga Group alluvium (TA1), overlying ECBF. The completely to moderately weathered ECBF (EW) profile is about 8 m thick overlying the slightly to unweathered rock (EU2) at depth. Numerous beds of ‘uncemented’ sandstone (EUs1), ranging in thickness from 0.2 to 4.0 m thick were logged in several boreholes around the Portal, as shown in the geological cross section in Figure 1 below. Standard Penetration Tests (SPT) in the uncemented sandstone were N=50+, however the borehole core had the consistency of dense sand.

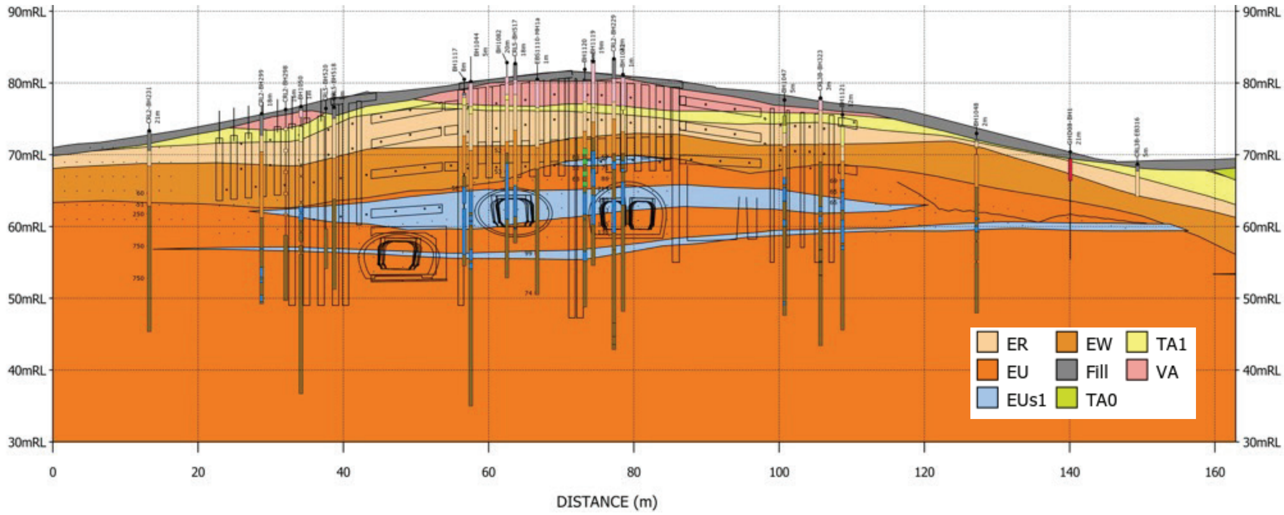

Figure 2: Geological model used for design through section cut along portal wall alignment, uncemented sandstone layers shown in blue.

The groundwater regime consisted of a regional water table within the ECBF rock at approximately 57 m RL (near the base of the excavation). A series of cascading perched 'leaky' aquifers were present in the surficial soil layers and weathered rock. The portal structure was designed as a drained structure.

2.2 UNCEMENTED SANDSTONE

Uncemented sandstone within ECBF rock has been encountered in projects throughout Auckland (Roberts, 2015). The geotechnical unit 'uncemented sandstone' (EUs1) first arose in the project Interpretative and Baseline Reports prepared by Aurecon. Kirk et al. (2021) discuss the development of the unit to describe sands that are difficult to recover in boreholes but typically test as very dense with SPT N values greater than 50. Typically, beds of uncemented sandstone within ECBF are thin, 70% of the beds measured from boreholes on CRL being <1m thick. The investigations at the portal, however, found that beds up to 4 m thick were present.

During the early design phase there were concerns regarding the stability of the uncemented sandstone in cut faces and the bond strength of ground anchors founded in this material. We found that the material behaved better than expected. The title 'uncemented sandstone' conjures up images of flowing sands unable to support angles steeper than 30°. However, this did not prove to be the case during construction. In this instance the title uncemented sandstone may have been a misnomer. Despite this we decided to keep the geotechnical unit label the same through design to avoid confusion.

Poor core recovery makes testing of the uncemented sandstone difficult. It is believed that drilling disturbance led to this geotechnical unit being classed as uncemented. Grain interlock creates apparent cohesion, especially under confined conditions. It is possible to sample intact core in this geotechnical unit with the aid of a high-quality experienced driller. Intact samples can be peeled with a knife and easily broken by hand. Figure 3 shows an example of the uncemented sandstone recovered from a borehole at the portal wall.

Several samples were recovered and tested during the detailed design phase. UCS testing was not possible as the samples easily broke when unconfined. CIU triaxial tests were undertaken on samples of uncemented sandstone, the results are shown in Figure 4, alongside testing of other ECBF units. The results were treated with caution as sampling bias was suspected, with stronger samples being recovered and weaker samples lost during drilling.

The results of lab testing indicated that EUs1 was

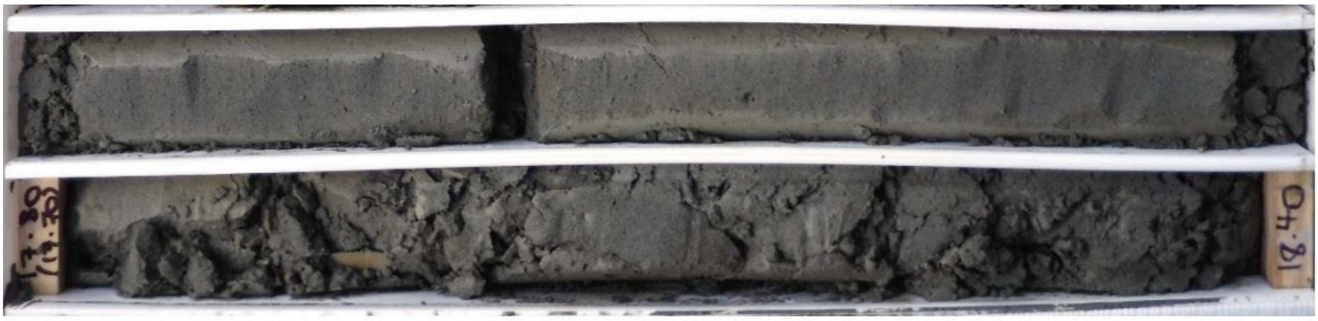

Figure 3: Core photograph of uncemented sandstone (EUs1) in BH517 from 17.0 to 21.3m

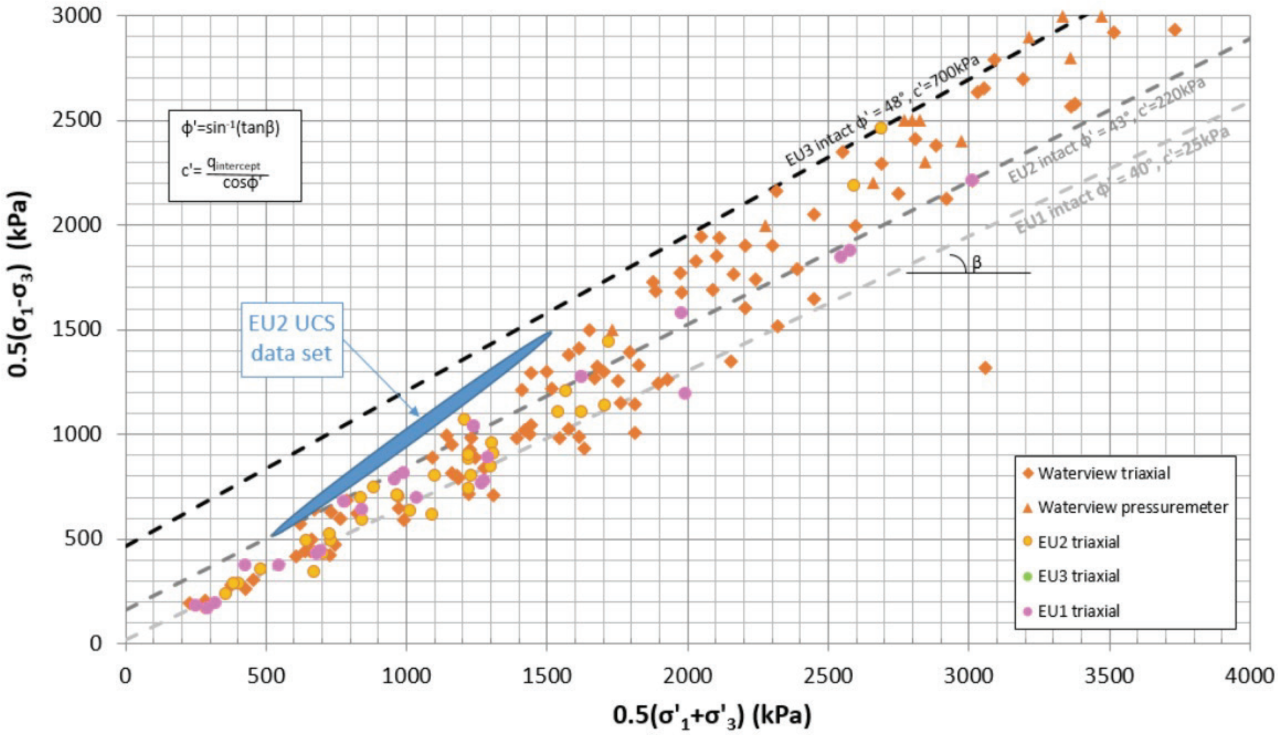

Figure 4: Triaxial MIT plot of ECBF rock units reproduced from Graafhuis (2020)

weaker than the typical lithology slightly to unweathered interbedded siltstone sandstone (EU2). However, the differences in strength were not as significant as initially thought. This paper does not cover the full extent of testing of the EUs1 unit. Only a sample of results are presented here.

In-situ testing showed comparable results to the lab testing, with pressuremeter testing undertaken in the EUs1 typically resulting in lower moduli values to that of EU2.

2.3 GROUND ANCHOR TESTING IN UNCEMENTED SANDSTONE

A series of four ground investigation load tests were undertaken on sacrificial anchors installed to test the grout-to-ground bond adhesion strength in the EUs1 geotechnical unit. Testing of bond strengths in well-cemented ECBF typically exceed 1000 kPa. However, the design team thought that the bond strength in the EUs1 unit may be lower than 1000 kPa due to the lower degree of cementation. The bond strength in the anchor testing indicated bond strengths more than 1500 kPa. Table 1 presents the results of the testing.

Tests were undertaken on vertical test set-up with a reaction pad and on an inclined test set up using the portal waler beam as a reaction frame. The tests did not indicate any statistically significant differences between the inclined and vertical test anchors. However, the sampled size is small, and two tests were not able to fail the bond as the maximum allowable test load was reached.

Table 1: Ground anchor test summary

Test number	T1	T2	T3	T4
Free length (m)	11.5	21.0	22.0	11.3
Inclination to horizontal (°)	90	35	35	90
Termination criteria	Pullout	Pullout	Maximum test load reached, plastic deformation in last load stage	Maximum test load reached, no plastic deformation
Maximum bond strength proven (kPa)	1814	1565	1772	2002

The bonded length in the test anchors was 3 m long and 150 mm diameter. Design bonds used were up to 10 m long and 200 mm diameter. To account for differences in the test conditions and the design conditions, the bond strength adopted for design anchor bond lengths was conservatively reduced by efficiency factors recommended by Barley (1997). The authors were unable to find published research into bond length efficiency in extremely to very weak rock similar to the conditions on site. The industry would benefit from further research on this topic; however, we acknowledge that the tests would be difficult to perform due to the loads (more than 5 MN) required to fail longer bond lengths in rock.

All production anchors were load tested up to 150% of the design serviceability (working) load. No issues were encountered during testing of production anchors. Anchors were prestressed to a specified lock-off load to limit wall deflection. Ten anchor load cells were installed in the headworks of the production anchors the results of which are discussed in Section 4.4.

3 DESIGN CHALLENGES

3.1 SETTLING ON AN ALIGNMENT

The design of the temporary tunnel portal retention structure was driven by several challenging design and construction constraints. Often, the solution to one constraint exacerbated the effect of another. The first design issue to overcome was the location of the portal structure itself. The specimen design originally included a smaller retaining wall located approximately 10 m further southwest along the tunnel alignment from the final design position. An existing stormwater shaft consisting of a ring of bored secant piles was located at the corner of Flower Street and Nikau Street (Fig. 5). One of the proposed mined tunnels clashed with the existing shaft piles. The entire portal structure was moved north to avoid tunnelling through the shaft. This enabled the stormwater shaft to be excavated and demolished in a top-down approach.

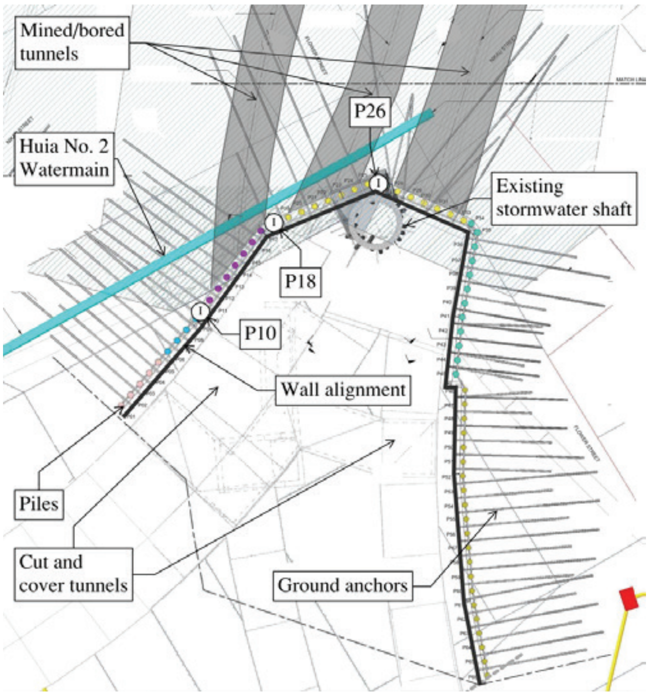

Figure 5: General layout of portal wall (I) = inclinometer

The new location of the portal structure came with its own set of unique challenges. The Huia No.2 watermain, a 1.35m concrete lined steel pipe that supplies water from the Huia treatment plant in Titirangi to the Khyber Pass Auckland reservoir runs along Nikau Street. The portal piles would come within 1.6m of the Huia No 2 and 700 mm of the cut and cover tunnels. Allowable construction tolerances on particular piles were more onerous than standard to ensure that clashes did not occur.

Limiting deflection of the watermain to prevent damage, or the need to relocate the watermain became a significant driver in the design. The retaining walls were now closer to existing buildings to remain during and after construction. Notably the Mediaworks building at 3 Flower Street within which live filming would take place during construction.

The new location of the portal structure also created a wider, open excavation. What could previously be described as a trench, now resembled an amphitheatre. The internal propping proposed in the reference design was no longer feasible, and inclined ground anchors were now required to support the retaining wall. The use of ground anchors had the added benefit of removing the obstructions that multiple props would have created in the excavation.

3.2 HANGING PILES

The retaining wall itself consists of a perimeter of bored concrete piles, 750 or 900 mm in diameter, with typical centre to centre spacing of 2000 mm. The soil between the piles was supported by a sprayed concrete (shotcrete) arch with steel mesh reinforcement. Three tunnel portal openings pass through the portal retention structure, MC50, MC20 and a combined MC30/60. The piles above the tunnel portals terminate above the crown of the tunnel, these 'short' or 'hanging' piles penetrate only 1 m into ECBF rock. The tunnel faces were supported by rows of glass reinforced polymer (GRP) rock bolts and shotcrete face. The shotcrete was reinforced with further GRP bars and PP fibre inclusions. The GRP bars and fibre reinforced shotcrete could be mined through without obstruction.

It was predicted that the short piles could exert additional vertical stresses onto the crown of the tunnel. The stresses develop from the vertical component of the inclined ground anchor tension forces, back of wall friction and self-weight of the structure (Fig. 6). The initial vertical stress at the toe of the pile was calculated to be in the order of 300 kPa, during the portal construction this was predicted to increase to 1200 kPa. Designing the tunnel primary lining to fully resist these stresses was not desirable and the portal structure was

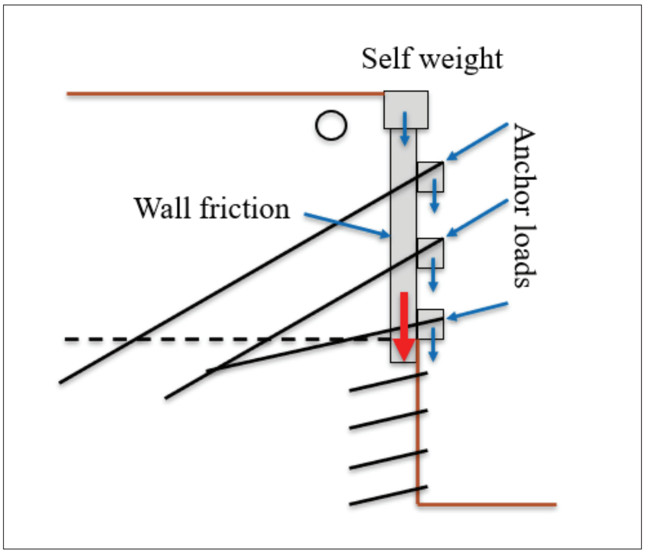

Figure 6: Typical cross-section of the retaining wall showing how vertical stresses develop at the pile toe

designed to bridge the tunnel. Long piles straddled the tunnels and transferred loads from the short piles to the rock below the tunnels. A reinforced concrete capping beam and the anchor waler beams were designed to fully support the vertical loads. In other words, the short piles did not rely on any vertical support from the rock below and were designed to hang. As the mining operations took place the stresses the rock beneath the piles were expected to relax and the stresses would redistribute through the portal structure down into the long piles.

Short piles were also used away from the tunnel face to reduce the total length of the more expensive piles. Tensioned steel rock bolts with a shotcrete facing supported the rock beneath these piles, however a capping beam was not necessary to redistribute the vertical stresses,

3.3 GROUND ANCHOR INTERACTIONS AND CONSTRAINTS

One of the most challenging aspects of the Portal retention design was setting out the ground anchors within the project designation boundaries whilst avoiding interaction between adjacent anchors and clashes with the tunnels.

The mined tunnels would be constructed with a ring of spiles around the crown. Loading on the spile occurs in opposing directions. Positioning these grout bodies too close together within a rock mass could cause a concentration of load opposing stresses resulting in shear and tensile failure of the rock leading to failure of the bonds. A minimum offset of 4m from the mined tunnel extrados to the ground anchor bond was required to reduce the effect of interaction between the anchor bond and the spile. A minimum offset of 800mm (4 x bond diameter) was adopted for adjacent anchor bonds and an assessment of a cone failure mechanism was undertaken.

Moderately conservative parameters were used for rock to grout bond strength to further reduce the risk of interaction, and robust anchor testing and monitoring regime was implemented.

The sub-strata designation boundary is a complex 3D shape that defined the boundaries that the tunnelling works must keep within. The boundaries allowed for future development of sites above and around the tunnel. These boundaries also limited where ground anchors could be located.

With constraints below the anchors (the mined tunnels), constraints above the anchors (the sub-strata designation and services), adjacent anchors to the sides, and suspect rock quality, locating each anchor in a suitable position became like threading a needle. A 2D design process was not sufficient for this complex 3D problem. All design elements were modelled in 3D and federated into a project wide BIM model weekly as the design progressed. This tool was crucial to accurately design the anchors. The BIM model also incorporated the 3D geological model developed in Leapfrog. The geological model was updated throughout the design phase as new investigation data was received.

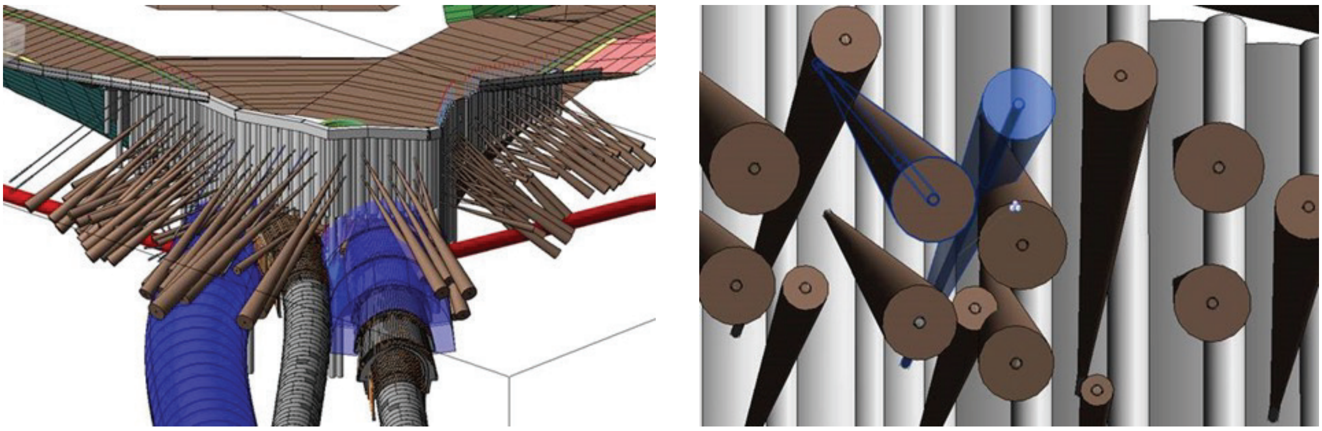

Figure 7: BIM model showing ground anchors, looking at back of Portal retention structure.
Left image: tunnels and exclusion zones shown in blue; anchor installation cones shown in brown.
Right image: a particularly constrained area where anchors were forced to 'thread the needle'.

The anchor installation tolerance of 2° was modelled as a cone for each individual anchor. Each anchor was checked against all the constraints, and fine tuning of the bond position, inclination and orientation was undertaken. Once complete, the anchor design loads were reviewed and then strand numbers and bond lengths determined to optimise the design. Multiple iterations of anchor design were required before the final product was complete. This resulted in 74 different anchor arrangements (i.e. unique inclination, orientation, strand number, free or fixed length) for the 95 ground anchors installed in the structure. The complexity later led to complications during construction where anchors could not be easily substituted for one another and quality control was vitally important.

An added benefit of BIM is that the anchors are each assigned unique identifiers and design properties such as anchor length, bond diameter, tensioning loads, design loads can be linked to the model. This helped speed up drawing production, changes during design and quality control. When the design package was handed over for construction, the model was provided with the drawing set as part of the design documentation.

3.4 STAGING OF THE PORTAL FACE ROCK CUT

The extent of uncemented sandstone within the tunnel faces and potential issues with stand-up time and groundwater seepage in this layer was considered in design. A complex methodology for a 'hit and miss' sequential installation sequence for the rock bolting installation, and shotcreting of the front face was adopted during design. 3D modelling of the rock bolted face was undertaken with FLAC numerical modelling. Past experience within the design team indicated that similar uncemented sandstone encountered in the Vector tunnel constructed in the 1990s had performed well during construction. The uncemented sandstone at the portal stood vertically for up to 48 hours in cut heights of up to 2 m and installation of rock bolts and shotcrete proceeded without any significant issues. One minor and inconsequential slabbing failure was observed during an excavation stage (Fig. 8). The slab was approximately 400 mm thick and appeared to have failed through the rock mass, not along any pre-existing defects. No issues were encountered with groundwater flows, with only minor seepages observed during excavation. Strip drains and weep holes were installed behind and through the shotcrete to relieve groundwater pressures.

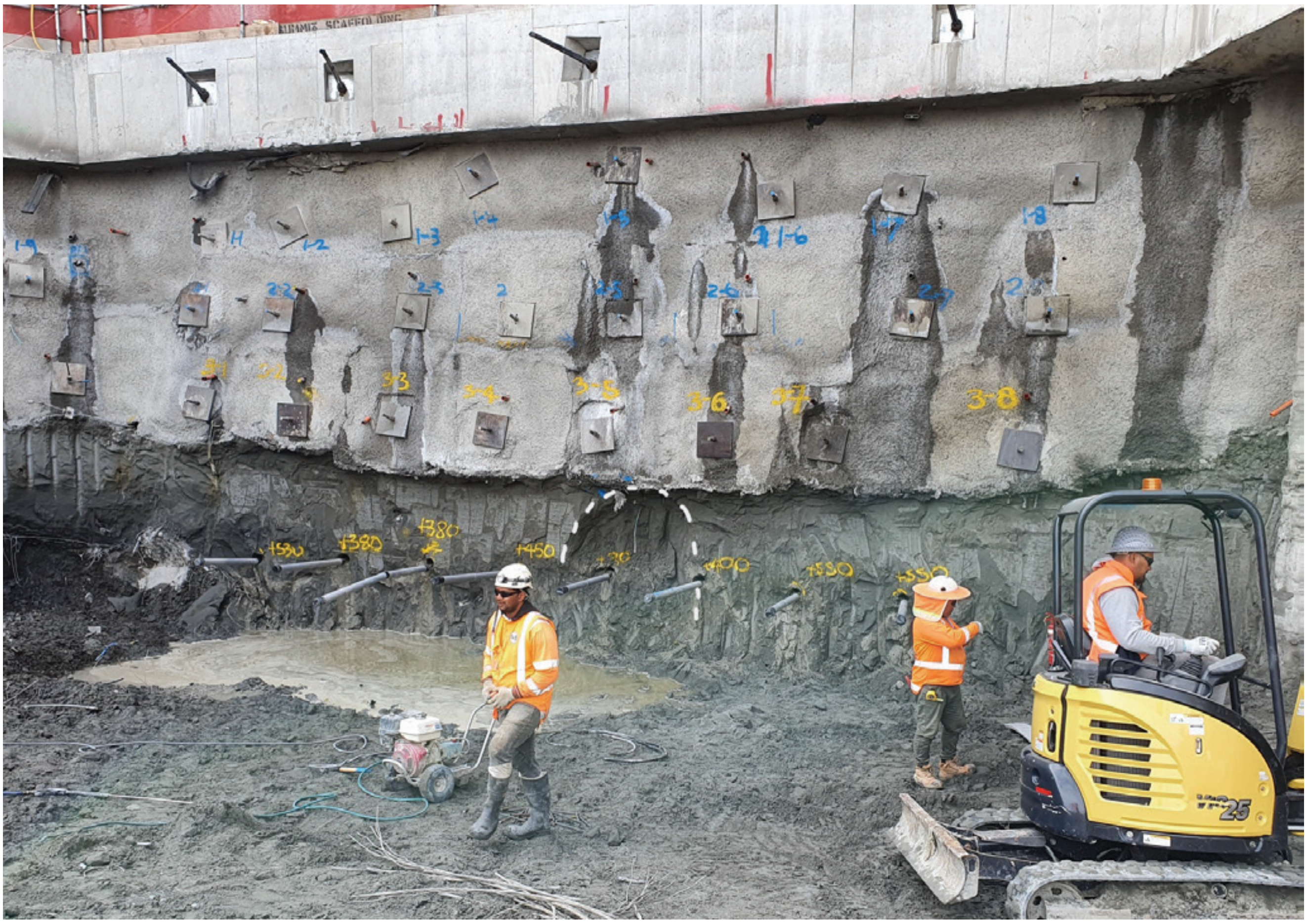

Figure 8: MC30/60 tunnel portal face during construction, uncemented sandstone (EUs1) present in face, note drop out failure highlighted with white dashed line, also note obliquely oriented anchors in waler beam above tunnel portal to avoid clashes with canopy tubes and spiles.

4 EXCAVATION MONITORING AND PERFORMANCE

4.1 INSTRUMENTATION AND MONITORING

Given the nearby sensitive infrastructure and importance of the portal to the project an extensive instrumentation and monitoring network was established to monitor wall and excavation performance. The instrumentation included:

- Survey monitoring prisms on the ground and retaining wall, measured by an automated total station.
- In-place inclinometers cast into select piles and boreholes drilled behind the wall.
- Anchor load cells installed between anchor plates and the waler beam.
- Vibrating wire piezometers cast into boreholes behind the wall to monitor groundwater.

All instrumentation was telemetered allowing for 'live' viewing of instrument readings during construction and automatic trigger level exceedance notification.

4.2 WALL DEFLECTION

The inclinometer and survey monitoring of the structure showed maximum deflections (up to 23 mm) approximately 60-70% of the predicted values. Figure 9 shows the maximum predicted horizontal displacement profile vs two of the inclinometers installed in the portal wall piles (P10 and P18, refer to Figure 5 for the location of the piles). The horizontal displacement at maximum excavation level and after backfilling and destressing of the anchors are both shown. Generally, the deflected shape of the pile length aligned well with the predictions. The waler beams and ground anchors restrain movement in the upper 10 m of the wall with the maximum deflection at approximately 12 m below ground level. Up to 8 mm additional deflection occurred as the wall was backfilled and anchors destressed.

We believe that the differences in the measured values vs the prediction are a primarily due to 3D effects of the concave portal wall alignment, and underestimation of the strength and stiffness of the uncemented sandstone unit.

The design team were aware of the limitations in the modelling undertaken. Given the tight design programme and high consequences of failure or unacceptable deformation, the limitations were accepted and a suitably conservative design approach was adopted.

4.2.1 3D effects

Plaxis 2D was primarily used to design the structure and assess displacements. The plane strain analysis could not account for the stiffening effects of the concave shape of the structure coupled with the large capping and waler beams (typically 1.2m high and 0.8m wide). The 3D effects of the structure are evident when comparing the deflection profile at pile P18 which was located at a corner with the profile at P10, located on a straight section. The deflection in the upper 7 m of the pile where the capping beam and waler beams are located is significantly lower than predicted, and lower than measured in pile P10 where the concavity of the wall shape is reduced. The inclinometer in pile P26 (located at a near right-angled corner of the wall) showed a similar profile to that in P18. The deflection profile for P26 is not shown for brevity.

4.2.2 Uncemented sandstone back analysis

A 2D back analysis was undertaken to understand the effect the uncemented ECBF sandstone (EUs1) strength and stiffness parameters were having on the model predictions. Noting that Figure 4 indicates that the cohesive strength adopted may have been conservative. Table 2 below shows the design and back analysed values for EUs1. Parameters for EU2 are also presented for comparison.

Table 2: Design and back analysed parameters

Parameter	Unit	EU2 design	EUs1 design	EUs1 back analysed
Effective cohesion (c')	kPa	100	25	75
Effective friction angle (φ')	degrees	40	40	40
Secant young's modulus ($E50^{ref}$)	MPa	400	100	200

All geotechnical units were modelled with a hardening soil (HS) model. The reference stress was set to 100 kPa with a stress dependency power factor (m) of 0.5. For brevity other parameters have not been presented here as they were not assessed as part of the back analysis.

The results of the back analysis are shown in Figure 9. The calculated deflected profile of the wall matches the measured values, however noting that in the upper 7 m of wall the model still overpredicts the deflection, possibly due to 3D effects associated with the structure and other complexities in the geometry of the tunnel excavation. It should be noted that only a single material unit the EUs1 material parameters were back analysed in what is a complex geotechnical problem. Therefore, this back analysis should not be overly relied upon for future projects. However, it does demonstrate that strength and stiffness parameters used for the uncemented sandstone unit did have a significant impact on the design predictions. The effective cohesion of the uncemented sandstone appeared to have a significant effect, with the material elements in the finite element model showing stresses at or near to the plastic limit using the original design parameters.

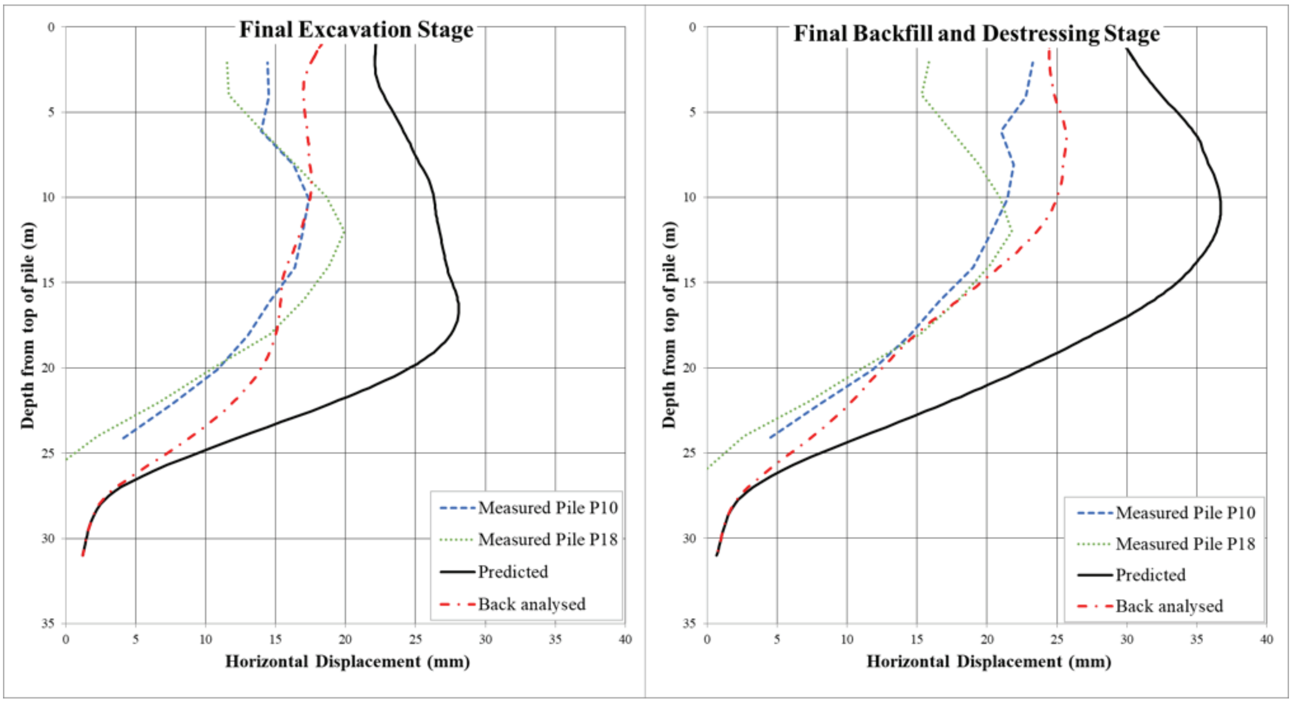

Figure 9: Predicted and back analysed deflection compared with measured deflection in piles P10 and P18

4.3 GROUND SETTLEMENT

Ground settlement behind the wall may have arisen from several different activities; lateral displacement of the portal structure, dewatering resulting in an increase in effective stress and mined/bored tunnelling. During design, each mechanism was considered separately and then combined to give a total settlement prediction. This prediction was then compared against deflection limits for the pavement, services, and nearby structures (Huia No. 2 Watermain). Multiple design iterations were required to find a suitable balance. The anchor prestress loads were adjusted to strike the right balance between horizontal displacement of the wall and vertical loads on the pile toe.

During construction ground settlement was measured using an array of surface monitoring pins installed along the surrounding streets and on adjacent buildings. There was a sufficient density of ground survey pins that detailed settlement contour plans could be developed. Given the complex construction staging it is difficult to accurately determine what proportion of settlement may have been related to each mechanism.

The settlement of a group of points located behind the wall (circled in Fig. 10) has been reviewed and is shown in Table 3 below.

Table 3: Settlement measured in monitoring points behind the portal wall on Nikau Street

Construction stage	Total settlement (mm)
Completion of bulk excavation	4 to 7
Completion of tunnelling works	9 to 11
Completion of backfill and destressing	12 to 15

The total settlement that occurred was lower than the predicted and values, noting that a level of conservatism was adopted during the design to reduce the risk of damage to nearby assets.

4.4 ANCHOR LOAD CELLS

Ten ground anchors were fitted with load cells to monitor loads during construction. After acceptance or suitability load testing, anchors were tensioned and locked off at a specified prestress load. As excavation progressed and the structure deflected an increase in load was measured in all but one (A112) of the load cells. None of the anchors reached their design working (serviceability) load, with the maximum load measured varying from 60 to 90% of the working load. This is not unexpected as the structure deflected less than predicted (Fig. 9).

5 CONCLUSIONS

The Mt Eden tunnel portal retention structure represented a challenging endeavour in underground construction in Auckland. The structure consisting of bored piles and sprayed concrete was supported with waler beams and ground anchors bonded in ECBF rock. Three tunnel alignments were mined and bored through and below the portal wall. Following construction of the tunnels the structure was backfilled, and all ground anchors destressed.

An intricate design was necessary due to multiple site constraints including complex geometry, interaction between design elements, existing infrastructure, sensitive neighbouring structures, and uncertain geological conditions.

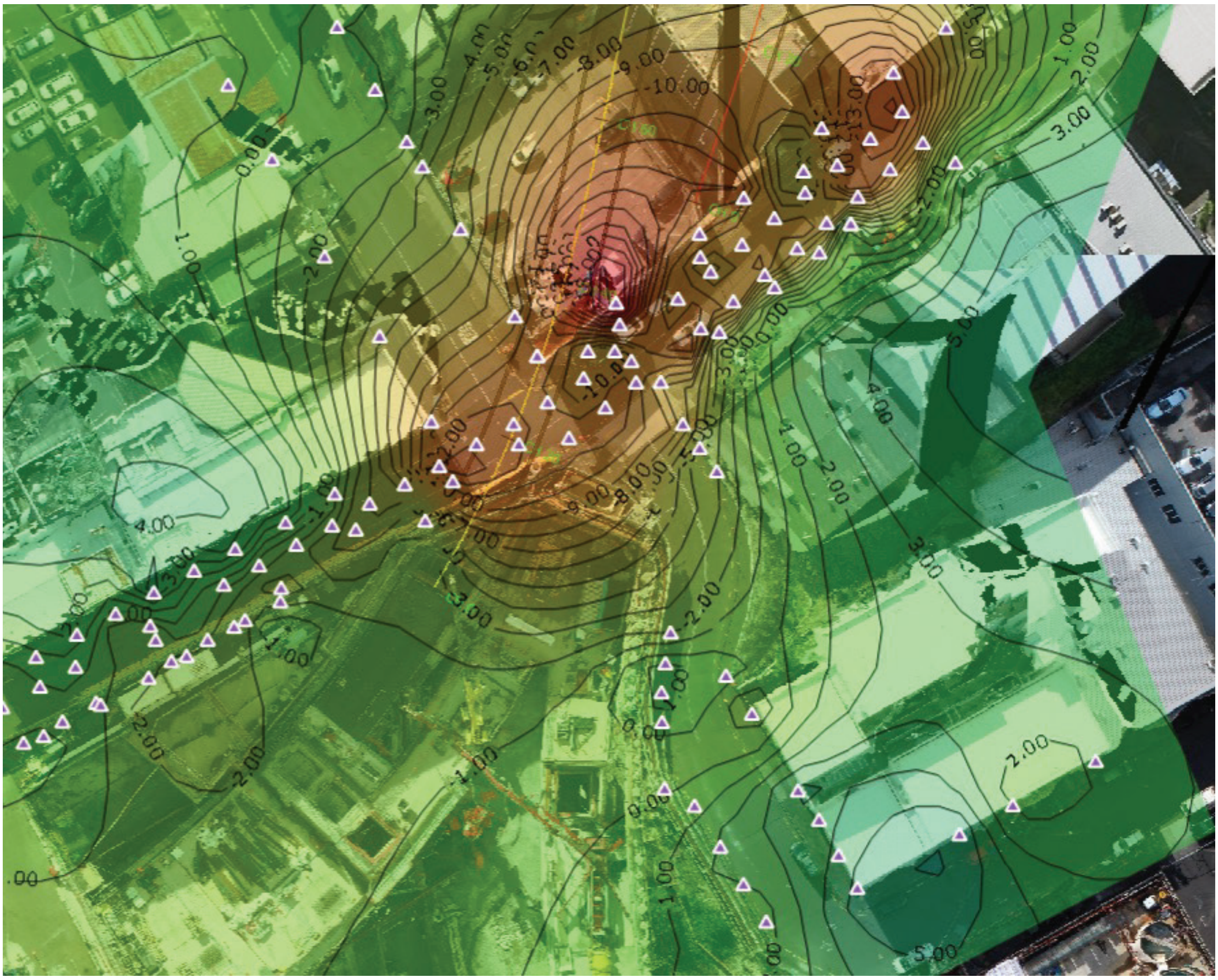

Figure 10: Total settlement contour at completion of construction generated from surface monitoring points (6/05/2024)

Figure 11: Maximum measured loads in anchor load cells compared with prestress load and design predictions.

3D modelling was used to overcome the geometric challenges of a constrained site. The innovative first-time use of BIM on a transportation project in New Zealand was incredibly valuable for the design and construction teams, especially when checking interaction between the ground anchors and tunnels.

The presence of thick beds of uncemented sandstone was a key concern for the design team. Core samples were often returned disturbed leading the design team to adopt low strength and stiffness parameters. A cautious staged approach was adopted for the tunnel face rock stabilisation design. During construction, the rock performed well with only minor slabbing failures noted.

Ground anchor investigation tests targeting the uncemented sandstone proved grout to ground bond strengths of 1500 to 2000 kPa for bond lengths of 3m.

An extensive array of instrumentation was installed to monitor the structure and surrounding assets during construction. Inclinometers, survey monitoring and anchor load cells all returned measurements that were lower than initial design predictions. A review of the monitoring results indicated that structural 3D effects and underestimation of the uncemented sandstone strength and stiffness may have contributed to the over prediction of displacement.

ACKNOWLEDGEMENTS

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PARAWEB GEOSYNTHETIC MSE REINFORCEMENT
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PGA adjustment factors for TS1170.5 to account for nonlinear site response on soft soils

C.A. de la Torre, M. Cubrinovski & B.A. Bradley – Department of Civil and Environmental Engineering, University of Canterbury, Christchurch, New Zealand.
S.S. Bora – GNS Science, Avalon, New Zealand.

C de la Torre,
University of Canterbury

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ABSTRACT

This paper summarises the development and implementation of an adjustment factor for PGA from the 2022 update of the New Zealand (NZ) National Seismic Hazard Model (NSHM2022) for adoption into the NZ technical specifications TS1170.5:2024 (TS1170). The study focuses on soft soil sites within TS1170 Site Classes IV, V, and VI (i.e., with $V_{S30} \leq 300$ m/s). The adjustment factor is based on nonlinear site-response analyses of NZ characteristic soft soil sites and an examination of observations from extensive national and global ground-motion databases. These simulations treat soil nonlinearity more rigorously than the approximations used in the empirical ground-motion models employed in NSHM2022. The scientific background and details of the analyses used to develop the PGA adjustment factors are documented in de la Torre et al. (2025a), and the parametrisation of the proposed adjustment factor for implementation into TS1170 is described in de la Torre et al. (2025b). We compare the adjusted PGAs to the PGAs from NSHM2022, and the PGAs from the 2004 NZ seismic loading standard NZS1170.5:2004. The adjustment factors reduce the PGA for all three site classes, and the amount of reduction increases as the PGA hazard increases. For example, the reductions for 500-year and 2500-year hazard levels for the highest hazard cities of New Zealand are approximately 10-20 % and 15-30 %, respectively. Despite this adjustment, compared with NZS1170.5:2004, the adjusted PGAs in these high-hazard regions are still 40-50 % higher for the 500-year return-period ground motion.

1 INTRODUCTION

The 2022 update of the New Zealand (NZ) National Seismic Hazard Model (NZSHM2022; Gerstenberger et al. 2024a) presented a major update since the previous 2010 update (Stirling et al. 2012), including a completely new set of ground-motion models (GMMs; Bradley et al. 2024) and a significantly improved source model (Gerstenberger et al. 2024b). The NSHM2022 results in peak ground accelerations (PGAs) that are approximately 1.5-2 times higher than the PGAs from the 2004 NZ seismic loading standard NZS1170.5:2004, for many cities in the highest seismic hazard regions of NZ (e.g., Wellington), depending on the site class considered (Kaiser et al. 2024, Bora et al. 2024). This increase in the PGA produced PGAs > 1.0 g for high hazard regions, even on soft soil sites, which triggered the need to scrutinise this particular output of NSHM2022.

The objective of de la Torre et al. (2025a, 2025b), which are summarised in this paper, was to carefully scrutinise the very high PGAs output by NSHM2022 for soft soil sites with V_{S30} < 300 m/s. The scrutiny involved comparison with historical observations from existing ground-motion databases, evaluation of the treatment of soil nonlinearity in GMMs, and quantification of the effects of this modelling aspect on the resulting PGA hazard. In the subsequent step, the nonlinear functions used in the GMMs were compared with equivalent relationships derived from more rigorous nonlinear site-response analyses, as well as with observations of soil nonlinearity from the records of 2010-2011 Canterbury Earthquake Sequence. This investigation revealed that the PGA for soft soil sites directly resulting from NSHM2022 is likely overpredicted due to the approximate treatment of the nonlinear site response in GMMs for high-intensity ground motions (i.e., PGA > 0.5 g). The interested reader should refer directly to de la Torre et al. (2025a, 2025b).

2 TREATMENT OF NONLINEAR SITE RESPONSE IN NSHM2022 GMMS

Nonlinear site-response effects in global GMMs are modelled as a simple reduction factor that is generally a function of V_{S30} and PGA on a reference condition (PGA^r), which is typically representative of rock conditions with V_{S30} = 760 – 1100 m/s. For weak shaking, the nonlinear site response models have no effect (i.e., the multiplicative factor is ~1). However, as the intensity of ground motion on rock (i.e. PGA^r) increases, more soil nonlinearity is expected in soft soils, which generally results in additional deamplification of the ground motion due to damping effects (primarily at short-to-moderate periods and PGA). This is illustrated in Figure 1, which shows the nonlinear site response models for PGA and V_{S30} = 225 m/s from all the GMMs adopted in the NSHM2022. Figure 1 shows that most of the GMMs produce similar levels of nonlinear deamplification for the V_{S30} values considered here. As explained in de la Torre et al. (2025a), many of the GMMs actually adopt the same nonlinear functions, or use the same or similar data to constrain the nonlinear function.

Given the scarcity of historical ground motion observations of very high intensities (i.e. PGA >> 0.3 g), the semi-empirical nonlinear site-response models adopted in GMMs utilise site response analyses to constrain the models at large intensities. For example, equivalent-linear site-response analyses by Walling et al (2008) and Kamai et al. (2014) have been used to partially or fully constrain the nonlinear models of most of the NSHM2022 GMMs. The equivalent-linear method approximates the nonlinear behaviour of soils by iterating to find a single value of shear modulus and damping, for each soil layer, that is representative of the expected level of strain (Idriss and Seed, 1968). These values of effective shear modulus and damping are then adopted for the entire duration of the ground motion. While this approximation is reasonable for weak-to-moderate levels of shaking, it is not appropriate for severe shaking where the behaviour of soil is strongly nonlinear and changes drastically throughout a ground motion (Kramer and Paulsen, 2004). For this reason, we compare results from equivalent-linear analyses and nonlinear analyses, and evaluate the sensitivity of the predicted PGA hazard to the method adopted for constraining nonlinear site-response models of GMMs.

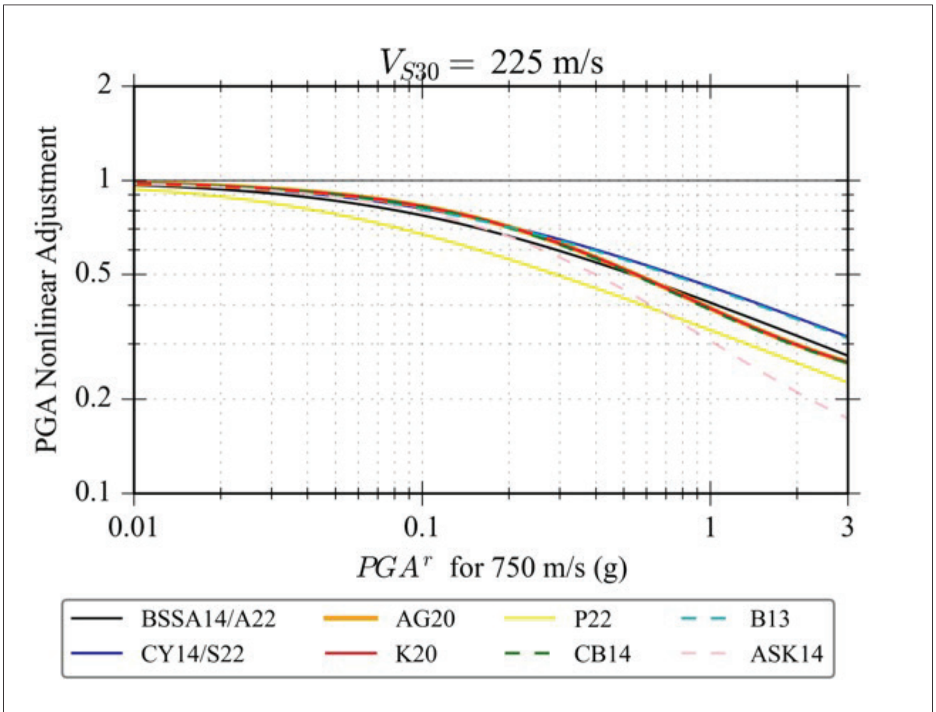

FIGURE 1: Nonlinear models adopted by the GMMs used in the NSHM2022 for PGA and V_{S30} = 225 m/s.

3 OVERVIEW OF NZ-SPECIFIC NONLINEAR SIMULATIONS

We used results from existing nonlinear simulations of New Zealand soft soil sites to evaluate the sensitivity of the PGA hazard from soil nonlinearity. The simulations used are those by de la Torre et al. (2024) and Cubrinovski and Ntritsos (2023), who performed nonlinear analyses for nine sites in Wellington, and thirteen sites in Christchurch, respectively. de la Torre et al. (2024) performed the nonlinear (NL) analyses in OpenSees and DEEPSOIL, and also performed equivalent linear analyses (EL) in DEEPSOIL. Cubrinovski and Ntritsos (2023) performed nonlinear total stress (TSA) and effective stress analyses (ESA) with the stress-density constitutive model (Cubrinovski and Ishihara, 1998) in the finite element code DianaJ. Both studies applied input motions with increasing intensity to evaluate the effect on the site response as intensity increases, which made their results easily adaptable for this application. For each site, we compute a moving average of nonlinear site amplification as a function of PGAr, as illustrated for two example sites in Figure 2. We then grouped sites by V_{S30} ranges of < 200 m/s, 200-250 m/s and 250-300 m/s, representative of Site Classes VI, V, and IV, respectively. We used these aggregated results to modify the nonlinear function of GMMs as summarised in the next section (Section 4).

4 ADJUSTED NONLINEAR SITE RESPONSE FUNCTIONS CONTRAINED ON NZ-SPECIFIC SIMULATIONS

In order to use the results of NZ-specific nonlinear site response analyses, we recalibrated the nonlinear functions used in GMMs to match these results. For each site class (i.e. Site Classes IV, I, and VI, with representative V_{S30} values of 275, 225, and 175), we calibrated three models to capture the range of results observed from different simulation approaches and different sites. The three models for each V_{S30} are qualitatively labelled 'Mild', 'Moderate', and 'Aggressive', to reflect increasing levels of nonlinearity and increasing departures from the default nonlinear models. Figure 3 shows the background data from simulations, and the proposed adjusted models that were fit to the simulation results for V_{S30} = 175 and 225 m/s. The left side of Figure 3 includes the nonlinear models themselves, in the same format as Figure 1, while the right side shows the surface PGA implied by the nonlinear models, given a reference condition PGA^r . As shown on the left side of Figure 3, the nonlinear simulation results imply a steeper gradient to the nonlinear functions (i.e., more deamplification) than the model by Seyhan and Stewart (2014) [SS14], which is the default model adopted by some of the GMMs. The adjusted models therefore reflect this stronger level of nonlinearity, which manifests as lower surface PGAs for all cases (right side of Figure 3).

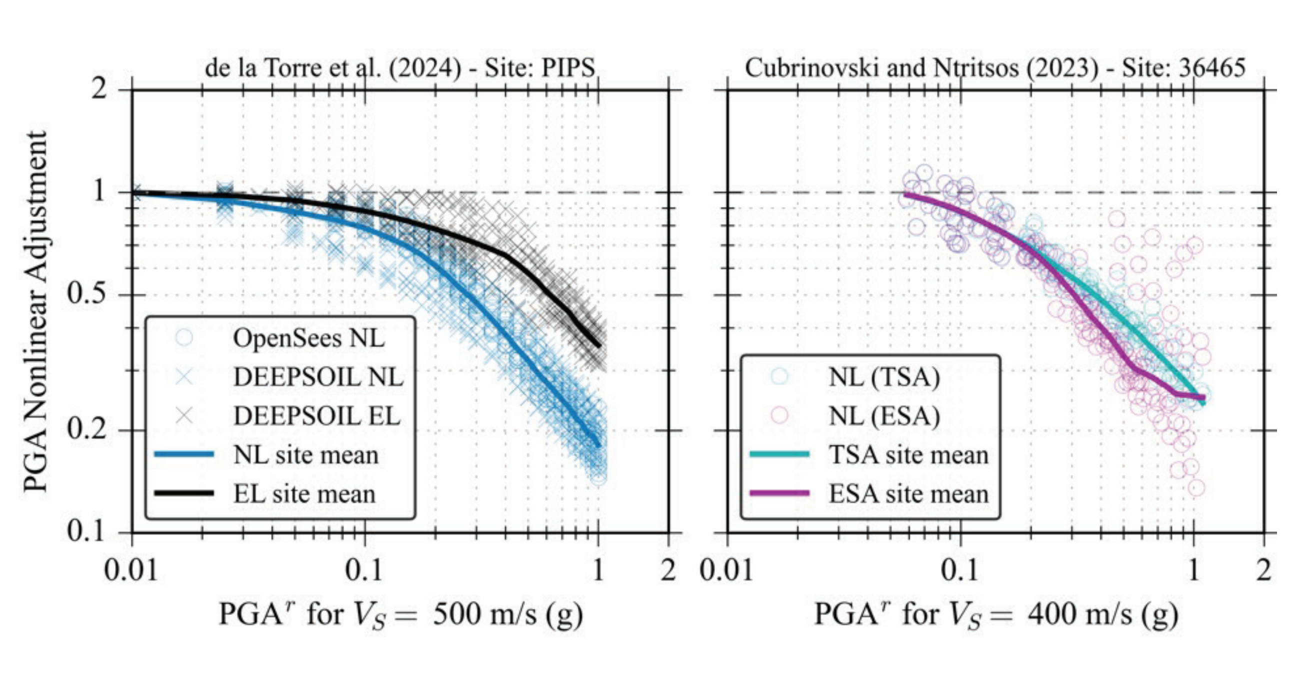

FIGURE 2: An example of the calculation of nonlinear site response from existing nonlinear simulations by de la Torre (2024) and Cubrinovski and Ntritsos (2023) for two sites. Each point represents the result for an individual ground motion and the solid lines are the moving average.

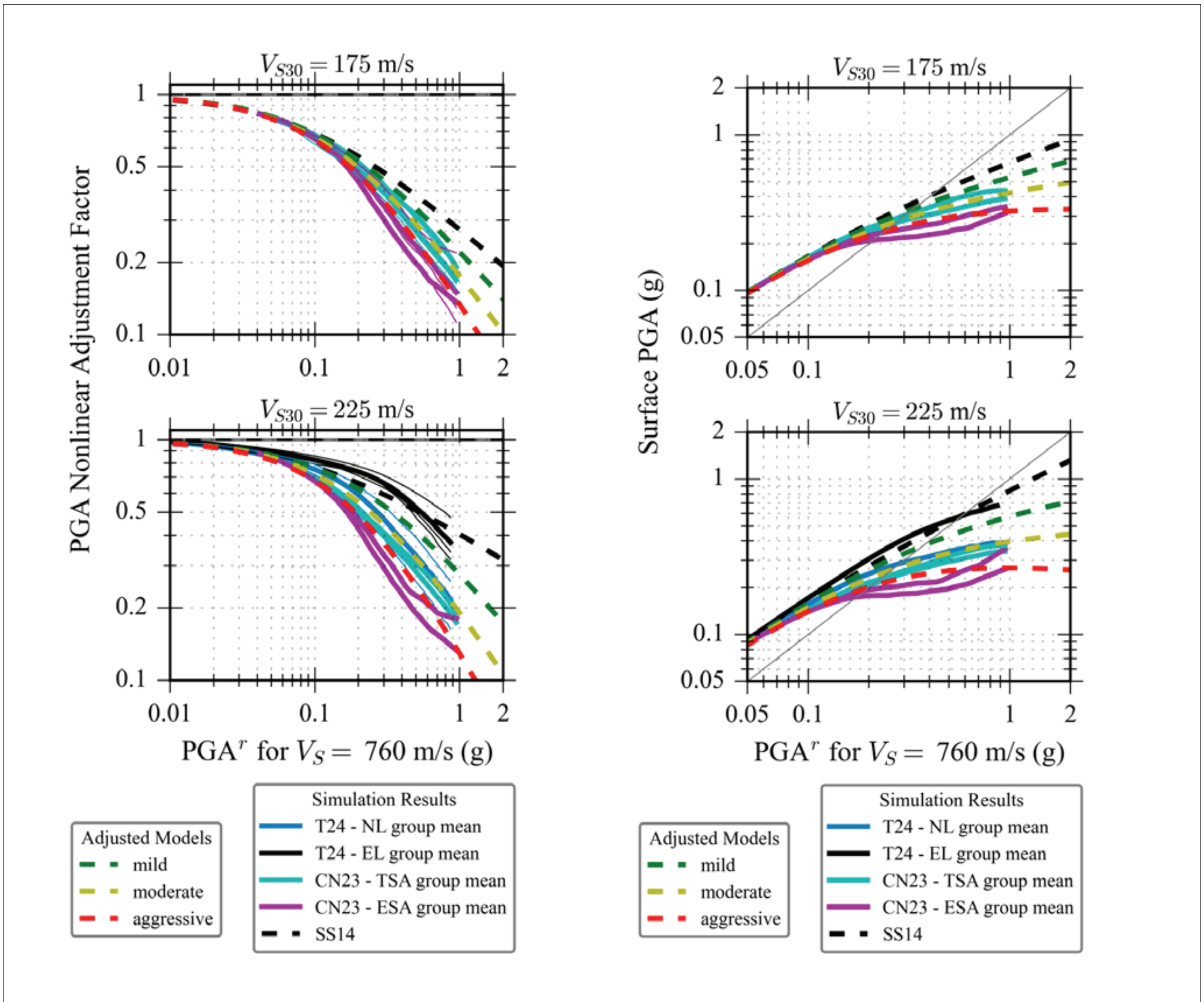

FIGURE 3: Left side: Adjusted nonlinear site-response models for PGA, as a function of PGA^r for a reference condition of V_{S30} = 760 m/s, based on site-response simulations results from de la Torre et al. (2024) [T24] and Cubrinovski and Ntritsos (2023) [CN23]. Results and modelling approximations are shown for two V_{S30} groups (i.e., site classes) in the different panels. For comparison the SS14 model, adopted by some GMMs, is also included. Right side: surface PGA as a function of PGA^r implied by the adjusted and default nonlinear site response models

5 INFLUENCE OF ADJUSTED MODELS ON THE OVERALL PGA HAZARD

The adjusted nonlinear functions shown above in Section 4 and Figure 3 were implemented into two GMMs used in the NSHM2022, and the hazard analysis was re-run with these new models to quantify the influence on the overall hazard. The full earthquake source model was used for this calculation. The hazard was calculated for six cities (Gisborne, Napier, Wellington, Blenheim, Christchurch, and Otira). Figure 4 shows the resulting PGA hazard curves for the city of Wellington for the 'Mild', 'Moderate' and 'Aggressive' adjusted nonlinear models, and the 'default' model for the V_{S30} = 175 and 225 m/s cases. The percent reduction in the PGA hazard (calculated from the hazard curves), relative to the default model, is also shown in Figure 4 for all six cities. The hazard curves in Figure 4 show a clear reduction in the PGA hazard for a given probability of exceedance, and this reduction is most pronounced for the 'Aggressive' case, as expected. It also evident from the hazard curves, that the reduction increases as the probability of exceedance decreases (i.e. as the return period and the hazard increases), which was also expected based on the adjusted models shown in Figure 3, which diverge from the default model as *PGA^r* increases. These trends are also visible in the plots of percent reduction to the PGA hazard in the right side of Figure 4. A weighted-average reduction in the PGA hazard was calculated by assigning degree-of-belief weights of 0.1, 0.4, 0.4 and 0.1 to the default (i.e., 0 % reduction), mild, moderate, and aggressive models, respectively. The weighted average percent reduction is included in Figure 4.

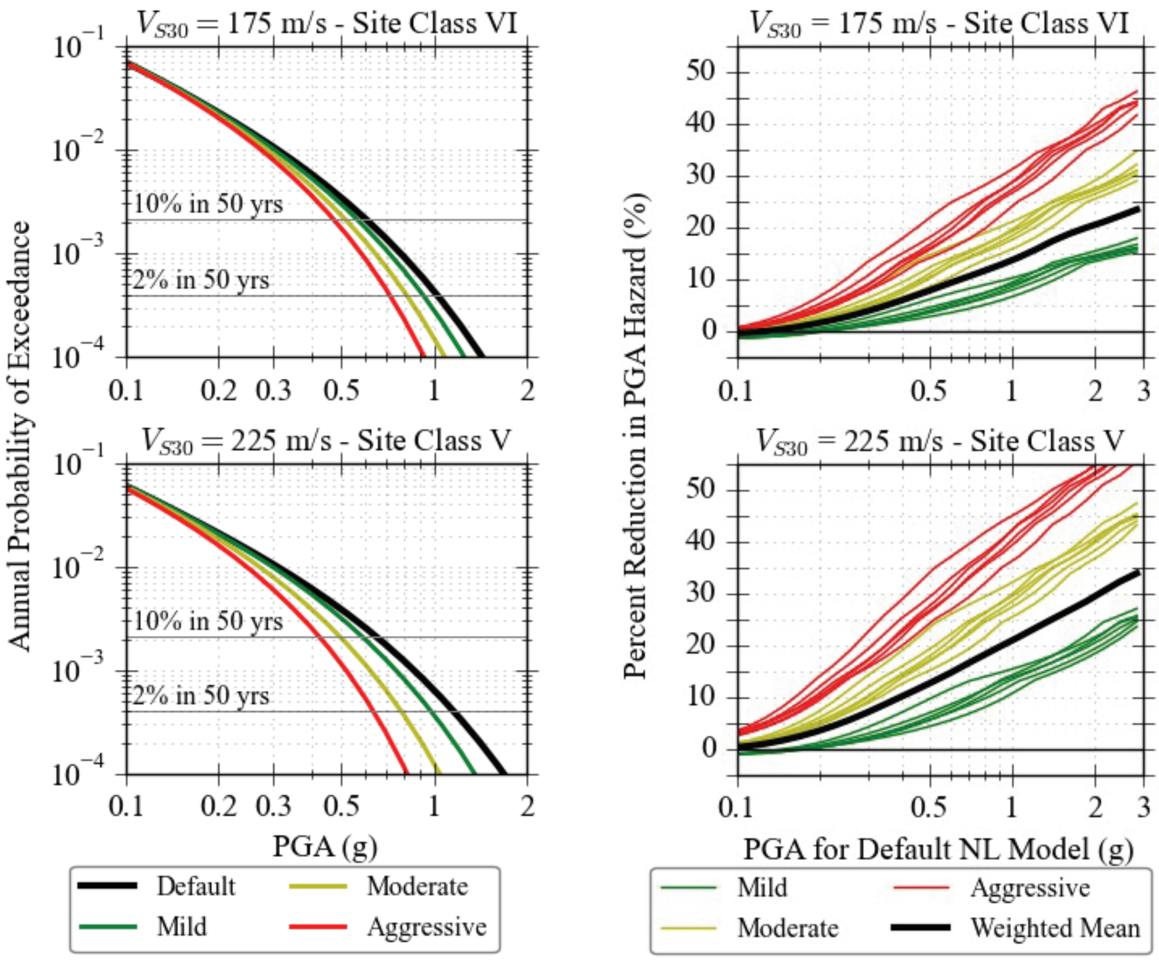

FIGURE 4: Left side: PGA hazard curves for Wellington using the Atkinson (2022) [A22] GMM with the default and three adjusted nonlinear site response models (shown in Figure 3) for VS30 = 175 m/s (top) and 225 m/s (bottom). Right side: percent reduction in PGA hazard as a function of PGA for the default nonlinear model, for all six cities and the three adjusted models.

6 PARAMETRIC MODEL FOR THE PGA ADJUSTMENT FACTOR

The weighted average percent reduction models (i.e. right side of Figure 4) were parametrised into a simple linear adjustment factor model, for adoption into TS1170.5. The adjusted PGA ($PGA_{adjusted}$) can be calculated using Equation 1:

$$PGA_{Adjusted} = PGA_{NSHM2022} \times (1 - R_{PGA}) \quad (1)$$

where $PGA_{NSHM2022}$ is the PGA obtained directly from NSHM2022 and R_{PGA} is the PGA adjustment factor which is calculated with Equation 2:

$$R_{PGA} = \begin{cases} A_0 \times \ln(PGA_{NSHM2022}) + A_1, & for\ PGA_{NSHM2022} \geq PGA_{thresh} \\ 0, & otherwise \end{cases} \quad (2)$$

where A_0 and A_1 are coefficients for the linear models as defined in Table 1, and PGA_{thresh} is the threshold PGA, below which no adjustment to PGA is required. The parameterised linear models for all three site classes, along with the background data used to constrain the models (i.e. the weighted means), are shown in Figure 5. This adjustment factor was then applied to all locations across New Zealand for Site Classes IV, V, and VI.

Table 1: Coefficients for the proposed linear models to calculate the PGA reduction factor using Equation 2.

Site Class	A_0	A_1	PGA_{thresh} (g)
IV	0.076	0.123	0.198
V	0.114	0.227	0.137
VI	0.085	0.171	0.133

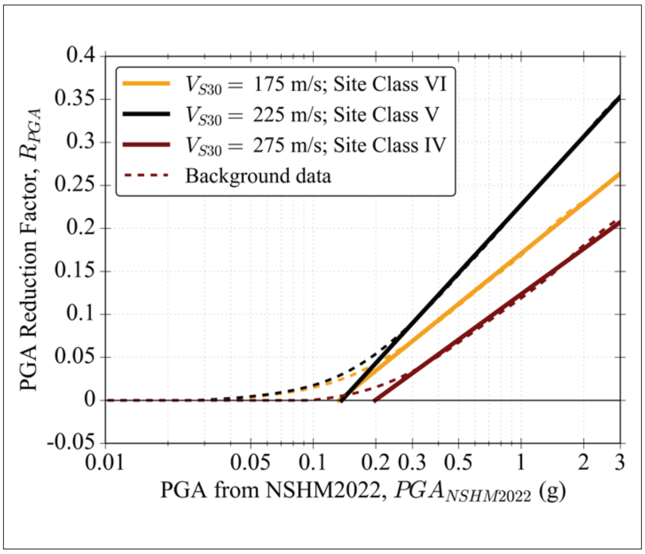

FIGURE 5: Recommended weighted mean percent reduction in the PGA hazard as a function of the NSHM2022 PGA values for V_{S30} = 175, 225, and 275 m/s, including the proposed simplified linear models given by Equation 2 and Table 1.

7 COMPARISON OF ADJUSTED PGA WITH NSHM2022 AND NZS1170.5

The adjustment factors (i.e. the percent reduction in PGA) in Figure 5 were applied to all locations listed in TS1170 for Site Classes IV, V, and VI using Equations 1 and 2. The ratios of the adjusted PGA ($PGA_{adjusted}$) to the NSHM2022 output ($PGA_{NSHM2022}$), and the values of $PGA_{adjusted}$ themselves, for the 500-year return period are plotted in Figure 6 for all locations. As the percent reduction increases with increasing PGA value (Figure 5), the highest hazard cities in NZ have the lowest ratios in Figure 6, with reductions in PGA of approximately 12% for Site Class IV, 15% for Site Class V, and 22% for Site Class VI. The greatest reductions occur for Site Class IV, as previously observed in Figure 5. This is because, for Site Class IV, the adjusted nonlinear site-response models deviate furthest from the default nonlinear models (e.g. SS14), as illustrated in Figure 3. In other words, the trends of percent reduction for the different site classes do not solely reflect the amount of nonlinearity expected for each representative V_{S30}, but they represent the difference between the adjusted model and the default model for a given V_{S30}.

The actual values of $PGA_{adjusted}$ for all locations are plotted in bottom half of Figure 6. As before, the PGAs for Site Class IV (i.e., the stiffest of the three considered) are the highest. The Site Class V PGAs are still higher than the Site Class VI PGAs, although they are much more similar after applying the adjustment factor. This is because the nonlinear simulation results for the V_{S30} = 175 and 225 m/s bins were not significantly different, resulting in similar nonlinear site-response models for both site classes. The abrupt changes in PGA ratios and PGA between nearby points in Figure 6 are caused by cities with similar latitude being distributed between the east coast and the west coast.

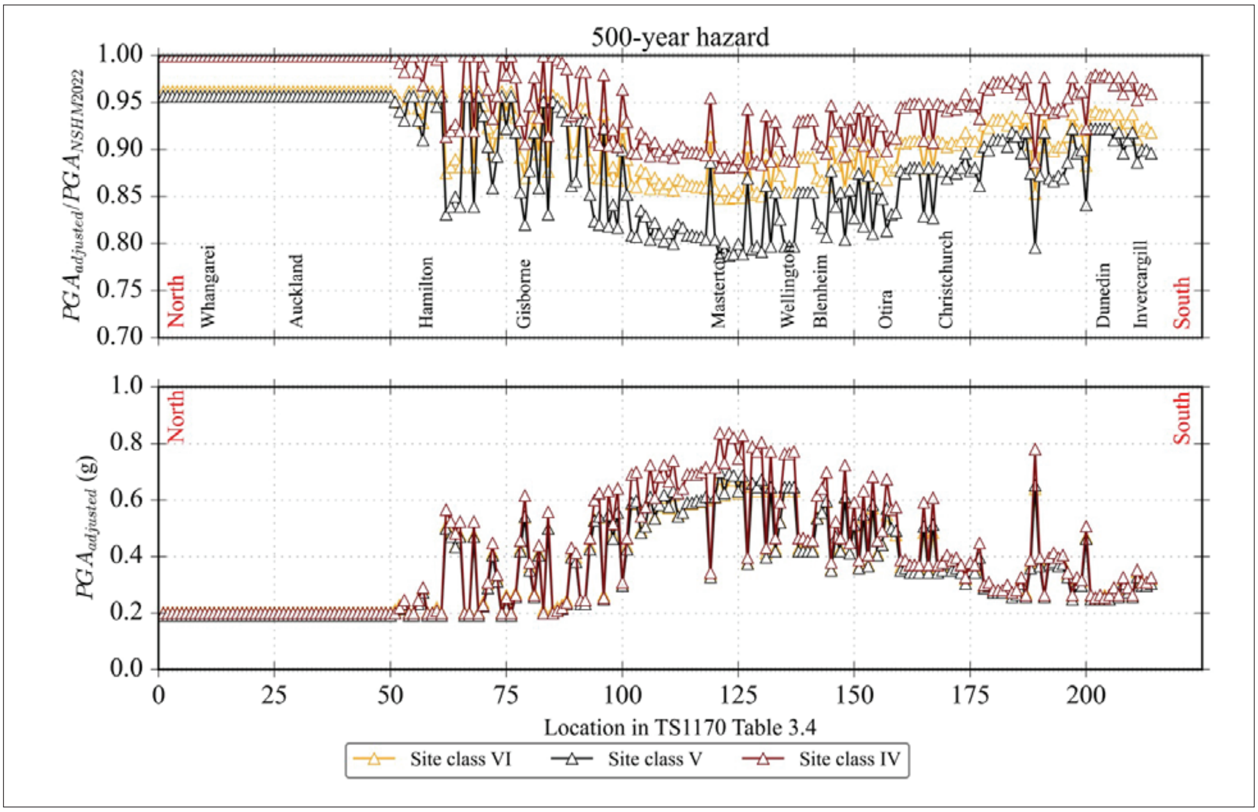

FIGURE 6: Top: Ratios of the adjusted PGA to the PGA from NSHM2022 for all cities in Table 3.4 of TS1170 for the 500-year hazard. Bottom: The $PGA_{adjusted}$ values for the three site classes and all cities. The x-axis represent the position of the city in the tables going from North on the left side of the figure to South on the right.

8 CONCLUSIONS

This paper summarises the findings in de la Torre et al. (2025a, 2025b), in which the PGA output by the 2022 NZ NSHM and the modelling of soil nonlinearity in empirical ground-motion models (GMMs) was carefully scrutinised. The results suggest that the PGA from the NSHM2022 on soft soil sites were likely overpredicted given the oversimplified treatment of soil nonlinearity in GMMs, which has conventionally been constrained using equivalent-linear simulations at high ground-motion intensities. The hazard calculation was rerun using improved nonlinear models, constrained on New Zealand-specific nonlinear site response simulations that rigorously account for the effects of soil nonlinearity. The nonlinear simulations suggest greater deamplification of PGA for high input motion intensities. To account for this, an adjustment factor that reduces PGA was developed. The adjustment factor was developed only for soft soil site classes (i.e. Site Class IV, V, and VI) and is a function if the PGA output directly from NSHM2022. As the PGA hazard increases, the adjustment factor produces more reduction in PGA, resulting in reductions to PGA of approximately 10-20% and 15-30% for the 500- and 2500-year hazards, respectively, for the highest hazard regions of New Zealand.

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Liquefaction Characteristics of Gravelly Soils Prepared by Water Sedimentation Method

L. Wang, G. Chiaro & S. Rees, Department of Civil and Natural Resource Engineering, University of Canterbury, Christchurch
C. Cappellaro, Tonkin & Taylor, Christchurch
A. Pokhrel, Beca Limited, Christchurch

L Wang,
University of Canterbury

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ABSTRACT

Field observations and evaluations from 32 case histories of liquefaction in gravelly soils worldwide, including three in New Zealand, have indicated that gravelly soils in alluvial deposits are the most susceptible to liquefaction. However, replicating these conditions in laboratory tests remains a challenge, particularly in achieving uniform specimen preparation for reliable liquefaction assessment. This study addresses these challenges by using a newly developed water-sedimentation (WS) method for gravelly soil specimens that can reproduce as much as possible the anisotropy and fabric of naturally deposited alluvial sand, enabling a better assessment of liquefaction potential. Notably, this WS method enhances density uniformity and minimises the inherent segregation between small sand and large gravel particles. A series of stress-controlled undrained cyclic triaxial tests were conducted on WS gravelly soil specimens reconstituted at relative densities (*Dr*) between 20% and 60% and isotropically consolidated at 100 kPa effective confining stress. The specimens were then subjected to cyclic stress ratios (CSR) ranging from 0.14 to 0.45. Comparisons with specimens prepared by the moist tamping (MT) method showed that soil fabric significantly influences liquefaction resistance, with the WS specimens generally less resistant to liquefaction. In addition, density and gravel content also play a critical role, with liquefaction resistance increasing with both density and gravel content. This study indicates that for a better evaluation of the liquefaction resistance of alluvial gravelly soils, the combined effects of fabric, density state and gravel content must be considered together.

1 INTRODUCTION

Gravelly soils (i.e., gravels, gravelly sands and sandy gravels) are commonly encountered in natural alluvial deposits and reclaimed fills and play a critical role in the seismic performance of infrastructure. However, a persistent challenge in geotechnical engineering is the lack of universally accepted guidelines for characterising and evaluating their liquefaction resistance. Traditionally, gravelly soils have been considered to exhibit higher liquefaction resistance than sandy soils due to their higher permeability, which is believed to inhibit the development of significant excess pore water pressures during earthquakes (Seed et al., 1976). This assumption has led to the general perception that these gravels are less susceptible to liquefaction than clean sands. Nevertheless, increasing field evidence from 32 earthquake events involving widespread liquefaction in gravelly soils, as summarised by Rollins et al. (2021, 2022) and Pokhrel et al. (2024), has challenged the validity of this long-held assumption. Consequently, gravelly soils are often regarded as 'problematic' due to their complex and poorly understood cyclic behaviour.

Previous laboratory studies on the liquefaction resistance of gravelly soils have primarily relied on conventionally reconstituted specimens, prepared using methods such as moist tamping (MT) (Kokusho et al., 2004, 2007; Hara et al., 2004, 2012; Chang et al., 2014) and air pluviation (AP) (Hubler et al., 2018; Evans and Zhou, 1995). While these techniques are widely used, they often fail to reproduce the natural alluvial characteristics of gravelly soils, thus offering limited insight into fabric-related influence on liquefaction resistance. Given that fabric plays a critical role in controlling the cyclic response of gravelly soils, there is a pressing need for novel specimen preparation methods that can better reproduce the natural fabric of alluvial gravelly soil deposits, thus enabling a better evaluation of their liquefaction resistance.

The water sedimentation (WS) method, which allows particles to settle in a manner more representative of natural hydraulic sorting, could offer a promising alternative to conventional specimen preparation techniques (Oda et al., 1978). However, experimental studies examining the liquefaction resistance of gravelly soils prepared using the WS method remain limited, constraining the current understanding of how soil fabric influences the cyclic response and liquefaction resistance of alluvial gravelly soils.

In this study, a systematic and repeatable WS specimen preparation technique was developed to simulate as much as possible the depositional characteristics of alluvial gravelly soils. Specimens prepared using the WS method were then subjected to cyclic undrained triaxial loading to evaluate their liquefaction resistance. The results were compared with those obtained from previous studies employing the conventional MT technique (Pokhrel et al., 2023, 2024) to examine the fabric effects on the cyclic behaviour of gravelly soils, thereby providing a deeper understanding of the liquefaction potential of alluvial gravelly soils and contributing to a more comprehensive understanding of fabric-related effects on their liquefaction resistance.

2 METHODOLOGY

TEST MATERIALS

In this study, New Brighton Sand (NB Sand), Dalton River Washed Sand (DRW Sand), and rounded Pea Gravel were used to prepare well-graded sand-gravel mixtures for testing. To create a less uniform host sand and minimize gap-grading, the two sands were mixed in equal proportions by mass (50%-50%). The pea gravel was then added to the host sand to produce sand-gravel mixtures (SGM) with 10% and 25% gravel content (G_C) by mass (i.e., SGM with sand-dominated structures).

As reported in Table 1, index properties were evaluated for the test soils and mixtures according to relevant standards from the Japanese Geotechnical Society (JGS) and New Zealand Standards (NZS). They include the maximum (e_{max}) and minimum (e_{min}) void ratios, specific gravity (G_s), mean grain size (D_{50}), coefficient of uniformity (C_u), and coefficient of curvature (C_c). In Figure 1, the particle size distribution curves are reported.

Table 1: Material properties

Materials	G_s	e_{max}	e_{min}	D_{50} [mm]	C_u	C_c
NB Sand	2.66	0.623	1.016	0.20	1.64	0.93
DRW Sand	2.65	0.598	0.900	0.68	3.14	0.95
Pea Gravels	2.66	0.482	0.665	5.60	1.38	1.11
0% GC	2.66	0.563	0.906	0.26	2.50	0.90
10% GC	2.66	0.739	0.494	0.29	2.77	0.66
25% GC	2.66	0.632	0.415	0.41	4.50	0.42

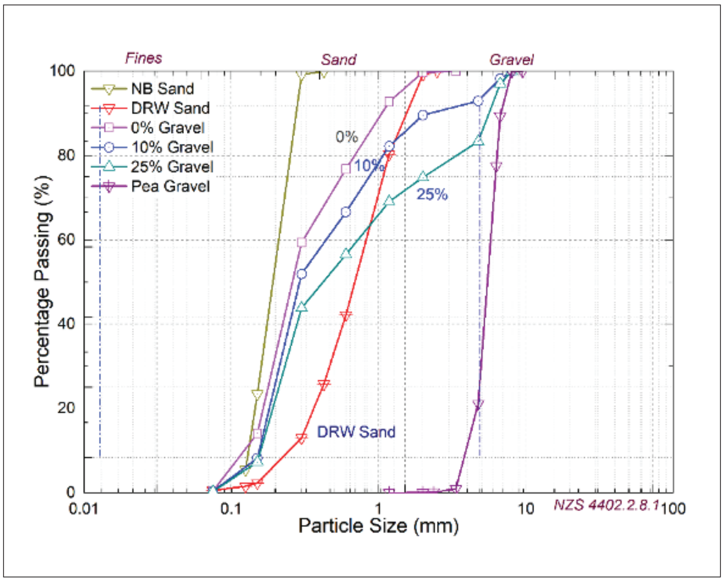

Figure 1. Particle size distribution curves of tested materials (adapted from Pokhrel et al., 2024).

2.2 SPECIMEN PREPARATION METHOD

Specimens of 60 mm diameter and 137 mm height were reconstituted by a new WS method developed by the authors (Figure 2a). In this method, sand-gravel mixtures were carefully poured into a pluviation device with a constant drop height and water level, at a constant rate, to create a uniform 'sand rain.' This pluviation system was derived from insights gained from previous research by Vaid and Negussey (1988) and Lagioia et al. (2006), with some necessary adjustments and modifications. Specifically, to minimize inherent segregation between sand and gravel grains, the multi-layer deposition method proposed by Dobry (1991) was adopted, and specimens were built in 10 identical layers.

After pluviation was completed, the deposit was left resting for approximately 12 hours. Following this, the water level was lowered down to the top of the deposited specimen. Fresh deposits were densified to the target *Dr* by vibration induced by a hammer impact around the sides of the deposition tube. To mitigate disturbance during handling, specimens were frozen in a freezer before being transferred to a triaxial cell for testing. To do so, excess free water in the deposited specimen was drained prior to freezing. Special attention was given to temperature control when handling the frozen specimen tube, particularly during PVC tube removal, drilling of bender element holes, and trimming of the top surface. Shear wave velocity (Vs) was measured for all specimens, and unique sets of values were obtained for the different specimens prepared at the same Dr and G_C, confirming the suitability of the developed WS method to create uniform specimens.

2.3 TESTING PROCEDURE

Once the frozen specimen was positioned on the triaxial pedestal (Figure 2b), a rubber membrane was carefully placed around the specimen, which was then thawed under 20 kPa cell pressure for 12 hours. The diameter and height of the specimen were measured both before and after thawing. To achieve a B-value ≥ 0.95, a multi-step saturation process was conducted, including carbon dioxide percolation, followed by de-aired deionized water saturation under double vacuum, and finally by application of 200 kPa back pressure. The specimen was then isotropically consolidated to a target 100 kPa effective confining pressure in 20 kPa increments.

Stress-controlled undrained cyclic triaxial tests were conducted on specimens subject to constant-amplitude sinusoidal axial load with a cyclic stress ratio (CSR) ranging from 0.14 to 0.45 at a frequency of 0.05 Hz using a pneumatic loading system (Figure 3c). The CSR was calculated as per Equation 1:

$$CSR = \frac{\tau_{cyc}}{\sigma'_{v0}} = \frac{\sigma_d}{2\sigma'_c} \tag{1}$$

where σ_d = target single-amplitude axial stress; and σ'_c = mean principal effective stress at the end of consolidation.

3 RESULTS

3.1 UNDRAINED CYCLIC RESPONSE

Typical undrained cyclic responses are reported in Figure 3 for a loose specimen (*Dr* = 30%) with 10% G_c, in terms

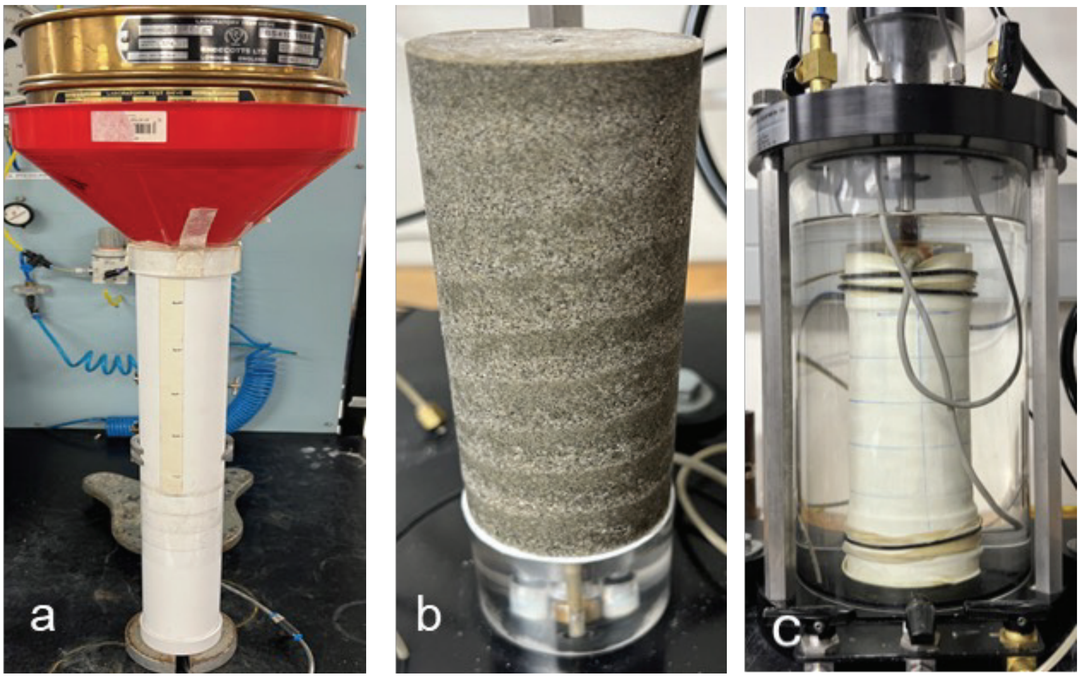

Figure 2. Specimen preparation and testing: (a) setup of the WS method developed in this study; and (b) example of layered frozen specimen prepared by the WS method; (c) a specimen tested in the triaxial device

of deviator stress, *q* (Figure 3a), excess pore water ratio, r_u (Figure 3b), and axial strain (Figure 3c). For completeness, the corresponding effective stress paths (Figure 3d) and stress-strain relationship (Figure 3e) are represented for the same specimen. Pore water pressure and axial strain increased progressively with increasing cycles of loading (N_c) until the r_u was equal to or greater than 0.95. The loading program was terminated when 5% single-amplitude axial strain was reached, and the specimen failed, typically under extension shear loading conditions. As expected, in the case of the denser specimens (*Dr* > 30%), a higher N_c under the same CSR was required to result in similar failure conditions (i.e., $r_u \geq 0.95$ or 5% double amplitude axial strain, ε_{DA}).

In this study, the state of initial liquefaction was defined as either $r_u \geq 0.95$ or ε_{DA} = 5%, and cyclic resistance ratio (CRR_{15}) was defined as the CSR value at 15 cycles of loading (N_C). The liquefaction resistance curves of sandy soil and 10% gravel content cases based on 5% ε_{DA} and $r_u \geq 0.95$ are plotted in Figure 4.

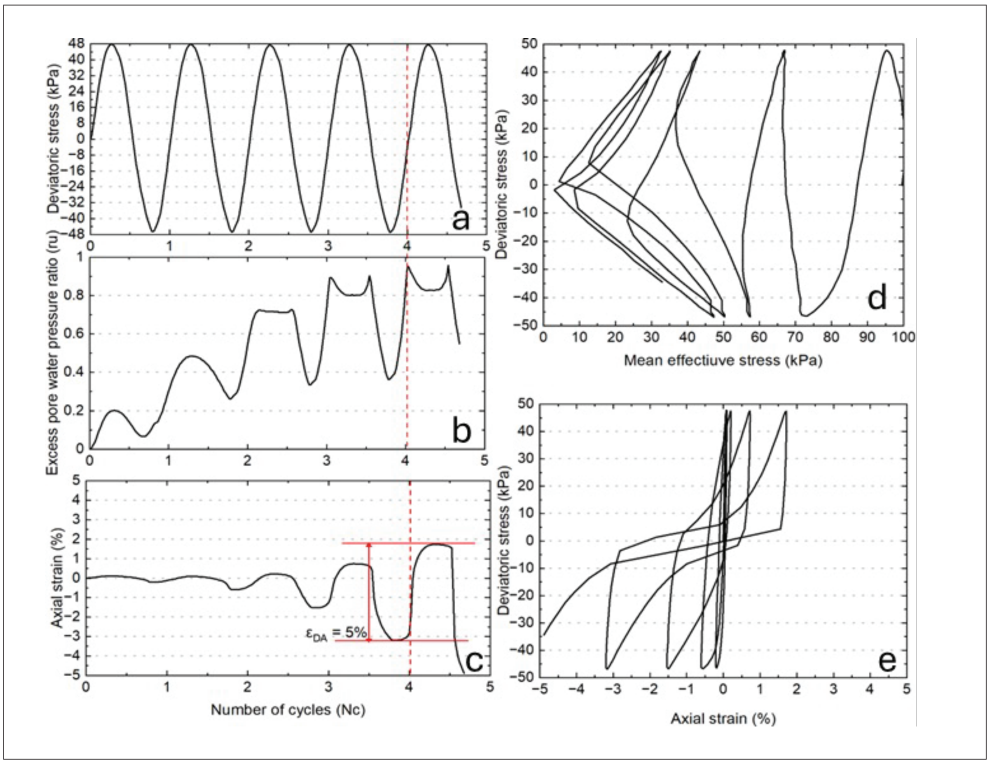

Figure 3. Typical undrained cyclic triaxial response of a loose specimen (Dr = 30%) with 10% gravel content.

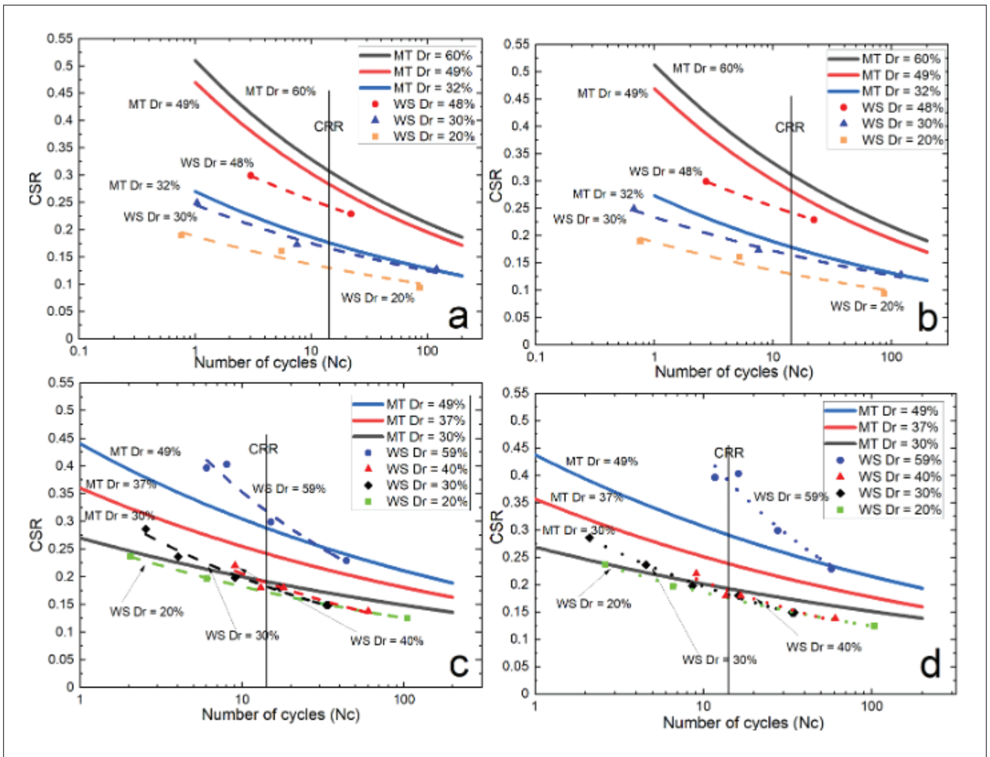

Figure 4. Liquefaction resistance curves (a) sandy soil, 95% ru; (b) sandy soil, 5% ε_{DA}; (c) 10% gravel, 95% ru; (d) 10% gravel, 5% ε_{DA}.

The liquefaction resistance curves for specimens with 10% GC, defined based on the 95% r_u and 5% ε_{DA} criteria, are nearly identical within the *Dr* range of 20% to 40%. This suggests that 95% r_u and 5% ε_{DA} thresholds were reached at approximately the same time during cyclic loading. This behaviour reflects the typical undrained cyclic response of loose specimens. However, at Dr = 59%, the CSR curve based on the 5% ε_{DA} criterion becomes noticeably steeper than that based on the 95% r_u criterion, leading to different CRR values, as shown in Figures (4c) and (4d). Notably, the significant divergence between the two initial liquefaction criteria at higher densities has also been reported in previous studies (Pokhrel et al., 2024). Therefore, in this study, the $r_u \geq 0.95$ criterion was adopted to define initial liquefaction and determine the CRR_{15} value of specimens.

3.2 EFFECTS OF RELATIVE DENSITY AND SOIL FABRIC ON LIQUEFACTION RESISTANCE

To evaluate the fabric effects on liquefaction resistance, comparisons with experimental data available for specimens prepared by the MT method by Pokhrel et al. (2024) are made in Figure 4.

At the same *Dr*, for any given value of Nc and CSR, the cyclic resistance of sandy soil specimens prepared by the WS method is lower than that of specimens prepared by the MT method, indicating a significant influence of specimen fabric on the liquefaction resistance.

However, for the 10% G_C case, the cyclic resistance (r_u = 0.95) of WS specimens is not consistently lower than that of the MT specimens. For instance, for Dr = 30% and 40%, the liquefaction curves of WS specimens are steeper than those of MT specimens (Figure 4c). It appears that at Nc > 10, the WS specimens exhibit a weaker cyclic resistance, while at NC < 10, the resistance of WS specimens exceeds that of the MT specimens. A similar trend is also observed for denser specimens (Dr = 59%), with the transition occurring at N_C = 25.

3.3 EFFECTS OF GRAVEL CONTENT ON LIQUEFACTION RESISTANCE

Based on the liquefaction curves shown in Figure 4, CRR_{15} was defined for all mixtures investigated in this study and by Pokhrel et al. (2024), prepared at different Dr and Gc = 0, 10 and 25% G_C. The results for the 25% G_C specimens are not yet complete and will be presented in full detail elsewhere in the future. Linear correlations between the CRR_{15} and void ratio (*e*) are obtained for each tested G_C configuration, as shown in Figure 5. The test results shown in Figure 5 indicate that the fabric effects are negligible for loose density conditions (i.e., higher void ratio values); however, as the density increases (i.e., void ratio decreases), the difference in liquefaction resistance between the two specimen preparation methods increases, with the MT specimens becoming progressively stronger than the WS ones.

It is clear from Figure 5 that fabric, Dr and G_C play a key role on the liquefaction resistance of alluvial gravelly soils. Therefore, to more accurately evaluate

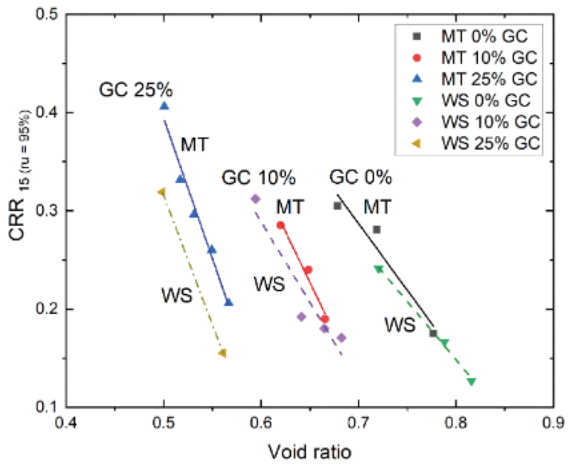

Figure 5. Correlation between CRR_{15} and void ratio for specimens prepared by different preparation methods.

the liquefaction resistance of alluvial gravelly soils, the combined effects of these three key factors must be considered all together.

4 CONCLUSIONS

In this study, a series of undrained cyclic triaxial tests were conducted to investigate the combined effects of fabric, relative density (Dr) and gravel content (G_c) on the liquefaction resistance of sandy gravelly soils. To do so, a new water sedimentation (WS) method was developed for gravelly soils, and the specimens were prepared using the same materials and sand-gravel mixtures tested by Pokhrel et al. (2023, 2024), who employed the moist tamping (MT) method, and tested under the same triaxial testing conditions.

It is found that the proposed water-sedimentation (WS) method for gravelly soils allows the preparation of specimens with uniform density and minimises the inherent segregation between small sand and large gravel particles, thus mimicking as much as possible the fabric of naturally deposited alluvial sands. Therefore, it enables a better assessment of the liquefaction potential of alluvial gravelly soil.

The experimental results show that in the case of sandy soils, the liquefaction resistance of those prepared by WS is less than those prepared by MT, irrespective of the density state. However, for the 10% Gc case, the cyclic resistance of WS specimens is not consistently lower than that of the MT specimens. Specifically, for Dr = 30% and 40%, it appears that for cycles loading number (Nc) > 10, the WS specimens exhibit a weaker cyclic resistance, while at Nc < 10, the resistance of WS specimens exceeds that of the MT specimens. A similar trend is also observed for denser specimens (Dr = 59%), with the transition occurring at Nc = 25. Moreover, irrespective of the density state, it is observed that the fabric effects are negligible for loose density conditions;

however, as the density increases, the difference in liquefaction resistance between the two specimen preparation methods increases, with the MT specimens becoming progressively stronger than the WS ones.

It is evident that fabric, Dr and G_C play a key role on the liquefaction resistance of alluvial gravelly soils. Therefore, to more accurately evaluate the liquefaction resistance of alluvial gravelly soils, the combined effects of these three key factors must be considered all together.

The results of ongoing laboratory investigations on specimens, prepared with higher Gc (i.e., 25% and 40%) across a broader range of Dr, will provide further useful information to better characterise the cyclic response of alluvial gravelly soils and the combined effects of fabric, Dr and GC on their liquefaction resistance.

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Simplified CPT-based liquefaction ejecta severity model using Christchurch data

K.M. Azul, R.P. Orense & L.M. Wotherspoon
University of Auckland, Auckland, New Zealand.

K Azul,
University of Auckland

NZGS SYMPOSIUM JOINT WINNER: Best Student Paper

ABSTRACT

Christchurch, New Zealand, and its surrounding areas experienced significant liquefaction-induced damage during the 2010-2011 Canterbury Earthquake Sequence (CES) events. Ejecta is one of the liquefaction manifestations commonly observed and can cover significant parts of a site. This study focuses on developing a model that classifies sites based on whether liquefaction ejecta manifests or not, and the severity of the ejecta manifestation. Utilizing a database with 5000+ CPT data investigations in the region, the input parameters used in the model are simplified representations of fundamental geotechnical properties closely linked to liquefaction and the surface manifestations. Several models were developed using various combinations of these input parameters to segregate the sites into varying levels of ejecta severity, and their performances were compared. The final model presented can estimate the severity of ejecta at a site, specifically the lowest and highest levels (i.e. no ejecta and most ejecta classifications). Further analysis of various predictive capacity measures showed how much the model under- and over-predicted the observations. Combining severity levels into a single level was also studied to see its effects on the accuracy and performance of the model. Overall, given the relative simplicity of some of the inputs, the model shows promise as part of a large-scale liquefaction severity prediction system.

1 INTRODUCTION

Liquefaction is one of the possible effects of an earthquake and can manifest itself as ejecta and lateral spreading. There have been many studies done to model and predict when liquefaction may be triggered and/or induce manifestations, such as those by Boulanger and Idriss (2014), Robertson and Wride (1998), Iwasaki et al. (1978), and Maurer et al. (2015b), while Youd et al. (2002) focused on the lateral spreading displacement prediction.

This study focuses on the ejecta induced by liquefaction and how it can be predicted, including its severity based on the amount of surface area covered. The study utilizes a database of records across three different earthquakes from Christchurch and surrounding areas to create a prediction model. The performance is analysed using various metrics, first by analysing how well it separated the records in the database, and then how it performed on a per-event basis. This includes spatial analyses to identify any possible avenues for improvement of the model. The model, while focused on CPT data, is aimed to be used with a suite of other models. Ultimately, this suite of models will be able to cover larger areas, with the CPT-focused model providing more detail in the prediction maps, where applicable. Hence, a key consideration in choosing the CPT-based predictors is the ability to determine/estimate them using other resources, if possible, while maintaining a certain amount of site-specific detail. Being able to predict if and when this damage will manifest is crucial in the proper planning and zoning of areas.

2 DATA AND MODEL DEVELOPMENT

The Geyin et al. (2020) database, with the published article by Geyin et al. (2021) presents a curated dataset containing ~15,000 cone-penetration-test-based liquefaction case histories compiled from three earthquakes in Canterbury. The compiled, post-processed data are presented in a dense array structure, allowing researchers to easily access and analyze a wealth of information pertinent to free-field liquefaction response i.e. triggering and surface manifestation explaining its contents and resources, is utilized in the study. This database contains 5000+ sites with CPT investigations along with other relevant liquefaction information for three earthquakes: M_W7.1 2010 Darfield, M_W6.2 2011 Christchurch, and M_W5.7 2016 Valentine's Day earthquakes. This results in ~17,000 records altogether. The relevant liquefaction information includes the classification of the manifestations observed for each site, the peak ground acceleration (PGA), and the water table depth for each event.

Geyin et al.'s (2020) database used a ~10 m radius around the CPT to classify the site's observed liquefaction manifestation for each of the events enumerated above. Their classification system utilizes 6-levels, modified after Green et al. (2014) M_w 7.1 Darfield earthquake and includes up to ten events that induced liquefaction. Most notably, widespread liquefaction was induced by the Darfield and M_w 6.2 Christchurch earthquakes. The combination of well-documented liquefaction response during multiple events, densely recorded ground motions for the events, and detailed subsurface characterization provides an unprecedented opportunity to add well-documented case histories to the liquefaction database. This paper presents and applies 50 high-quality cone penetration test (CPT, with an additional level, coded as level 10, for sites where manifestation classification cannot be done. Table 1 shows the six key levels of the system used by Geyin et al. (2020, 2021) New Zealand, earthquakes: A Curated Digital Dataset (Version 2), focusing on the descriptions related to surface ejecta and/or lateral spreading. This study focuses on developing a model that can predict the liquefaction ejecta severity; hence, only the sites classified as level 0-3 are utilized, and ejecta severity is consequently tied to how much of the ground is covered around the site.

Table 1: Liquefaction manifestation classification system (after *Geyin et al. 2020, 2021)* New Zealand, earthquakes: A Curated Digital Dataset (Version 2.)

Level	Key descriptions related to ejecta and lateral spreading
0	No ejecta; No lateral spreading.
1	<5% ejecta coverage of ground surface; No lateral spreading
2	5%-40% ejecta coverage of ground surface; No lateral spreading
3	>40% ejecta coverage of ground surface; No lateral spreading
4	Ejecta possible; Main manifestation is lateral spreading with crack-displacement widths<200mm
5	Ejecta possible; Main manifestation is lateral spreading with crack-displacement widths>200mm

The CPTs were processed using Boulanger and Idriss's (2014) methodology to determine the normalized and clean sand-equivalent penetration resistances, q_{c1N} and q_{c1Ncs}, respectively, while Robertson and Wride's (1998) methodology was used to determine the soil behaviour type, I_C. The study also used Lees et al. (2015a) alongside Boulanger and Idriss's (2014) methodology to determine the fines content, *FC*.

The liquefaction ejecta severity predictors (hereafter referred to as 'predictors') used in the model development were representative values of the subsurface characteristics derived from the CPT investigations and the load associated with the event. To represent the resistance to liquefaction of the subsurface related to its density, the study used the average normalized cone penetration resistance, R_{qc1N}, for the first 10 m of the subsurface. For the same 10-m zone, the averaged soil behaviour type, RI_C, was used to represent the soil type behaviour. This is the same as the I_{C10} defined in Maurer et al. (2015a). In addition, a modified version of Ishihara's (1985) concept of a protective crust was employed as it may inhibit the surface manifestation despite subsurface layers liquefying underneath it. The modified protective crust, H_{cap}, is defined as the thickness from the ground surface to the first liquefiable layer, defined as below the groundwater table (GWT) with I_C < 2.6. The load is represented by the peak ground acceleration (PGA) on the site. The PGA values used are mostly from the Geyin et al. (2020) database, although much of the PGAs for the 2011 Christchurch event have been updated with the data provided to the authors (Upadhyaya et al. 2019). This was done as Wotherspoon et al. (2014, 2015) have noted that particular strong motion stations' recordings (and, consequently, PGA) were affected by spikes associated with cyclic mobility and have suggested using revised PGA values for these sites in the modelling of the PGA distribution.

In the development of the model, all records from the Geyin et al. (2020) database were used, except for when: (i) the site's record has a liquefaction manifestation level of 4, 5, or 10 to limit the study to instances where liquefaction ejecta is the main manifestation, (ii) the site has less than 10 m of CPT data to allow for proper representation of the subsurface based on the predictors previously discussed, or (iii) the CPTs are co-located in which case the CPT with less data is removed. The resulting model development database (hereafter referred to as 'development database') has ~36% records with liquefaction ejecta manifestation, i.e. levels 1, 2, and 3, collectively called "cases" or *C*. Level 0 records are "non-cases" or *NC*. The broader groups, *C* and *NC*, are defined as such, except during threshold determination as explained below.

The model development was performed using binary logistic regression. Through this, a function is defined for the probability of an event *X, P(X)*, as shown in Equation 1, where z is the logit, which connects the predictors to the probability as shown in Equation 2 (Kleinbaum and Klein 2010).

$$P(X) = 1/(1+e^{-z}) \tag{1}$$

$$z = \beta_0+\beta_1 x_1+\beta_2 x_2+\cdots+\beta_n x_n \tag{2}$$

where β_i are the coefficients related to the predictor x_i, with β_0 being the intercept of the model.

This study utilized 10-fold cross-validation in the model development; this divided the development database into 10 groups, where 9 groups were used to develop a model (training set), which was then tested on the remaining group (validation/test set) (Refaeilzadeh et al. 2009). This meant that 10 models were developed and tested on 10 different groups. This allowed testing of each model's results on a separate subset of the development database and checking of the consistency of the models developed. The final model presented in this paper was based on the average of the coefficients, β, of the 10 models. For the purposes of the discussion, any reference to a model later refers to the final model, unless otherwise specified.

The model separated *C* from *NC* via a threshold value, T_1, i.e. records where $P(X)>T_1$ are predicted as *C*, otherwise, *NC*. T_1 is chosen such that it is maximized while at least 80% of *C* are correctly predicted. To separate *C* into the different levels, two more thresholds, T_2 and T_3, are identified. T_2 is identified using the same model and process above, but this time treating levels 2 and 3 as *C* and levels 0 and 1 as *NC*. A similar process is done for T_3 with only level 3 being *C*. In doing so, it was expected that $T_1<T_2<T_3$. Using a single model this way assumes that the effect of each predictor on the logit, *z*, is the same across all levels.

In binary logistic regression, model performance can be measured through a classification table along with other relevant metrics derived from it (Kleinbaum and Klein 2010). These same concepts were used to evaluate the final model using the development database, although slightly modified to account for the multiple levels introduced by the multiple thresholds.

3 RESULTS AND DISCUSSION

The 10 models from the cross-validation are consistent with each other, as supported by the standard deviations of their β coefficients and of their area under the curve (AUC) values being <1%, except for β_0's being ~3%. The AUC is the area under the Receiver Operating Characteristic (ROC) curve, which measures the discriminatory power of the model (Kleinbaum and Klein 2010). The AUC of the final model was 88%, with the cross-validation models' AUCs ranging from 87% to 89% as tested on their validation set.

The final model's logit equation is shown in Equation 3 with the thresholds, T_1, T_2, and T_3 being 0.36, 0.68, and 0.74, respectively.

$$z = 5.41 - 0.40\left(\frac{RI_C}{PGA}\right) - 0.004\left(\frac{Rq_{c1N}}{PGA}\right) - 0.10\left(\frac{H_{cap}}{PGA}\right) \tag{3}$$

The final model's predictors utilized a form similar to the factor of safety (FoS) utilized, i.e. the ratio of the resistance or capacity of the system to the load or demand. It was found that this form worked best for the target models and made physical sense, even in its simplified form, reflecting how the capacity and demand representations need to interact with each other to show their relative values to better represent the phenomenon. Each predictor also has a negative coefficient, showing that an increase in the ratio (increase in resistance and/or decrease in demand) will result in a lower logit, leading to a lower probability value. RI_C and Rq_{c1N} were explicitly paired together to represent the profile liquefaction resistance. RI_C aimed to quantify how susceptible to liquefaction the layers are based on their soil type behaviour, reflecting whether the layers are closer to being clean sands, clay, or in between. Rq_{c1N}, on the other hand, quantified the relative density of the subsurface layers. Together, these represented the liquefaction resistance in relation to the profile's strength and susceptibility based on the soil type behaviour. It can also be thought of as a deconstructed form of Boulanger and Idriss' (2014) CRR to explicitly separate the two resistance aspects previously mentioned. The third predictor is an aspect geared towards resistance against surficial manifestation and not liquefaction triggering. Ishihara (1985) noted that liquefaction triggering of subsurface layers does not necessarily mean manifestation above ground as the protective crust can inhibit manifestation. The modification in defining H_{cap} was done to keep the relative density incorporated to Rq_{c1N} and make H_{cap}'s determination simpler. Ishihara

(1985) also showed that the protective crust interacts with the shaking intensity, which is done in the model as well, although in a different approach. Together, the three predictors represented the susceptibility to liquefaction triggering and possible surficial manifestation.

Table 2 shows the model's overall performance metrics using the development database. Unlike a binary classification system, multi-level system performance cannot be reflected completely by the correct predictions at each level, as the predictions can now be "completely wrong", "slightly wrong", or "correct", depending on how far off the prediction is. Hence, aside from *correct* predictions, the evaluation of the model's performance includes investigating the number of *overpredictions, underpredictions*, and *absolute misclassification* predictions by the model. The true positive rate (TPR) from Kleinbaum and Klein (2010) was modified slightly to account for multiple levels, and is now simply the number of correctly predicted records in said level divided by the total records observed in that level. TPR can be likened to the model's accuracy, but specific to that level. The conservative prediction rate (CPR) is only applicable for *C* levels and is like the TPR but the numerator includes the overpredicted records, e.g. for level 1, the numerator is the sum of observed level 1 records predicted as level 1, 2, or 3. While overpredictions are not desirable, they can be viewed as "safer" mistakes in the context of liquefaction ejecta prediction. Underpredictions are the opposite of overpredictions and are still only applicable to *C* levels. Absolute misclass is when a *C* is predicted as *NC* or vice versa. Absolute misclass rate (AMR) is the absolute misclasses divided by the total records observed in that level. The use of these metrics together provides a better view of the model's performance rather than using a single metric, such as accuracy, to judge the whole model.

Table 2: Classification table for the development database (for all earthquake events combined)

Observed Levels	0	1	2	3
Total records	6614	1961	1492	346
TPR, CPR, AMR (%)	83, - , 17	19, 71, 29	15, 78, 12	81, 81, 5

Extending the binary versions in Bobbitt (2021), accuracy is the sum of correctly predicted records for all levels divided by the total records, while balanced accuracy is the average of the TPRs of each level, addressing the imbalance among levels by giving each level the same weight in its calculation. The accuracy and balanced accuracy of the model are 61% and 49%, respectively. The balanced accuracy was significantly lower than the accuracy, showing how the imbalance may affect typical measures of model performance.

Table 2 shows that the extreme levels, i.e. levels 0 and 3, have good TPR values, with the intermediate levels suffering quite a bit in this metric. One of the contributing factors to the low performance for the middle levels is the high overpredictions of observed level 1 and 2 records as level 3, also resulting to high CPR. AMR values across all levels are low, except for level 1. However, what separates level 0 and level 1 is not necessarily the lack of liquefaction triggering, but the lack of ejecta (maximum of 5% of surface coverage), which makes misclasses between the two levels more understandable. In relation to this, 12% of the 17% AMR for level 0 is attributable to those misclassed as level 1. The study also looked at consolidating levels 1 and 2 into a single level, resulting only to a slight increase in the TPR of the middle level.

Table 3 is Table 2 broken down into separate events, while Figure 1 shows the spatial distribution of the prediction classifications across the study area for each event. Table 3 shows that most of the records for the 2010 and 2016 events are *NC* (81% and 96%, respectively), while 2011 only has 15%. The 2011 event also accounts for 78% of *C* records and 94% of level 3 records. The previously discussed high overall TPR for level 0 is attributed mostly to the 2010 and 2016 events, while the high overall TPR for level 3 is attributed completely to the 2011 event. All level 2 and level 3 records from the 2010 and 2016 events were underpredicted (TPR and CPR being 0). Most of the overpredictions are attributed to the 2011 event as seen in the increase between TPR and CPR for levels 1-2, as well as the abundance of overpredictions in Figure 1. Most of this overpredictions are predicted as level 3. This highlights a possible event-specific characteristic that needs to be included in the model. Another key observation is how much of the *C* misclassed as *NC* are from the 2010 event (80-90% AMR).

Table 3: Classification table for the development database (delineated for each earthquake event)

Observed Levels	0			1			2			3		
Event	2010	2011	2016	2010	2011	2016	2010	2011	2016	2010	2011	2016
Total records	3030	518	3066	540	1310	111	159	1325	8	20	326	0
TPR (%)	87	14	91	14	17	67	0	17	0	0	86	-
CPR (%)	-	-	-	16	94	67	0	88	0	-	-	-
AMR (%)	13	86	9	84	6	33	89	2	13	85	0	-

Figure 1 shows that most of the absolute misclasses occurred near water bodies for all events. However, these are not the same kind of absolute misclasses across events. For example, focusing on the sites around the Avon River, for the 2011 and 2016 events, most of the absolute misclasses are *NC* misclassed as *C*; while for the 2010 event, it is the opposite. Seeing only the absolute misclass label around these water bodies will imply that a geospatial predictor related to the water bodies may improve the model; however, looking deeper into the absolute misclass kind also implies that it may need other predictors to balance it out. The 2011 event also has considerable overpredictions around the Avon River and considering that *NC* misclassed as *C* can be viewed as "overprediction", this highlights the overprediction extent for the 2011 event.

As noted in previous discussions, there are some identified avenues for improvement of the model. This includes the possibility of using Rq_{c1Ncs}, the averaged q_{c1Ncs} for the first 10 m of the subsurface, to replace RI_C and Rq_{c1N}. This is because q_{c1Ncs} incorporates the fines content, *FC*, which is derived from I_C in this study, into q_{c1N}. This allows for a more robust liquefaction resistance quantification, showing the effect of fines on the liquefaction resistance more directly. Alongside this Rq_{c1Ncs}, the soil type aspect of liquefaction resistance can be represented by the thickness of layers falling into particular I_C ranges. For example, the thickness of liquefiable layers based on Robertson and Wride's (1998) boundary, $I_C < 2.6$, or clean sands following Cubrinovski et al.'s (2019) suggested boundary ($1.3 < I_C \leq 1.8$). Non-uniform averaging is an option to give more bias to upper layers, as shallower layers are more likely to influence the probability of ejecta. The simple definition of H_{cap} currently employed has the advantage of being replaced by the GWT in areas where the subsurface is known to be predominantly liquefiable ($I_C < 2.6$) or be reasonably estimated by a practitioner with sufficient geomorphological and/or geological knowledge of the area. It can be improved by incorporating cone penetration resistance, more akin to the original definition by Ishihara (1985), but will lose out on the advantage previously mentioned. On a related note, Geyin et al. (2021) has pointed out that there are uncertainties in relation to layers under the GWT being partially saturated in some cases, and this can possibly increase the H_{cap} of a site. There is also the possibility of utilizing event-specific parameters to improve the model such as distance from the event or magnitude. However, attempts to incorporate these into the PGA did not significantly improve the model's performance. And lastly, a predictor related to proximity to water bodies is needed – although defining the limits as to what water bodies should be included is a key consideration.

90% of the CPTs in the Geyin et al. (2020) database were conducted between the 2011 and 2016 events, 0.3% after the 2016 event, and the rest in between the 2010 and 2011 events. Geyin et al. (2021) discussed whether the CPTs are representative of the site conditions across all events considered, where a key resource cited was Lees et al. (2015b). Lees et al. (2015b) noted that CPT measurements in Christchurch did not indicate significant strengthening across their studied earthquakes and majority of the CPT comparisons showed that the tip resistance generally remain unchanged. This makes the authors more confident in the model developed despite

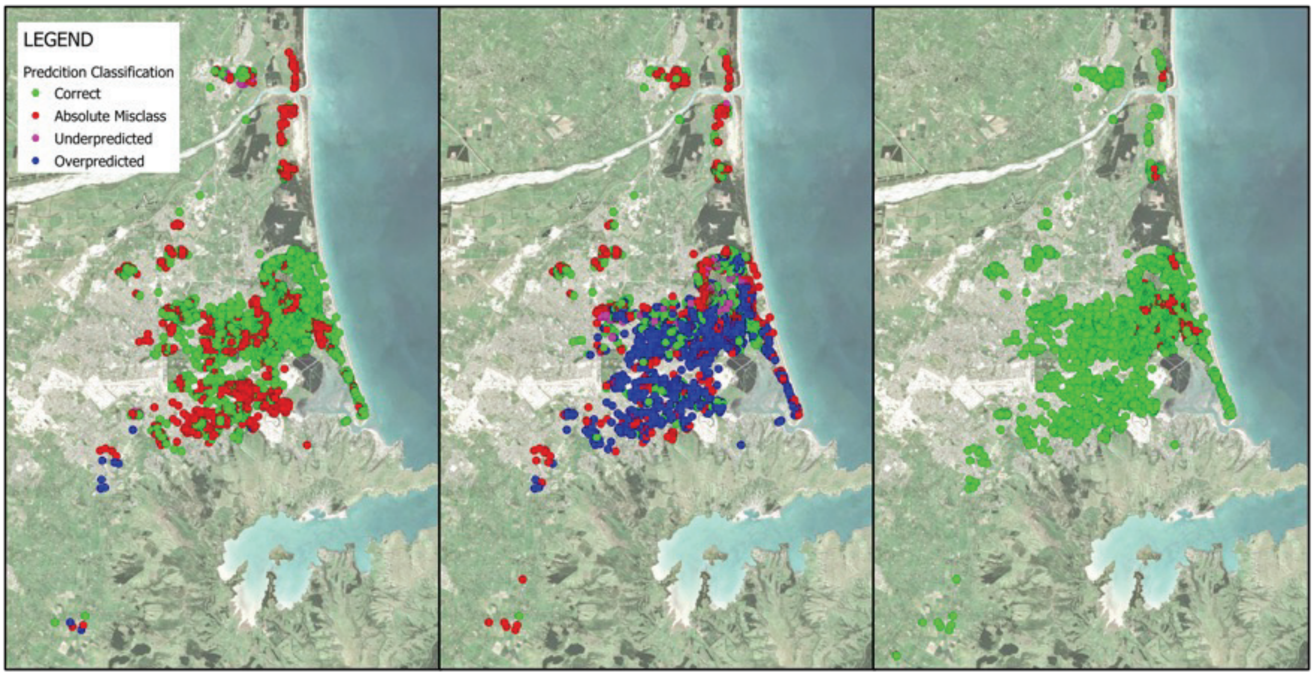

Figure 1: Model prediction classification for each of the 3 events: (left) 2010 Darfield, (middle) 2011 Christchurch, and (right) 2016 Valentine's Day earthquakes.

*Basemap sourced from the LINZ Data Service (https://data.linz.govt.nz/layer/109401-nz-10m-satellite-imagery-2021-2022/) and licensed by Sinergise Ltd for re-use under the Creative Commons Attribution 4.0 International (https://creativecommons.org/licenses/by/4.0/). [Colors modified] using the same CPT for events across a 5+ year space. It is also worth noting that Geyin et al. (2021) highlighted that there are no standard or best practice in terms of the soil investigation timing relative to the earthquake event. This is understandable as soil investigations are generally expensive. Thus, liquefaction developments generally use what is available, and in this case, much of the CPTs were done after most the earthquake events included.

4 CONCLUSIONS

Using the binary logistic regression with a modified usage based on several thresholds, a model to predict the severity of liquefaction ejecta in a site was developed. The model utilized simplified representations of the subsurface derived from CPT data to represent different aspects of the subsurface's liquefaction resistance. The final form of the predictors utilized a form similar to the concept of the factor of safety, with the load represented by the PGA. The model's performance was measured using a suite of metrics to provide better insights: the balanced accuracy highlighted the need for different metrics for imbalanced databases, showing how a good performance in the majority group can pull up the accuracy metric, while the analysis using TPR, CPR, and AMR showed that despite the model's performance varying for each level, the model is ultimately more "conservative" as applied to the development database. The per-event and spatial analysis highlighted differences in the performance across events and space, leading us to identify various possible avenues to improve the model.

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NZGS Symposium 2025

Liquefaction Characteristics of Gravelly Soils Prepared by Water Sedimentation Method

L. Wang[1], G. Chiaro[1], S. Rees[1], C. Cappellaro, A. Pokhrel[2]

[1]Department of Civil and Natural Resource Engineering, University of Canterbury, Christchurch

[2]Beca Limited, Christchurch

UNIVERSITY OF CANTERBURY
Te Whare Wānanga o Waitaha
CHRISTCHURCH NEW ZEALAND

INTRODUCTION

Gravelly soils (e.g., gravels, gravelly sands, sandy gravels) are often considered resistant to liquefaction. Yet, at least 32 global case histories, including three in New Zealand, have reported liquefaction in such soils. These events have caused significant damage to civil infrastructure, with alluvial deposits (Rollins et al., 2022) being particularly susceptible.

To investigate this phenomenon, a systematic and repeatable water sedimentation (WS) technique was developed to replicate characteristics of alluvial soil deposition. Specimens prepared by WS were tested under cyclic undrained triaxial loading to evaluate their liquefaction resistance.

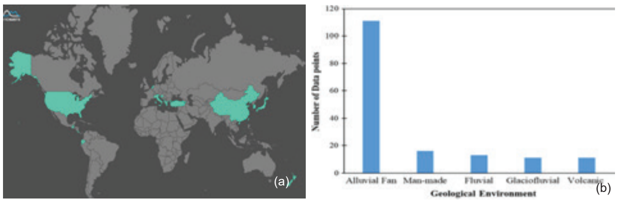

Figure 1. (a) Case histories of liquefaction in gravelly soils (Amcharts, 2023); (b) Different deposition environments (Rollins et al., 2022) .

MATERIALS

Selected sand and gravel mixtures were created by combining New Brighton sand, Dalton river washed sand and a well-graded round pea gravel. As illustrated in Fig. 2, these mixtures are representative of those found in documented liquefaction case histories.

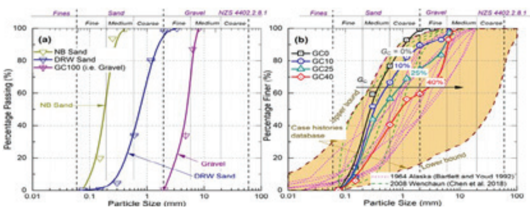

Figure 2. Material tested in this study (Pokhrel et al., 2024).

WATER SEDIMENTATION & SPECIMEN PPREPARATION

An optimal WS procedure (Fig. 3a) was established by considering drop height, water column height, pluviation rate, funnel opening size, and the number/thickness of deposited layers.

Specimen uniformity was verified using miniature cone penetration tests (CPTs) (Fig. 3c), and segregation was checked from frozen cross-sections.

To minimize disturbance and densification, specimens were drained, frozen, trimmed, and then placed in the triaxial cell for testing (Fig. 3b).

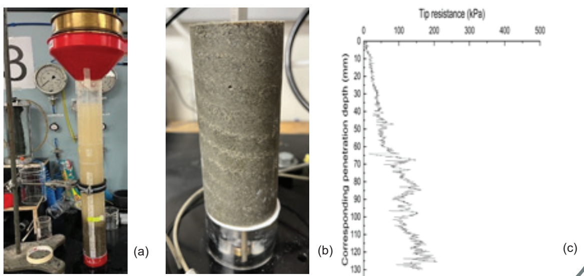

Figure 3. (a) WS set-up; (b) Example of frozen specimen; and (c) Typical miniature CTP results

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TRIAXIAL TEST RESULTS & DISCUSSION

Stress-controlled undrained cyclic triaxial tests were conducted on specimens subject to constant-amplitude sinusoidal axial load with a cyclic stress ratio (CSR) ranging from 0.14 to 0.5 at a frequency of 0.05 Hz using a pneumatic loading system.

- Typical undrained cyclic responses are presented in Fig. 4 for a gravelly soil specimen with D_r = 46% and G_c = 25% under 0.248 CSR.

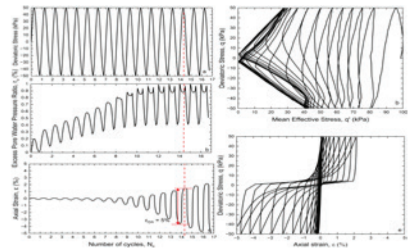

Figure 4. Typical undrained cyclic responses: (a) deviator stress, q; (b) excess pore water pressure, r_u; (c) axial strain, ε_a; (d) effective stress paths; (e) stress-strain relationship

- The r_u = 95% criterion was adopted in this study to define initial liquefaction criterion. Cyclic resistance ratio (CRR_{15}) was defined as the CSR value at 15 cycles of loading (N_c).
- The liquefaction resistance curve of WS specimens defined by the criteria of r_u = 95%, and 5% ε_{DA} are presented in Fig. 5 and 6. These curves, correspond to specimens with gravel content (G_c) = 0%, 10%, 25%, and 40%.
- To evaluate the fabric effects on liquefaction resistance, comparisons with experimental data available for specimens prepared by the moist tamping (MT) method by Pokhrel et al. (2024) are drawn in Fig. 7 as solid points and lines.

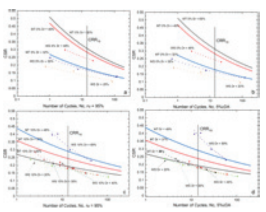

Figure 5. Liquefaction resistance curve (a) 0% G_c at r_u= 95%; (b) 0% G_c at 5% ε_{DA}; (c) 10% G_c at r_u= 95%; (d) 10% G_c at 5% ε_{DA}

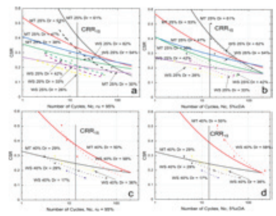

Figure 6. Liquefaction resistance curve (a) 25% G_c at r_u= 95%; (b) 25% G_c at 5% ε_{DA}; (c) 40% G_c at r_u= 95%; (d) 40% G_c at 5% ε_{DA}

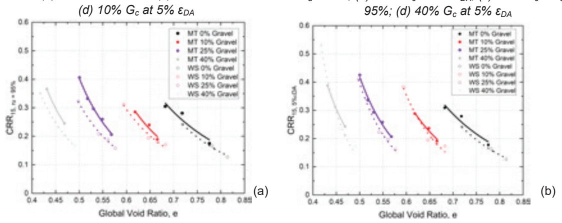

Figure 7. Correlation between global void ratio and (a) $CRR_{15,\, ru = 95\%}$; (b) $CRR_{15,\, 5\%\, \varepsilon DA}$

Through regression analysis of the experimental results, a power-form correlation was adopted to represent the relationship between *CRR* and *e*, expressed as $CRR = a\,(e)^{-b}$.

The relative position of the MT and WS curves under the same G_c case reflects the effect of soil fabric on the liquefaction resistance. The following observations can be made:

- At the same D_r, G_c, and *CSR*, WS specimens showed lower cyclic resistance than MT ones, highlighting the fabric effect on liquefaction resistance.
- The influence of soil fabric is insignificant at loose states and low gravel contents but becomes increasingly evident with higher D_r and G_c, where MT specimens consistently demonstrate greater cyclic resistance than WS specimens.

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Lessons Learned from the Liquefaction Analysis

Ageing Factor (K_{DR})

Sajjad Anwar, AECOM

Sajjad Anwar, Aecom

ABSTRACT

Ageing is known to enhance the liquefaction resistance of soils due to microstructural development processes such as cementation and secondary consolidation. However, the quantification of ageing effects remains highly uncertain and is not commonly incorporated into standard design practice. This study investigates the identification and assessment of microstructure development and its influence on liquefaction resistance through the combined use of Cone Penetration Testing (CPT) and Seismic Dilatometer Testing (sDMT). Two complementary approaches are employed: (1) evaluation of the empirical parameter K^*_G, derived from small-strain shear modulus (G_o) and normalized cone resistance (Q_{tn}), and (2) comparison of sDMT directly measured and CPT-estimated shear wave velocities (Vs) to infer stiffness enhancement due to soil ageing. An age-related liquefaction correction factor (K_{DR}) is calculated using the methodology of Hayati and Andrus (2009), allowing adjustment of cyclic resistance ratio (CRR) or cyclic stress ratio (CSR) to more accurately reflect the behavior of aged soils. The findings in this study highlight the importance of incorporating ageing effects into simplified liquefaction assessment procedures and provide a practical framework for integrating seismic and penetration test data to improve the reliability of liquefaction hazard evaluations in structured soils.

1. INTRODUCTION

It is widely acknowledged that ageing enhances the liquefaction resistance of soils. However, the effects of ageing are difficult to quantify and are typically not explicitly incorporated into standard design procedures such as those outlined in MBIE & NZGS Module 3 (2021). Early research by Youd and Perkins (1978) indicated that liquefaction resistance increases significantly with geologic age, primarily due to processes such as cementation and secondary consolidation. As a result, Cone Penetration Test (CPT) cone resistance (q_c) values generally increase with the age of the soil.

To account for ageing effects, it has been proposed that a correction factor be applied to either Cyclic Resistance Ratio (CRR) or Cyclic Stress Ratio (CSR), as follows (Arango et al., 2000; Lewis et al., 2004; Andrus et al., 2004b):

Equation 1: $CRR_K = CRR * K_{DR}$

Equation 2: $CSR_K = CSR / K_{DR}$

Where CRR_K and CSR_K are cyclic resistance and stress ratios corrected for age and cementation respectively; and K_{DR} is ageing factor to correct for influence of age and cementation on deposit resistance.

Currently, there is no international consensus on appropriate values for the ageing correction factor K_{DR}. As a result, the commonly recommended practice is to apply ageing corrections only when supported by shear wave velocity (Vs) data. Shear wave velocity testing is considered more responsive to ageing effects, as both Vs and cyclic resistance tend to increase proportionally with time.

Clayton and Johnson (2013) suggested that the methodology developed by Hayati and Andrus (2009) can be used to quantify an age-related correction factor K_{DR}, based on the measured-to-estimated shear wave velocity ratio (MEVR), as follows:

Equation 3: $K_{DR} = 1.08 * MEVR – 0.08$

Where MEVR is Measured to Estimated shear-wave Velocity Ratio and is estimated using following relationship:

Equation 4: $MEVR = 0.0820 \log_{10}(t) + 0.935$

Where t is time in years since initial soil deposition.

To evaluate whether soils are aged, it is first necessary to determine if the soils exhibit significant microstructure development, such as cementation or bonding. These characteristics can have a considerable influence on in-situ soil behaviour and can affect the reliability of classification systems based on in-situ testing methods.

The integration of data from Cone Penetration Testing (CPT) and seismic CPT ($_S$CPT) or seismic Dilatometer Testing (sDMT) provides a valuable means of identifying potential microstructure development. Eslaamizaad and Robertson (1996) and Schnaid (2009) suggested that the relationship between small-strain shear modulus (G_o), net cone resistance (q_n), and normalized cone resistance (Q_{tn}) can be used to infer the presence of microstructure. This is based on the observation that both ageing and bonding tend to increase the small-strain shear modulus (G_o) significantly more than they increase large-strain strength parameters, such as q_n and Q_{tn}.

Building on this concept, Robertson (2009) and Schnaid (2009), as well as Schneider and Moss (2011), proposed an empirical approach to identify sandy soils with microstructural development. They introduced the parameter K^*_G, defined as a function of small-strain stiffness and cone resistance, to evaluate the degree of structure in the soil. K^*_G is calculated using the following relationship:

Equation 5: $K^*_G = (G_o/q_n)\,(Q_{tn})^{0.75}$

Where: G_o is small strain shear modulus (shear strain, $\gamma_s < 10^{-4}$ %) measured using elastic theory as below.

Equation 6: $G_o = \rho\, V_s^2$

Where: ρ is the mass density of the soil ($\rho = \gamma/g$) and V_s = shear wave velocity measured using a downhole technique during pauses in the CPT or DMT.

q_n = net cone resistance = ($q_t - \sigma_{vo}$), where q_t is corrected (i.e., q_c corrected for pore water effects) cone resistance, σ_{vo} is vertical total stress.

Q_{tn} = Normalized cone resistance = $(q_n/P_{a2})\,(P_a/\sigma'_{vo})^n$, calculated using a stress exponent (n) that varies with soil type via soil behaviour type index (I_c).

Where n = linear stress exponent = $0.381 (I_c) + 0.05 (\sigma'_{vo}/P_a) – 0.15$.

P_a is a reference pressure in the same units as vertical effective stress σ'_{vo} (i.e., P_a = 100 kPa if σ'_{vo} is in kPa) and P_{a2} is a reference pressure in the same units as q_n (i.e., P_{a2} = 0.1 MPa if q_n is in MPa).

A threshold value of K^*_G is used to interpret the presence of microstructure:

- **K^*_G < 330** indicates unstructured, young, and uncemented soils
- **$K^*_G \geq$ 330** suggests significant microstructural development, with increasing values indicating stronger bonding or cementation

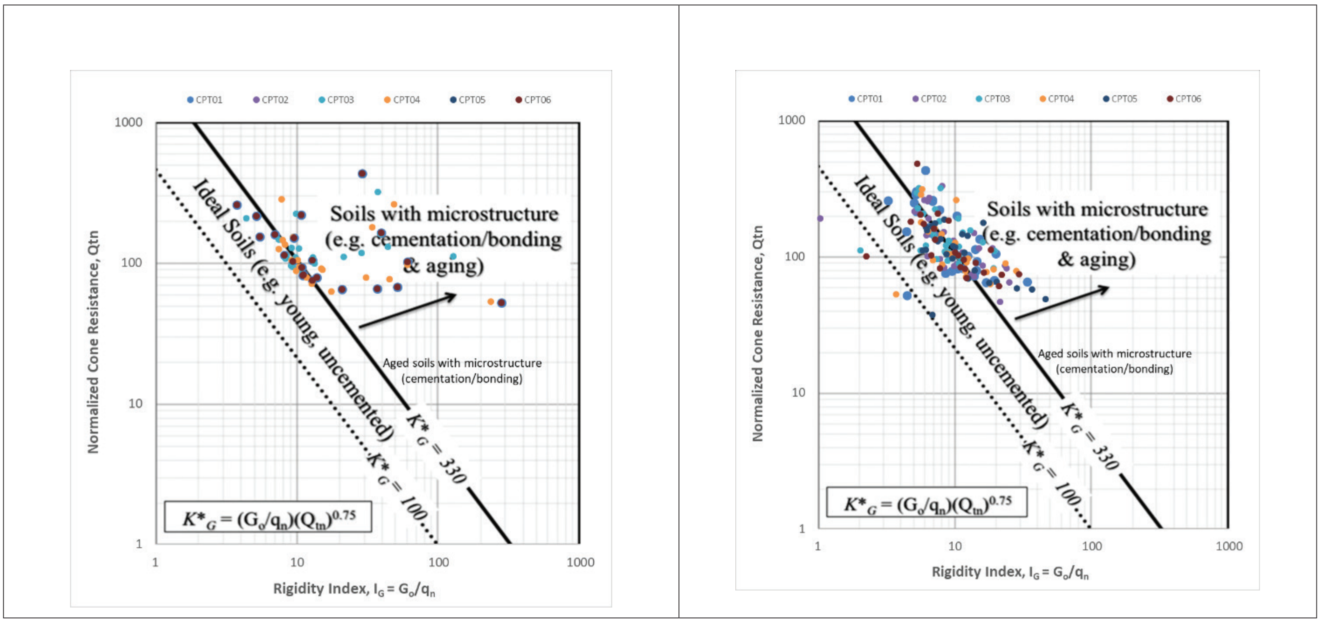
This method provides a practical framework for assessing the age and structure of soils based on in-situ test data.

2. SOIL MICROSTRUCTURE DEVELOPMENT ASSESSMENT METHOD

Two methods are discussed to assess the development of microstructure in the soils.

2.1 METHOD 1

In the first method, combined data from the CPT and sDMT are used to assess the potential development of microstructure in the soils. The small-strain shear modulus (G_o), measured from the sDMT, is used in conjunction with normalized cone resistance parameters (q_n and Q_{tn}) obtained from the CPT to calculate the empirical parameter K^*_G. The results of this assessment are presented in Figure 1. As shown in Figure 1, the data trend towards K^*_G values of 330 or higher, indicating the likely presence of microstructure, such as cementation or bonding within the soil profile.



Go based on sDMT01

Go based on sDMT02

FIGURE 1: Q_{tn} – I_G Chart to Identify Soils with Microstructure Development (modified from Robertson, 2016)

2.2 METHOD 2

In the second method, microstructure development in the soils is assessed by comparing shear wave velocities (V_s) estimated from CPT data with those directly measured in situ using the sCPT or sDMT. A plot illustrating this comparison with depth is presented in Figure 2.

As shown in Figure 2, the estimated and measured V_s values correlate reasonably well within the upper 5 m of the soil profile, corresponding to the upper Holocene sand layer. This suggests that these materials are relatively young and not significantly influenced by ageing or cementation. Therefore, standard liquefaction analysis procedures are considered appropriate for these layers.

Below approximately 5 m depth, however, the V_s values estimated from CPT data are generally lower than those directly measured using sDMT. This indicates that the small-strain shear stiffness (G_o), derived from measured V_s, is higher than would be expected based on the large-strain strength indicated by CPT-derived cone resistance (q_n). The q_n is a measure of large strain soil strength, and the shear wave velocity (V_s) is a measure of small strain soil stiffness (G_o). This discrepancy suggests that the soils at these depths are aged and have undergone microstructure development, such as cementation or bonding, which tend to increase small-strain stiffness (G_o) more significantly than large-strain strength soil strength (q_n).

3. SHEAR WAVE VELOCITY (V_S) MEASUREMENT METHODS

Shear wave velocity (V_s) can be measured directly using seismic CPT or DMT or indirectly using published empirical correlation of Robertson (2009) using CPT cone resistance data.

3.1 DIRECT MEASUREMENT WITH SEISMIC CPT (sCPT) AND DMT (sDMT)

A surface source, such as a horizontal impact, is used to generate a shear wave that propagates through the soil. A geophone or receiver is placed at a known depth within the CPT cone or DMT blade. The arrival time of the shear wave at the receiver is recorded. By knowing the distance between the source and the receiver and the travel time, the shear wave velocity (V_s) is calculated as V_s (m/sec) = distance (d) / travel time (t). The V_s is measured during pauses at regular internal during CPT or DMT testing usually at 1m to 1.5m depth intervals, which generates a continuous profile of V_s as shown in Figure 2.

3.2 INDIRECT ESTIMATION USING EMPIRICAL CORRELATION

For this paper, the method adopted is based on Robertson (2009) using the relationship below.

Equation 7: $V_s\ (m/sec) = [a_{vs}\ (q_n / P_a]^{0.5}$

Where a_{vs} is the shear wave velocity cone factor equivalent to $10^{(0.55\ Ic\ +\ 1.68)}$

4. AGEING CORRECTION FACTOR (KDR) ASSESSMENT

4.1 GENERAL

The combined CPT (Q_{tn}) and sDMT (G_o) data, as outlined under Section 2, indicates that microstructure (e.g., cementation or bonding) is likely present in the soils below approximately 5 m depth. In accordance with the approach recommended by Andrus, Hayati, and Mohanan (2009), an age-related liquefaction resistance factor (K_{DR}) can therefore be assessed for liquefaction analysis.

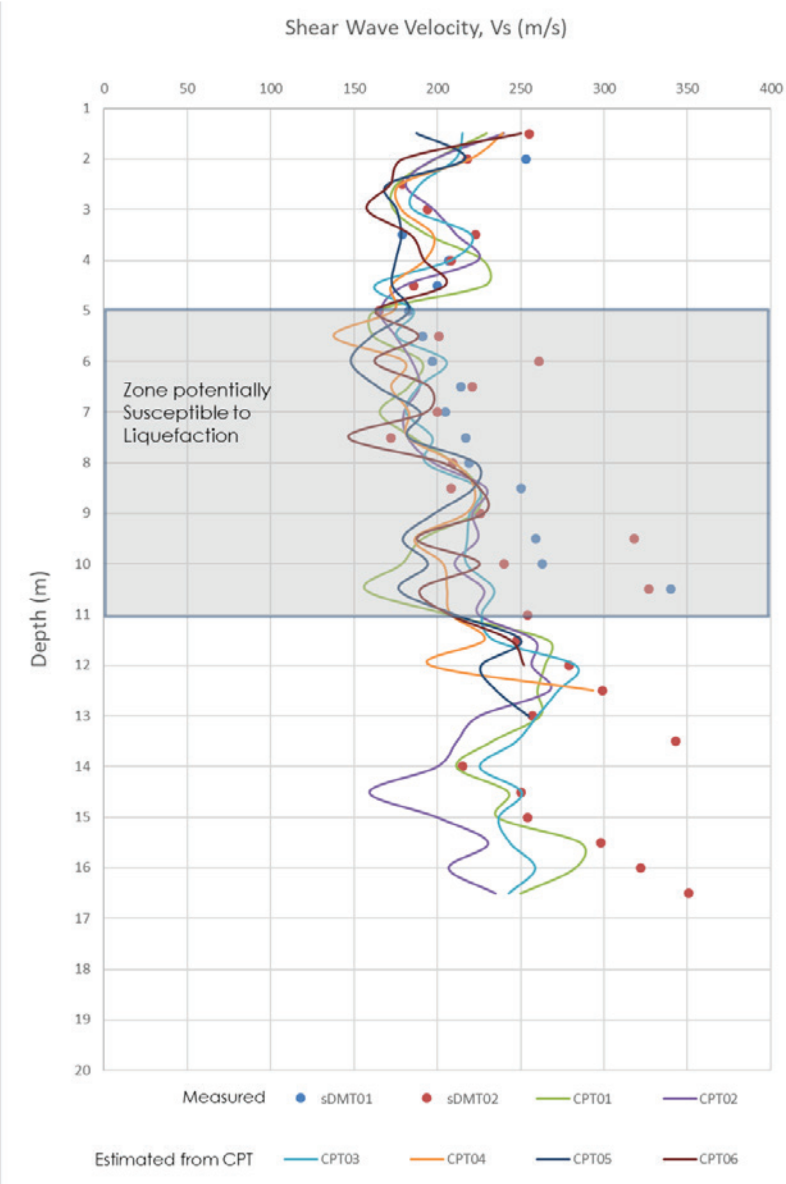


FIGURE 2: Comparison of Measured and Estimated Shear Wave Velocities

4.2 HOLOCENE AGE DEPOSITS

The data presented in this paper is for Holocene aged deposits. The Holocene aged soil deposits are estimated to be approximately 11,700 years old. Using the Holocene age, t = 11,700 years in Equation 4, the Measured to Estimated Shear Wave Velocity Ratio (MEVR) is calculated as 1.27. This yields an age-related correction factor of 1.3 based on equation 3.

MEVR = $0.0820 \log_{10}(t) + 0.935 = 0.0820 \log_{10}(11700) + 0.935 = 1.27$

$K_{DR} = 1.08 * MEVR - 0.08 = 1.08 \times 1.27 - 0.08 = 1.29 \simeq 1.3$

4.3 TAURANGA GROUP ALLUVIUM

The Tauranga Group alluvium in the Auckland and Tauranga regions dates from the late Pliocene to Pleistocene epochs, with ages generally ranging from about 2 million years ago to as young as 128,000 years. These alluvial sediments are composed of sand, silt, and gravel, often with layers of peat, pumiceous material, and volcanic ash. Where the combined CPT (Q_{tn}) and sDMT (G_o) data, as outlined under Section 2, indicates that microstructure (e.g., cementation or bonding) is present, the age-related liquefaction resistance factor (K_{DR}) may be assessed for liquefaction analysis following recommendation by Andrus, Hayati, and Mohanan (2009).

For Pliocene Age (2 million year):
MEVR = $0.0820 \log_{10}(t) + 0.935 = 0.0820 \log_{10}(2e6) + 0.935 = 1.45$
$K_{DR} = 1.08 * MEVR - 0.08 = 1.08 \times 1.45 - 0.08 = 1.49$

For Pleistocene Age (128,000 year):
MEVR = $0.0820 \log_{10}(t) + 0.935 = 0.0820 \log_{10}(128e3) + 0.935 = 1.35$
$K_{DR} = 1.08 * MEVR - 0.08 = 1.08 \times 1.35 - 0.08 = 1.38$

For Late Pleistocene Age (15,000 year):
MEVR = $0.0820 \log_{10}(t) + 0.935 = 0.0820 \log_{10}(15e3) + 0.935 = 1.28$
$K_{DR} = 1.08 * MEVR - 0.08 = 1.08 \times 1.28 - 0.08 = 1.30$

In summary, when microstructure development is considered in Tauranga Group alluvium, the seismic demand expressed as CSR (per Equation 2) may be reduced by approximately 30 to 49%, or equivalently, the soil resistance expressed as CRR (per Equation 1) may be increased by 30 to 49% in liquefaction analysis.

4.4 EAST COAST BAYS FORMATION (ECBF)

ECBF residual soils in Auckland originate from the Early Miocene epoch and are approximately 20 to 25 million years old. They have developed through extensive weathering and alteration of the Waitemata Group sedimentary rocks. Due to their age and cemented structure, significant microstructure is likely present. As a result, these soils are generally excluded from liquefaction risk assessments. Nonetheless, where required the age-related liquefaction resistance factor (K_{DR}) following the recommendation by Andrus, Hayati, and Mohanan (2009) of 1.58 may be considered.

5. SIMPLIFIED LIQUEFACTION ANALYSIS INPUT

Based on this study example (see Figure 2), the assessed K_{DR} value of 1.3 can be adopted in the simplified liquefaction penetration data-based analysis method for soils between 5 m and 20 m depth, where ageing and microstructure effects are considered significant. Based on this particular data as presented on Figure 2, no ageing correction is applied above 5 m, where soils are interpreted to be unstructured and uncemented.

A snippet of the Geologismiki's CLIQ software version 3 input menu, illustrating the field for entering the K_{DR} value used for age-related liquefaction resistance correction, is presented in Figure 3.

6. CONCLUSION

This study demonstrates that soil ageing and microstructural development significantly influence liquefaction resistance, particularly in aged and structured alluvial deposits. By integrating Cone Penetration Testing (CPT) and Seismic Dilatometer Testing (sDMT), the presence of microstructure can be effectively assessed using the empirical parameter K^*_G and measured-to-estimated shear wave velocity (V_s) comparisons. The application of an age-related correction factor (K_{DR}), following the methodology of Hayati and Andrus (2009), provides a practical means to adjust cyclic resistance ratio (CRR) or cyclic stress ratio (CSR) in simplified liquefaction analyses.

Key findings include:

- Based on site soil profile, the above 5 m depth, young, unstructured soils show no significant ageing effects, supporting the use of standard penetration-based liquefaction procedures. The soils below ~5 m depth show enhanced small-strain stiffness and $K^*_G > 330$, indicating significant microstructure development due to ageing and bonding.
- Age-related correction factors (K_{DR}) for Holocene to Pleistocene-aged deposits range from 1.3 to 1.49, with older, cemented soils (e.g., ECBF residuals) requiring higher corrections up to 1.58. Incorporating K_{DR} into liquefaction assessment reduces overestimation of seismic demand and improves reliability of hazard evaluations in aged soils.

Overall, this study highlights the importance of considering soil microstructure and ageing effects in liquefaction analyses. Simplified methods can be enhanced through site-

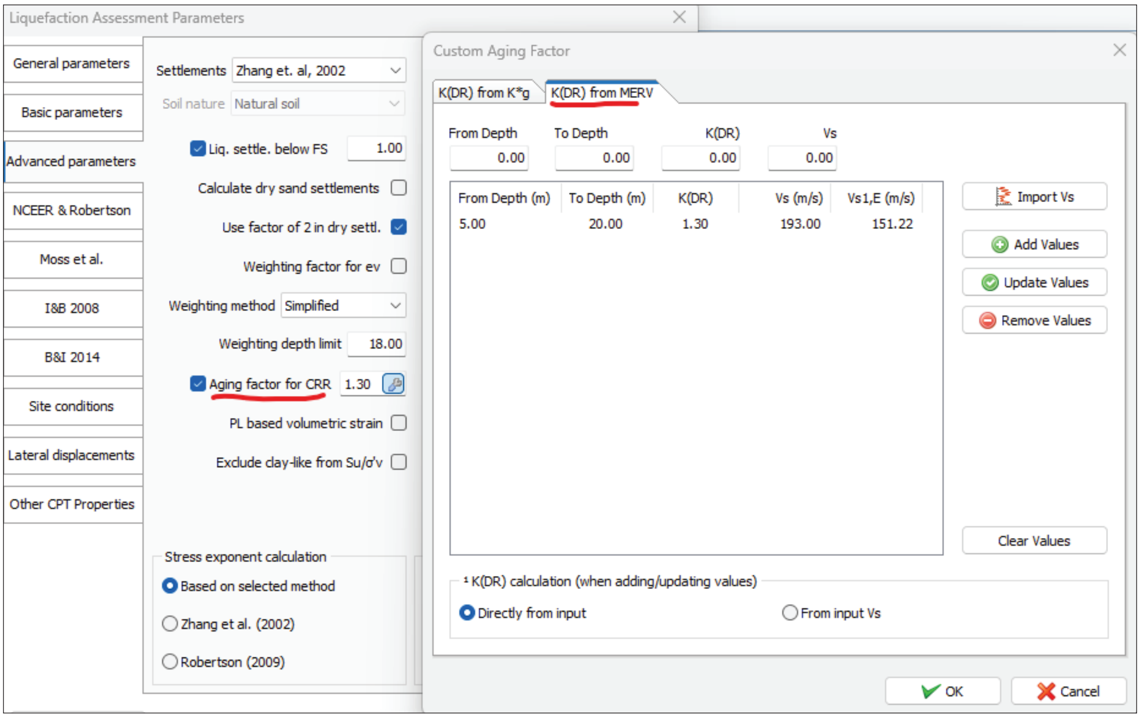


FIGURE 3: CLIQ Software Input for K_{DR}

specific assessment of K^*_G and V_s, enabling more accurate evaluation of liquefaction potential and guiding appropriate mitigation measures where necessary.

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Geogrid reinforced load bearing bridge abutments – Design and Construction

Dr Jan Kupec, Aurecon New Zealand, Christchurch, New Zealand
Don Mackintosh, Aurecon New Zealand, Auckland, New Zealand
Aidan Thorp, Tonkin + Taylor, Melbourne, Australia
Gordon Ashby & Andy Walsh, Tonkin + Taylor, Auckland, New Zealand
Michael Spies, Fulton Hogan, Auckland, New Zealand

Jan Kupec

A new highway to the north of Auckland in New Zealand features four, up to 35m long, single span bridges. The project used geogrid reinforced soil (GRS) abutments to support the integral bridge bank seats, rather than steel soil reinforcing elements. The use of geogrids for abutments is unusual in NZ and Australia as 'extensible' reinforcements are not generally permitted under the NZ Transport Agency (NZTA) guidance. The project received permission to use geogrid reinforced soil abutments on the above project and this paper describes design considerations and observations from construction and monitoring of instrumentation. The design was undertaken using North American design guidance from FHWA and AASHTO, but with NZ specific seismic design provisions. One bridge was founded on cohesive (alluvial) subsoils that were prone to consolidation settlement in excess of 200mm and preloading was used to meet project timelines and strict displacement limits. The designers reduced post-construction settlements by applying a preload at the top of the bridge abutment comprising five vertical multi wire strand anchors tensioned to a combined load of 1,050 tons. Additionally 3 metres height of soil preload was placed behind the anchors.

1 INTRODUCTION

A new motorway connection north of Auckland in New Zealand will require several bridges, including four single span bridges constructed with geogrid reinforced soil (GRS) bridge abutments to support the integral bridge bank seats, rather than more common steel reinforcements. Geogrid reinforced soil abutments are not new, but they are unusual in New Zealand and not been widely used on the main State Highway network (Kupec, 2021). The project team requested and was granted a Departure from established standards, namely NZTA Bridge Manual v3.3, (New Zealand Transport Agency, 2018). Thus, allowing the use of extensible polymeric reinforcements on the project. Amongst other design considerations the post construction strain in the geogrid was limited to 0.5% strain development (creep) to limit deformations.

The design was undertaken using the North American design guidance following the recommendations of the FWHA (Federal Highway Administration) and

AASHTO (American Association of State Highway and Transportation Officials) design manuals with actions (loads and deflections) taken from NZ local codes of practice. Two different geogrid types and facing systems were chosen for the shallow founded bridge abutments. Bridges 1, 3 and 4 were designed with reinforced concrete panels with cast in HDPE geogrid tails that were connected with a bodkin (proprietary rod like connector) to the primary reinforcements.

One bridge, Bridge 2, as detailed in this paper, was located on settlement prone firm to stiff cohesive soil and the designers anticipated that consolidation settlement of around 200mm would occur. Thus, a different facing system was chosen using a geogrid wraparound soft facing to allow for larger construction deformations to occur. The permanent non-load bearing facing will comprise of precast full height panels that will be erected after consolidation has completed and prior to bridge deck placement.

The GRS abutment for Bridge 2 is up to 6.5m high above the existing ground level and supports an integral bridge with 26m span and 17° skew. The bridge Super T beams are supported on a 16m long, 2.7m high, 2.5m wide concrete bank seat setback 0.5m from the front face of the GRS abutment front face with a service loading of 190kPa, of which 38kPa was live load.

Construction is underway (October 2025), with the abutments located either side of East Coast Road at full height and preload applied. Preload has been applied using five 30m long 13-wire strand anchors that were drilled through steel casings installed within the GRS abutment, down into underlying rock. The anchors are positioned 1.5m back from the front face of the GRS wall and tensioned against a steel reaction frame to distribute load via concrete blocks atop 2.4m wide steel plates. This provides an equivalent surcharge load of 220kPa. The approach to drive out deformations of the geogrid reinforced soil block and underlying soil through prestressing was developed in New Zealand but is based on Japanese research in the early 2000s.

2 BACKGROUND

Bridge 2 is being constructed across East Coast Road, just east of the State Highway 1 (SH1) main alignment (Figure 1). The hills surrounding the BR02 site are dominated by rocks of the Northland Allochthon. These rocks detached from the main emplacement of Allochthon in Northland and slid southwards into the deep Waitemata Basin approximately 20 million years ago. The emplacement onto Northland and subsequent sliding southwards into the Waitemata basin has resulted in intense shearing of softer rocks and more brittle breeciation of harder rocks. The Northland Allochthon rocks beneath the BR02 site and in the surrounding hills are comprised of intensely sheared mudstones with trace amounts of other rocks including indurated siltstones, limestone and seams of extremely weak red, brown and green shales which often form persistent shear surfaces. Weathering and erosion of the Northland

FIGURE 1 Site setting

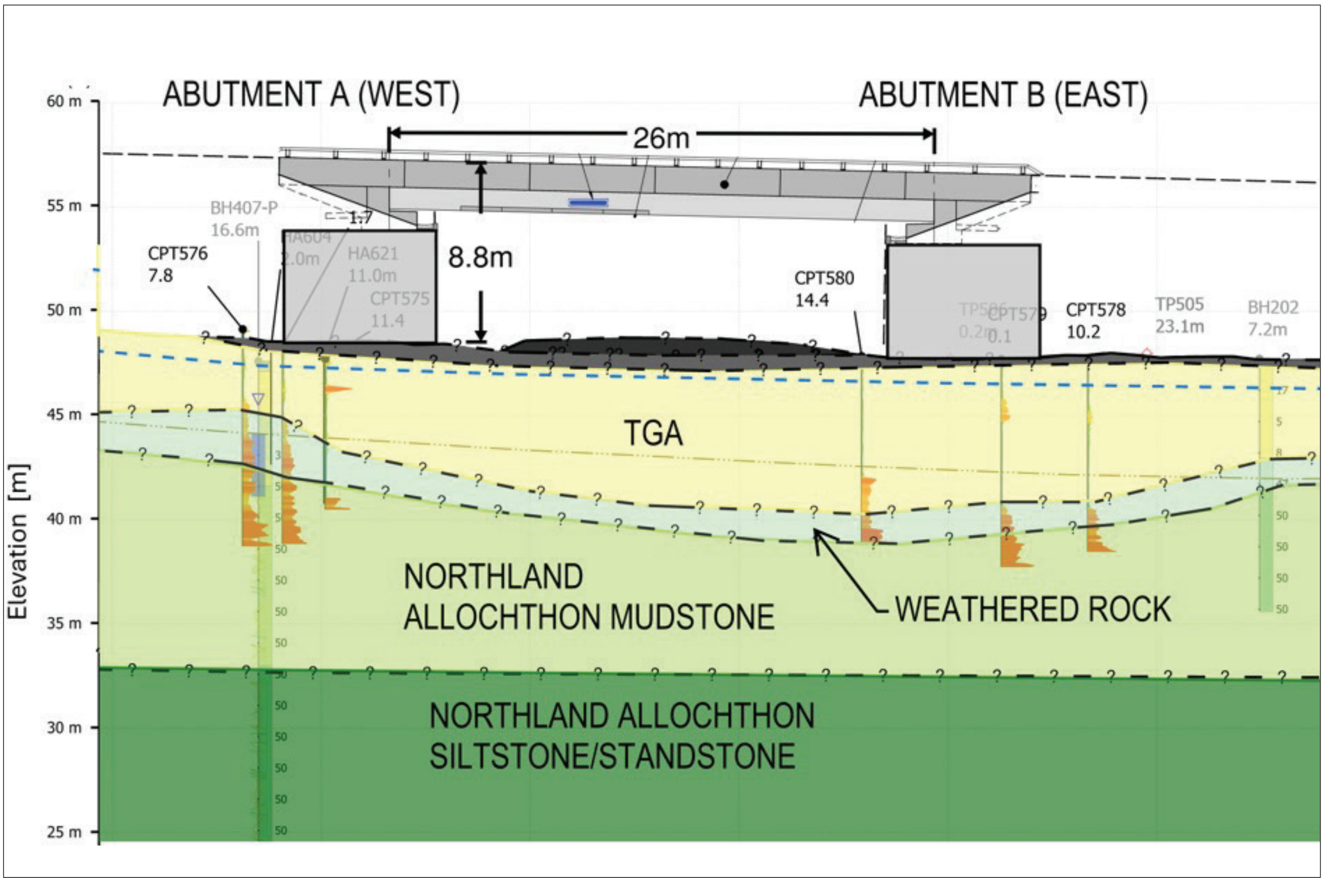


FIGURE 2 Geological setting

Allochthon rocks in the surrounding hills has resulted in the accumulation of alluvium in the valley floor derived from these rocks. Often being fine grained in nature comprising Clay and Silt with occasional lenses of coarser colluvium comprised of clays, silts with varying amounts of gravel sized fragments of rocks making up the surrounding hills. The alluvium beneath the abutments comprised firm to stiff silts and clays and range in depth between 3m and 7m below ground. The contact between the alluvium and the mudstone has been correlated to a series of CPTs at each abutment with the transition marked by a sharp increase in the measured cone resistance. Observations during construction have confirmed the contact to be within +/- 0.5m of that derived from the CPT's.

Bridge 2 is being constructed across the East Coast Road, just north of the State Highway 1 (SH1) main alignment and the proposed SH1 Southbound on-ramp. The existing ground contours are generally flat. A geological long section is shown in Figure 2. Groundwater is at approximately 1m below the surface and varies with precipitation.

3 TYPICAL NEW ZEALAND STANDARD DESIGN SOLUTIONS

Common design solutions in New Zealand and Australia are to pile the bridge structures and construct the approach embankment as a MSE (Mechanically Stabilised Earth) structure (Figure 3). Given the earthquake propensity in NZ, the approach embankment is commonly designed as a heavily geogrid reinforced soil block, often on improved ground using stone columns, soil cement mix or CFA columns or lattice structures. Where the MSE body is integral to the MSE embankment the piles are often sleeved providing a 75 to 150mm airgap to allow for seismic movement and isolate the piles from settlement and horizontal movement. The approach embankment will only be connected to the bridge with a concrete settlement slab to isolate the bridge from seismic shaking or be allowed to settle independently.

Where the ground is not prone to seismically triggered liquefaction or excessive settlement then shallow founded abutments are considered. However, the only permissible option in NZ is to use metallic reinforcements. Geogrid reinforcements for abutments were excluded from the use on the State Highway network by NZTA due to their potential for creep and unknown seismic performance, refer to Chapter 6 (New Zealand Transport Agency, 2018). MSE retaining walls and slopes, however, are common.

4 DESIGN CONSIDERATIONS

The region where the project is located is NZ's lowest seismic area. The design accelerations were derived using a Site-Specific Seismic Hazard Assessment (SSSHA) but the minimum actions set out in the NZTA Bridge

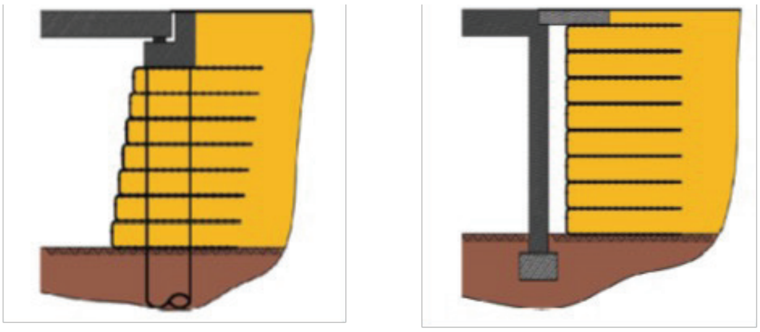


FIGURE 3 Common bridge support in NZ

Manual are higher and those were adopted, with Damage Control Limit State (DCLS) Peak Ground Accelerations (PGA) of 0.19g, and Collapse Avoidance Limit State (CALS) PGAs of 0.29g. The site is directly adjacent to a local road and the State Highway 1 corridor and thus relatively constrained in access.

The GRS abutment internal and external design checks have been completed using the Simplified Method in accordance with AASHTO (2020) / FHWA GEC 011. Strength Reduction Factors (SRFs) for sliding, bearing and passive resistance for static load combinations were chosen in accordance with the NZ Building Code B1/VM4, (Ministry of Business, Innovation and Employment, 2023). SRFs for sliding and passive resistance for seismic load combinations were assumed to be 1.0, with utilisation above 100% accepted for sliding modes as allowed by a performance-based design approach, where permanent displacements are compared against structural limits. The SRF for bearing in seismic loading was the same as the static SRF.

Strength reduction factors for geogrid reinforcement are in accordance with AASHTO (2020) and seismic inertial loads have been specified by the structural engineer, (AASHTO, 2020). The in-service bearing pressure under the bank seat footing was limited to 200kPa in line with FHWA-HRT-17-080, (US Department of Transportation, 2018).

The geogrid reinforcement and GRS abutment stability has been designed for rare and extreme events including a provision for sliding of the bank seat, but preventing unseating of the bridge. The GRS abutment has been checked against external bearing, sliding and overturning failure of either the bank seat footing or the combined bank seat / GRS abutment structure. The maximum foundation eccentricity of the bank seat and MSE wall was limited to 1/3 width in accordance with AASHTO (2020).

Due to the alluvial and compressible soils (consolidation) below the bridge footprint, the designers chose ground improvement by surcharging to drive out primary consolidation settlement. The surcharge was calculated to be approximately 120kPa.

5 FINAL DESIGN SOLUTION

Figure 4 shows the final design. Prefabricated vertical band drains (wicks) were installed at 1.5m centres in a triangular grid through the alluvium to refusal on the underlying weathered rock [purple]. A water main pipe and underground 11kV power line ran adjacent to the abutment block and were unable to be relocated. Timber piles [orange] were concreted into pre-drilled holes to support the road during excavation, and also to provide protection from lateral movement due to abutment construction. The alluvium was undercut to a depth 3.3m below existing ground level and backfilled with compacted granular material reinforced with an uniaxial geogrid at 400mm vertical centres [yellow]. The geogrid throughout the entire GRS structure was kept the same. The geogrid comprised of coated PET filaments and had a characteristic short term strength of 120kN/m. To aid compaction and to act as a formwork for the wrapped face, a galvanised steel basket was used. This basket has no long-term strength function once the wall is completed.

The GRS abutment body [green and blue] was formed from high quality crushed well graded gravel with a maximum particle size of 65mm in the lower 2/3 portion [green] and upper 1/3 with 40mm maximum particle size due to close spacings on the geogrids. The upper 1/3 portion of the abutment [blue] had geogrid reinforcements at reduced vertical spacing from 300mm [green zone] to 150 mm [blue zone] (closely spaced reinforced zone).

Two high strength geotextiles with a characteristic strength of 1,600kN/m were embedded from the face through the backfill to increase slope stability during construction and earthquake loading.

The facing panels are non-load bearing and are erected after primary consolidation settlement has been completed and prior to the bridge deck being placed. The gap in the bottom 2m of the wall between the panel and front face of the reinforced soil will be grouted to account for collision loads.

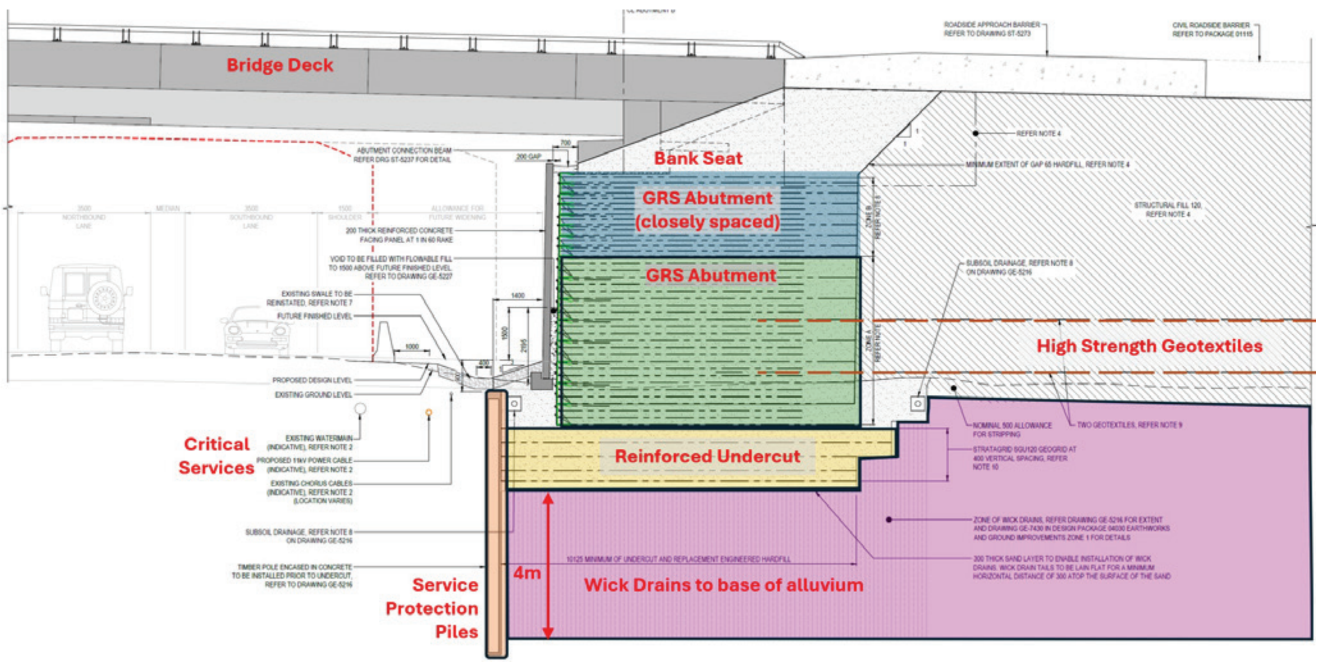


FIGURE 4 Main components of the GRS Abutments for Bridge 2 – eastern abutment shown

6 CONSTRUCTION AND MONITORING

The designers required a preload to be placed on top of the GRS abutment body to drive out primary consolidation settlement. Initially, the design required preloading the abutments with up to 5.5m high stacked concrete blocks on top of the finished vertical wrap around wall providing approximately 120kPa surcharge load held for around six months to allow settlement to occur. This was essential for the eastern abutment which was sited on over 7m of Tauranga Group Alluvium. The construction team was concerned with the feasibility and temporary work requirements of constructing tall concrete block surcharges adjacent to a busy road and therefore an alternative approach was developed. The alternative option was based on a Japanese research conducted on prestressed piers and abutments in the early 2000's. The Japanese research was led by Dr Uchimura and Prof Tatsuoka, (Uchimura, Tamura, Tateyama, Tanaka, & Tatsuoka, 2005), and (Uchimura, Tateyama, Tanaka, & Tatsuoka, 2003).

The construction team proposed to use five 30m long 13-wire multi wire strand anchors, installed through the GRS abutment in steel casings into the underlying rock. The five 13-wire strand anchors on each abutment were prestressed upon the completion of the abutment and apply 2,100kN (~210t) per anchor. The individual anchor loads are applied 1.5m back from the face of the wall, and distributed with steel beams, concrete blocks and 2.4m wide steel plates set back 300mm from the front face of the wall, as shown in Figure 5. This provides an equivalent surcharge across the width of the abutment beam footprint of 220kPa (equivalent to full in-service load). Thus, each abutment was preloaded with over 1,050t through the temporary anchors distributed through a temporary bank seat. An advantage of using the anchors was that the load could be applied in predefined stages to manage the risk of instability. It was hypothesized that if significant instability were to start to initiate, the anchors would begin to unload and therefore the system was considered more robust than a conventional preload. Additionally, the anchors could have been relatively quickly destressed if required for stability. To provide additional stability and increase the effective preload pressure, subsoil drains were installed through the undercut and pumped to 2.5m below ground level throughout the construction and preload period.

The surcharge was held in place for over six months to allow for consolidation settlement to occur. Regular restressing was necessary as the reinforced soil block was settling into the underlying alluvium. Conventional surcharge with soils was placed in front of the reinforced soil abutment (1.1m height for a width of 3.5m out from the face of the wall) to provide additional preload in front of the wall, and improve passive resistance and stability during preloading. A 3m high conventional preload was also placed behind the reaction frame for the ground anchors to provide additional preload.

Monitoring and instrumentation were provided to check total and differential settlements. The survey points were monitored in three dimensions, which

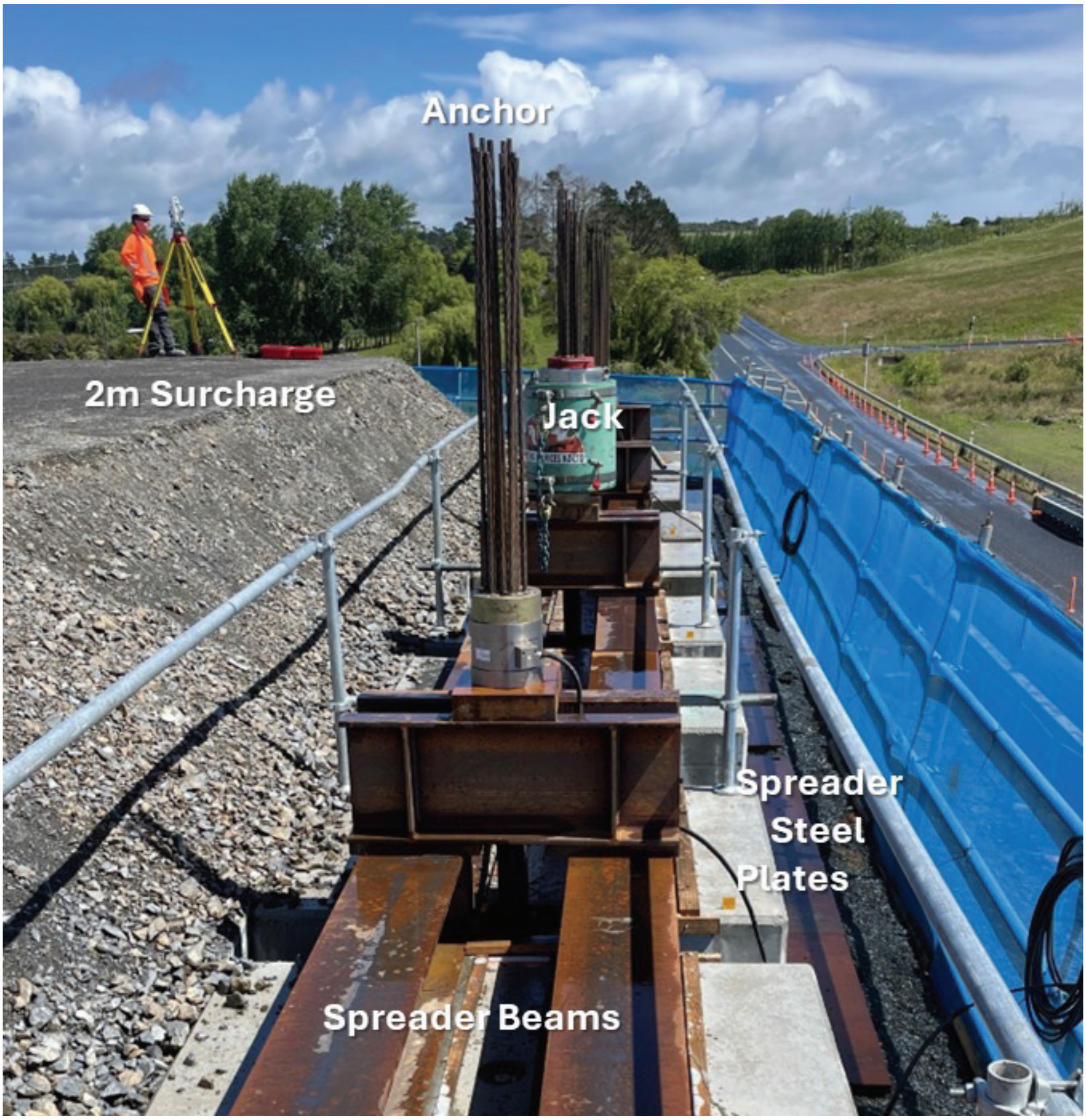


Figure 5 Eastern abutment at full height with prestressing and 2m of conventional surcharge in place

allowed for determination of lateral deformations of the soil block walls (front and wing walls). Geogrids were instrumented to measure strains at three distances back from the face at three separate elevations. The intent was to compare the closely reinforced highly loaded zone with the lower 2/3 of the wall. As well as compare the strain development along the geogrid reinforcements. Porewater pressure piezometers were installed at three locations beneath each abutment to monitor the development of porewater pressures during construction and prestressing.

Construction related settlements were in general agreement with the predictions, but dissipation of porewater pressures, despite provisions of wicks, was slower than anticipated at the eastern abutment, requiring a long hold time for the surcharge. The longer than anticipated hold time may be related to Allochthonous soils, once more confirming the difficult nature of these materials. Monitoring results are provided in the following figures and graphs.

Back calculation of the settlement and developed geogrid strains and overall behaviour of the block

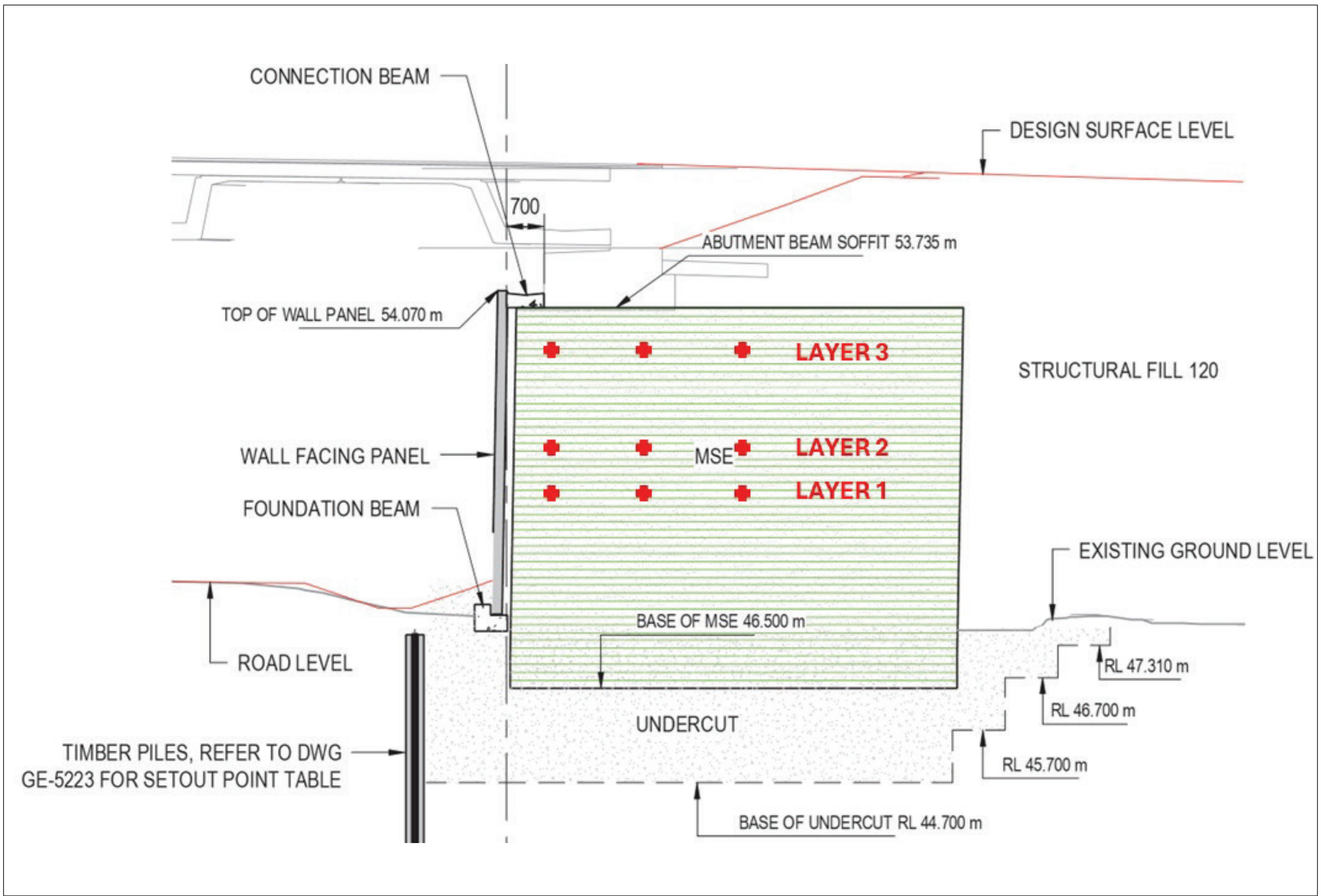


FIGURE 6 Geogrid strain monitoring locations

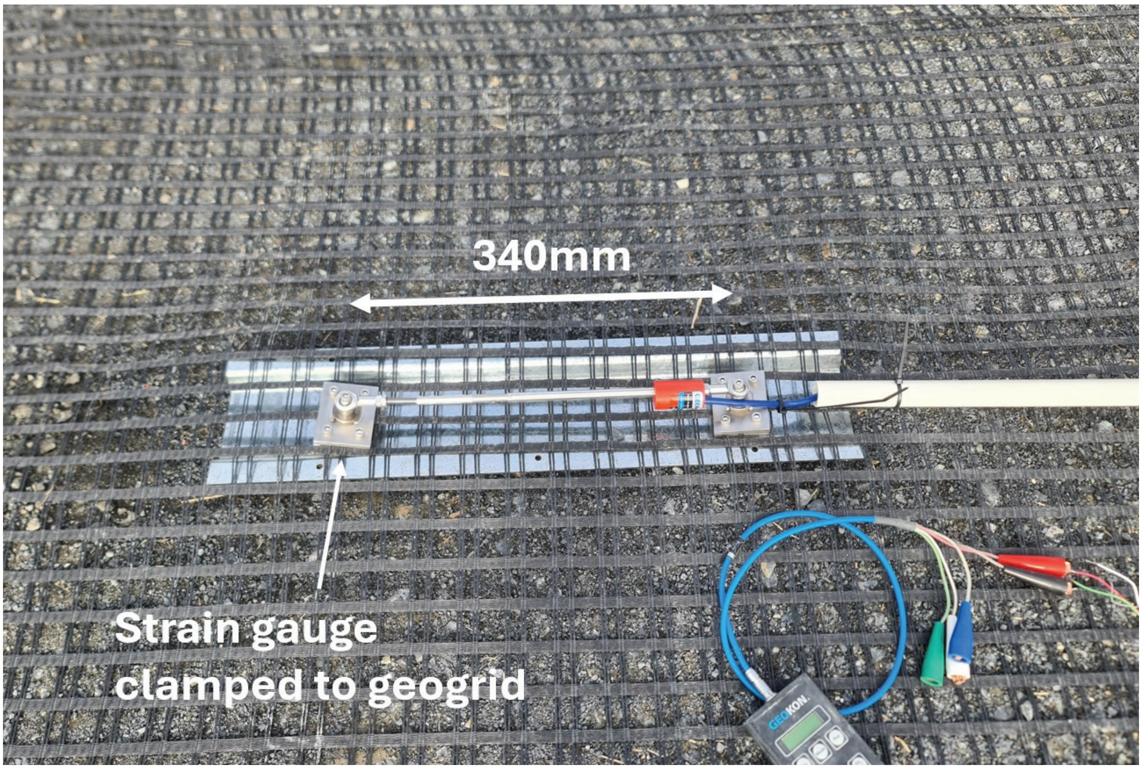


FIGURE 7 Photo of vibrating wire extensometer clamped to geogrid to enable strain in the geogrid to be calculated

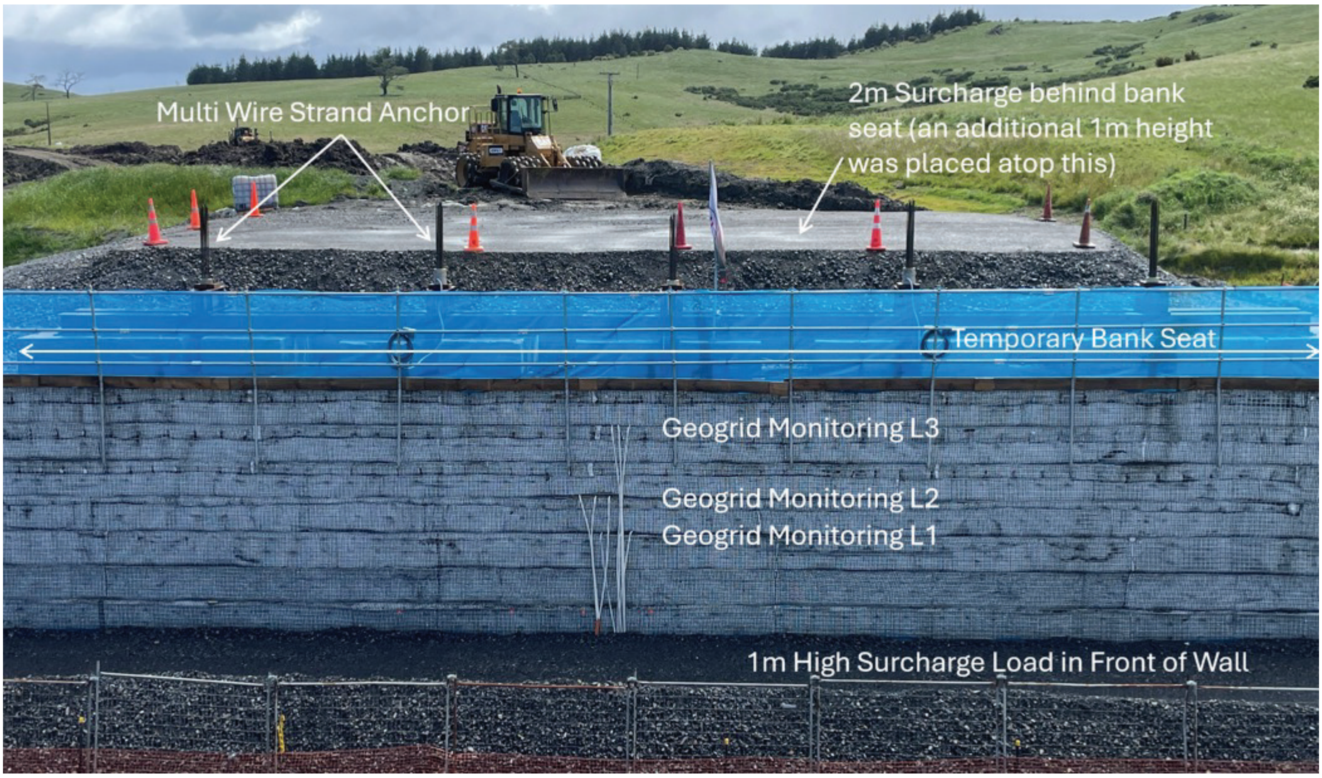


FIGURE 8 Photo showing prestressing arrangement and temporary bank seat

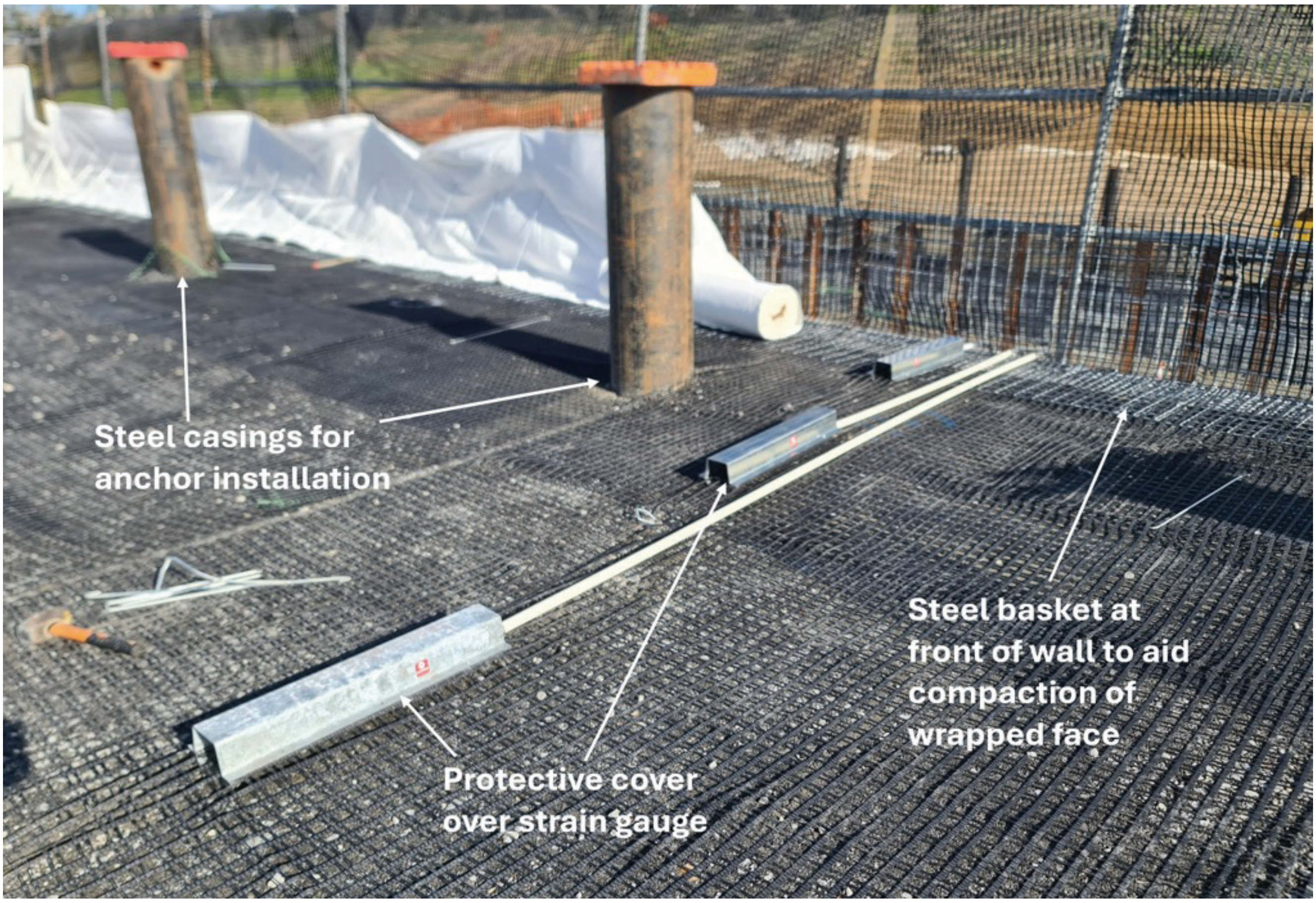
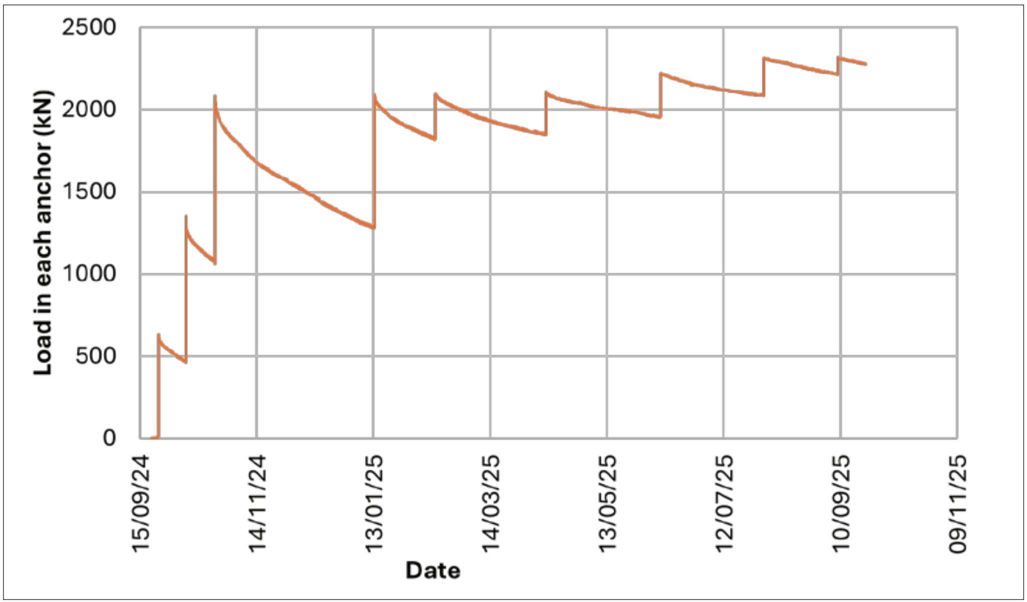
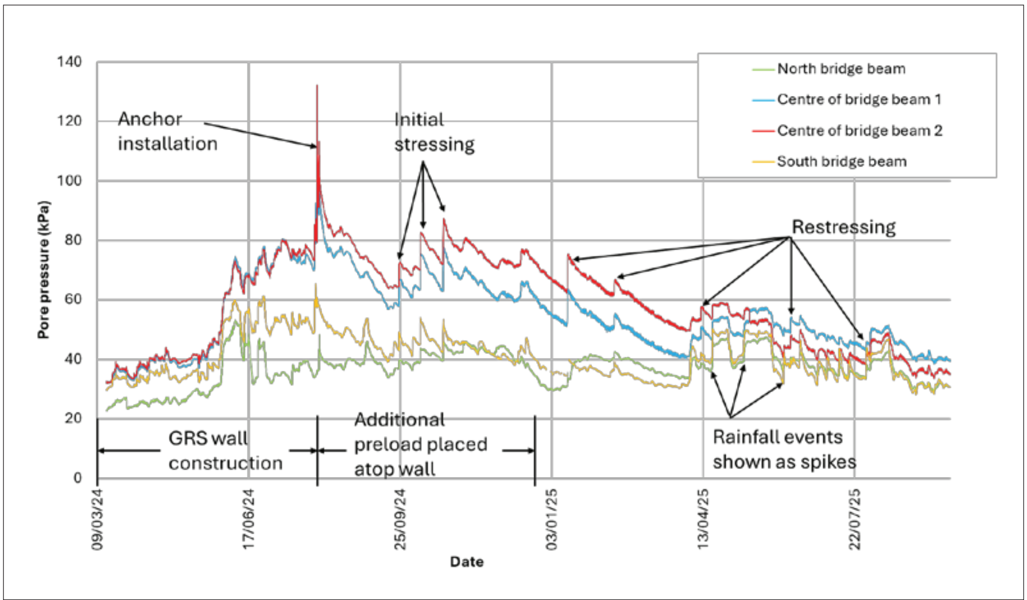


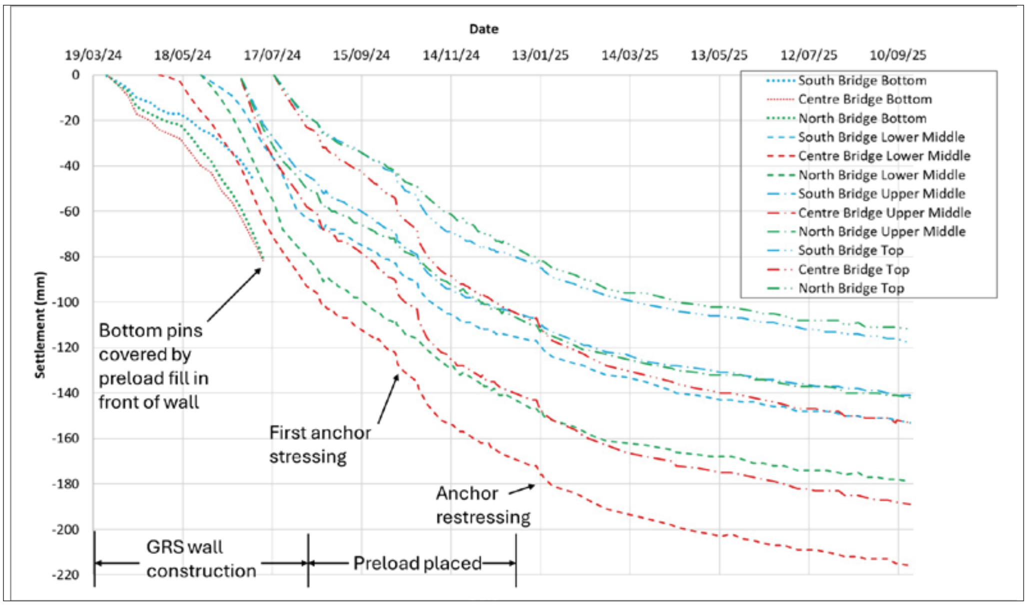
FIGURE 9 Photo of geogrid strain gauge installation



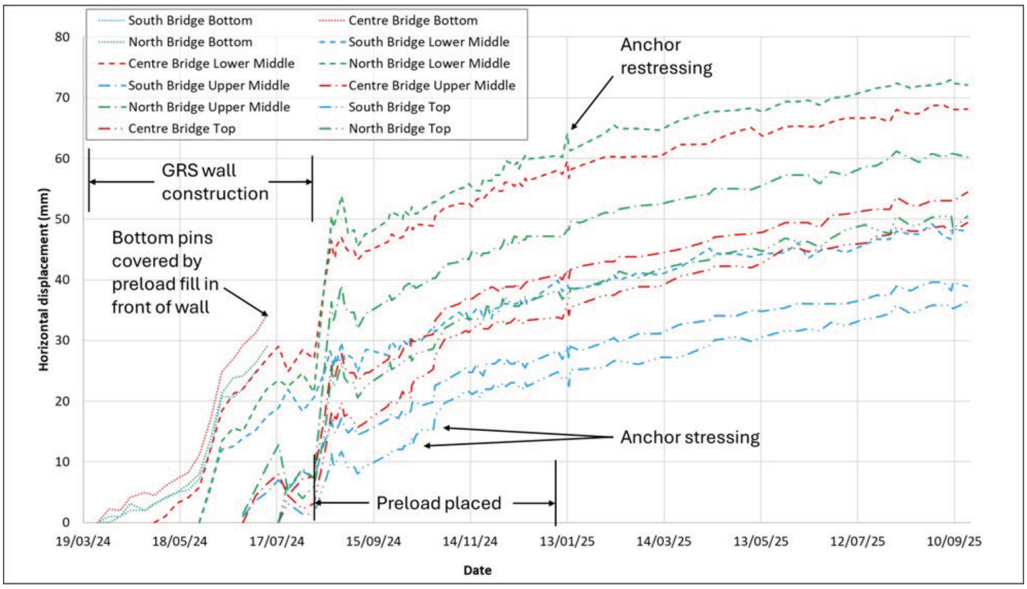
Plot 1 Anchor stressing sequence



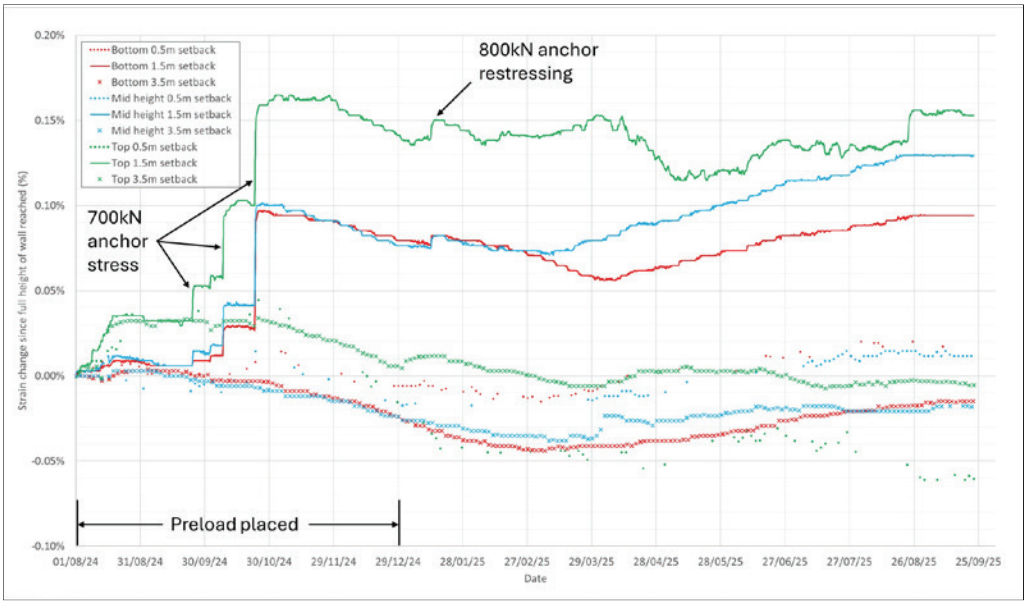
Plot 2 Pore pressure measurements beneath eastern abutment. Spikes in pore pressure from stressing and anchor installation noted. Other large sub vertical spikes in early 2025 due to heavy rainfall events.



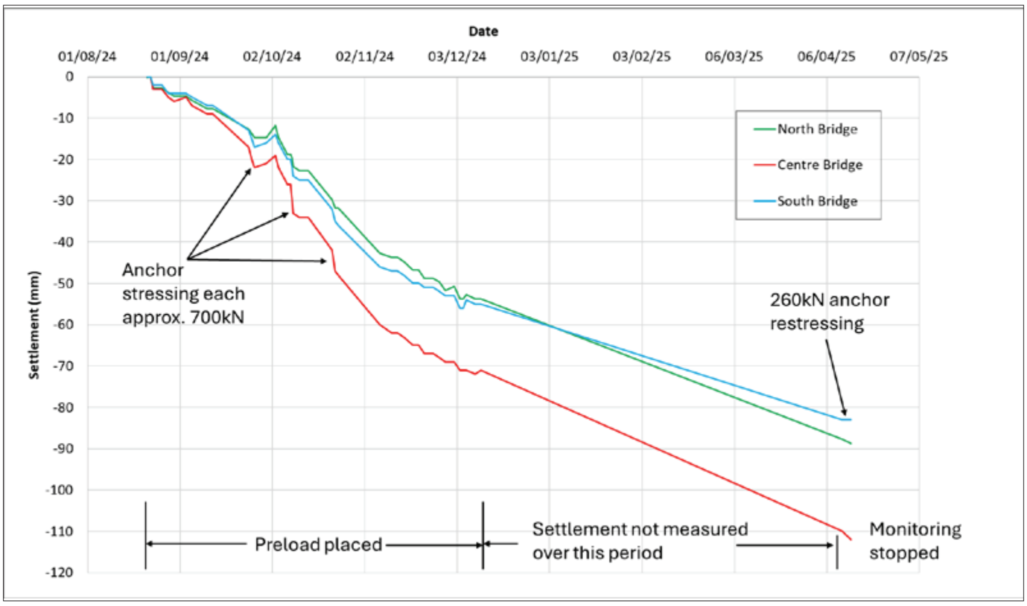
Plot 3 Vertical settlement plots for eastern abutment, showing construction and stressing stages. Note limited settlement from initial anchor stressing up to 650kN per anchor.



Plot 4 Horizontal displacement plots for eastern abutment, showing construction and stressing stages. Note large increase in horizontal movement early August 2024 which occurred after a rapid placement of around 1m height of preload fill atop the wall. Placement of filling was halted at this point until the porewater pressure substantially dissipated to around 60 kPa.



Plot 5 Measurements of change in geogrid strain after GRS wall constructed to full height. These demonstrate a) small strain increases in response to anchor stressing, particularly at distances of 0.5m and 3.5m setback from the face, b) very small strain increases in responses to anchor restressing, and c) very limited creep under sustained loading over a period of over 12 months and some indication of stress relaxation in select locations.



Plot 6 Reaction frame displacements (note that settlement occurs in the underlying alluvium when compared with total settlement plot above (Plot 3), indicating minimal settlement of the reinforced soil abutment)

indicated that the geogrid reinforced abutment performed as designed, (Weng, 2024).

7 CONCLUSIONS

The design and construction observations indicated that the bridge abutments can be constructed with commonly available construction plant and equipment, no specialist personnel or equipment such as piling rigs are necessary. The omission of piled bridge supports reduces overall cost and reduces construction complexity. The construction time is very similar to conventional construction as approach embankments also require settlement hold times and/or ground improvements. Arguably piling would be an additional step, which is not needed with load bearing bridge abutments.

Load bearing bridge abutments demonstrate very high resistance to extreme loadings, such as impact, fire and seismic actions (Kupec, 2021). The intention is to use the developed design on other NZ projects, with design on other Roads of National Significance (RoNS) projects being progressed in 2025, two of those are in high seismic zones with design PGAs >0.8g. There will be a need to obtain a Departure for each of these projects to use geogrid reinforcements to create load bearing bridge abutments.

Monitoring settlement and geogrid deformations compared against a back calculation of reinforced soil block stiffness indicated that the system performs as anticipated with creep deformations being negligible. Distribution of highest strains also matches established design guidance with the highest stressed geogrids being directly below the bridge bank seat.

The authors believe that load bearing bridge abutments offer a more sustainable alternative to conventional designs and potentially have greater resilience to extreme events. Another RoNS project in NZ is using site won aggregate to construct the reinforced soil body, thus reducing material transport cost.

The project described in this paper used two different facing systems, namely concrete panels with cast in geogrids, and wrap around geogrid soil block with non-load bearing full heigh precast panels for aesthetics. The authors have already developed abutments with large precast concrete mass blocks, thus meeting train impact actions as per NZ KiwiRail standards and further standardizing the presented design. The ability to vary the facing systems allows the use of load bearing bridge abutments in a range of different cultural and landscape environments.

8 ACKNOWLEDGEMENTS

The authors wish to thank NZTA for their permission to publish this paper. We also acknowledge and appreciate the trust demonstrated by granting the Departure, which allowed for the use of geogrid reinforcements on significant transport infrastructure. We also wish to acknowledge Aurecon and Tonkin & Taylor staff working with us on this project, especially Rhys Graafhuis and Susan Tilsley for the development of the engineering geological models, as well as, our contracting partners Fulton Hogan and HEB Construction.

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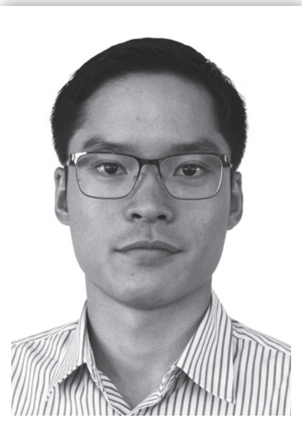
Beyond the Format: A Practitioner's Guide to AGS4 NZ Data

Tools, challenges and solutions

Kevin Chew, Stantec New Zealand



Kevin Chew
Stantec New Zealand

ABSTRACT

AGS4 data is a valuable resource for Geotechnical practice in New Zealand, but inconsistent formats and hidden errors present challenges for practitioners. This guide highlights some common issues, essential tools and practical solutions gathered from working with real-world data on the New Zealand Geotechnical Database.

1. INTRODUCTION

Geotechnical practice in New Zealand has made significant strides in the adoption and provision of AGS4 data formats for completed geotechnical investigations. This format is now widely accepted and routinely required as a standard deliverable, serving as the official electronic data transfer file across the industry, including consultants, contractors, and client organisations. With data intensive 3D geological modelling becoming increasingly common in the industry and the enhanced accessibility of historical data through the updated New Zealand Geotechnical Database website, there is a growing reliance on AGS4-formatted data as a trusted source for geotechnical analysis and interpretation.

Although AGS4 is a standardized format, its content can be inconsistent. Have you ever attempted to upload an AGS4 file into your company's geotechnical data management software or the New Zealand Geotechnical Database (NZGD), only to be met with a long list of validation errors? This issue is more prevalent than widely acknowledged and often remains unnoticed until it interrupts your workflows.

In this article, the author shares his insights gained from working with thousands of AGS files, both from the NZGD and large-scale investigation programmes, highlighting common issues, practical tools for viewing and manipulating AGS4 data, and strategies for identifying and resolving errors. It also explores typical compilation mistakes that can arise when relying on AGS4 data for analysis and modelling, offering suggestions to improve data quality and reusability.

2. WHAT IS THE AGS DATA FORMAT

Not to be confused with the Australian Geomechanics Society, also abbreviated as AGS, the AGS data format is a standardised framework developed by the Association of Geotechnical and Geoenvironmental Specialists (AGS) in the United Kingdom. First introduced in 1991 and now

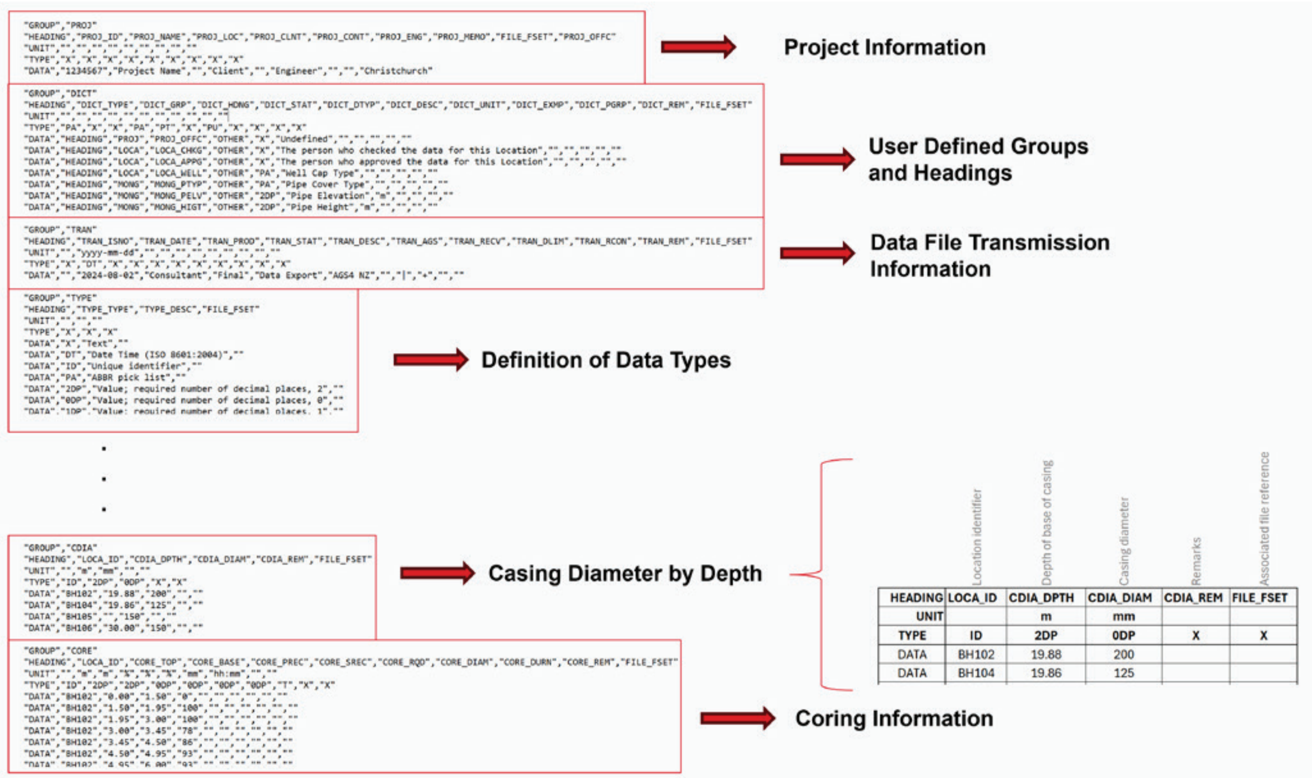


Figure 1: Typical content and layout of an AGS file.

in its fourth edition (commonly referred to as AGS4), the format enables the electronic transfer and structured storage of ground investigation and monitoring data. It organises geotechnical and geoenvironmental information, such as borehole logs, laboratory test results, and field measurements, into clearly defined groups and headings, allowing for seamless data exchange across different platforms and software systems.

AGS4 NZ closely follows the AGS (UK) format, with minor modifications and additions to suit New Zealand conditions. For local practitioners, the AGS4 NZ Electronic Transfer of Data guideline (https://www.nzgs.org/libraries/ags4-nz-electronic-transfer-of-data/), published by the New Zealand Geotechnical Society (NZGS) provides the specification and rules for the electronic transfer of geotechnical and geoenvironmental data using the AGS data format in New Zealand.

Stripping away the technical jargon, an AGS file is simply a text-based file format, meaning it uses plain characters and numbers. Its contents are comma-separated and contains a series of tables as shown in Figure 1, each representing a distinct data group, such as location details, sample information, or standard penetration test (SPT) results etc. These groups follow a predefined schema, including standardised naming conventions for the headers, as outlined in the AGS guidelines. This consistency in formatting is what enables the data to be read across different software platforms.

3. THE CHALLENGE OF DATA IMPORT

Every project starts with a review of existing geotechnical information to gain an understanding of the site conditions and potential geohazards. In practice, this information is typically derived from the following sources:

- New Zealand Geotechnical Database (NZGD)
- Internal / company-owned geotechnical database
- Information provided by the client
- Well logs from council-owned databases
- New geotechnical investigations

Traditionally, historical geotechnical data are provided in Portable Document Format (PDF). In recent years, especially on larger projects, AGS files compiling entire investigations from earlier phases have become increasingly common. For a typical geotechnical analysis, borehole logs and cone penetration test (CPT) results represent the primary sources of information and will therefore be the main examples discussed in this article.

3.1. BOREHOLE

Borehole logs are mainly provided in PDF format. These logs may be included as part of geotechnical factual report, provided individually as downloaded from the NZGD, or even presented as scanned copies of historical, handwritten records. More recent NZGD datasets would also include the AGS data file(s) as well.

Since most borehole data are stored in PDF format, manual log transcription is the most common way to extract and incorporate historical data. However, this process remains highly manual and time-consuming, particularly for large datasets. Thus, it requires selectively extracting relevant information to maintain some level of cost and time efficiency. With recent technological advancements, AI-powered log digitisation software, such as those offered by Civils.ai or SAALG Geomechanics, now provides effective and viable alternatives to manual transcription, although these tools do incur additional costs.

3.2. CONE PENETRATION TEST

CPT results are typically presented in spreadsheet formats such as .csv, .txt, or Excel. Over the past decade, most operators have consistently supplied CPT data in AGS format and thus, it is common to find CPT files on the NZGD accompanied by both a PDF log and an AGS file.

Unlike boreholes, CPT data needs to be presented in a tabulated format for practical use. For situations where only a PDF log is available, which often occurs with older CPT data or when information is provided through a factual report, extracting CPT results from the graphs can be challenging. While plot digitisation software exists, they may not be suitable when handling large amounts of data unless advanced technologies such as computer vision are employed. In these circumstances, it is advisable to request the digital data directly from the original supplier or consultant where possible.

It is worth noting that while CPT results are provided in a spreadsheet format, the way the results are tabulated and labelled can vary between the different CPT operators in New Zealand. This inconsistency also impacts the efficiency of data extraction, particularly when handling large amounts of investigation data points from different CPT operators.

3.3. VALUE OF THE AGS DATA FILE

From these examples, it becomes evident that AGS's consistent data format and storage can offer significant benefits for the data import process. For boreholes, processing an AGS file (if available) can greatly reduce the manual effort required to extract geological information from the logs. Conversely, CPT data in spreadsheet formats can be inconsistent between CPT operators, thus some degree of manual intervention is still required to accurately identify and extract the CPT results. The availability and use of the accompanying AGS file can help eliminate many of the limitations mentioned above. However, AGS data files also come with their own set of challenges and issues that users should be aware of (refer to Section 5).

4. VIEWING AND VALIDATING AGS4 FILES

Many commercial software applications offer built-in features for importing AGS files. While these tools generally perform well under ideal conditions, in practice, minor discrepancies in software configurations or database schemas can often disrupt the import process. Diagnosing the causes of such issues can be complex, especially when error messages can be vague or misleading.

Table 1: Tools and software to view, validate and convert AGS data files.

Tool	Description	Pros / Cons		Tips
Notepad / Notepad ++	Basic text editors that can open AGS files as plain text.	☑ Useful for inspecting AGS files that fail to open in other software. ☒ Difficult to read due to lack of formatting (see Figure 1). ☒ No validation features.		AGS files are comma-separated; you can copy the content of each table/group into Excel for easier viewing.
KeyAGS	An Excel add-in for importing, viewing, creating, and exporting AGS files.	☑ Easy to use. Excel is a familiar environment. ☑ Widely used in industry. ☒ Officially retired and no longer supported (Bentley Systems Incorporated, 2025).		A newer AGS Data Toolkit for Excel by GeotechnicalData (2025) may serve as a replacement, though it has not been tested by the author.
British Geological Survey (BGS) website	A web-based tool hosted by the BGS for validating and converting AGS files to Excel.	☑ No installation required. ☑ Quick access and easy to use. ☒ Some validation rules may be specific to BGS standards and it does not check against the standard data dictionary for AGS4 NZ.		A desktop (offline) version of the AGS Validator is available for download, but its use may be restricted by your organisation's IT policies.
Python-AGS4	The official Python library maintained by the *AGS Data Format Working Group.*	☑ Highly versatile and up-to-date. ☑ Forms the foundation of many other AGS tools. ☒ Requires basic programming skills. ☒ Not user-friendly.		Use platforms such as Google Colab to learn and experiment. Their "notebooks" provide interactive environments where you can write and run Python code directly in your browser without installation.

The first essential step in troubleshooting these problems is to inspect the contents of the AGS file directly. To support this, Table 1 presents a selection of primarily free and open-source tools designed to help users view, validate, and edit AGS files, especially when commercial solutions fall short. The functionality of commercial software is outside the scope of this article.

The list of tools presented in Table 1 is not exhaustive. With many new geotechnical data platforms emerging globally and frequent updates to existing commercial software, this is a constantly evolving area. The Association of Geotechnical and Geoenvironmental Specialists (AGS) maintains a list of software compatible with the AGS Data Format, which can be found here: https://www.ags.org.uk/data-format/software/

5. COMMON AGS DATA ERRORS

In theory, an AGS file should contain a complete digital representation of all the information presented in a PDF geotechnical log. However, as the examples below will demonstrate, that may not be the case in practice.

Before diving into common issues, it's important to distinguish between rules violations and contextual errors. AGS has established a set of rules that an AGS file must follow. For example, Rule 19 requires that all group/table names be uppercase with letters and/or numbers and no more than four characters long. These rules must be followed when compiling an AGS file. The list of rules is described in detail in Section 9 of the AGS4 NZ guide.

On the other hand, a more problematic but less documented issue arises when data is stored incorrectly, even though the file may pass validation. These errors are not structural but contextual, meaning the data may be technically valid but misleading or incomplete when used for analysis. In many cases, the AGS file export is treated as an afterthought and the focus is on making the data presentable on the PDF logs, without sufficient consideration for whether the data is stored in the correct AGS fields.

Based on the author's experience extracting geotechnical investigation data from thousands of historic AGS files submitted to the New Zealand Geotechnical Database (NZGD), a number of recurring issues have been observed. These are compiled in Table 2.

In addition to these issues, it's worth noting that NZGD has its own upload rules. At the time of writing, NZGD provides an AGS Code Reference, which outlines a list of accepted abbreviations (ABBR) for certain fields, which must be adhered to for successful uploads. Users should refer to the NZGD website for more information.

6. CONCLUSION AND RECOMMENDATIONS

The New Zealand geotechnical industry has made significant progress in adopting the AGS4 data format. Once a niche concept, AGS4 is now widely accepted, and many client organisations expect it as a standard deliverable. The collective effort to normalise its use deserves commendation.

At its core, AGS data is a simple structure comprising a collection of tables with predefined headers, descriptions, suggested units, and data types (e.g., text or numeric), making the data accessible.

Table 2: Common errors when working with AGS4 files. Note: The list is not exhaustive and is intended to highlight common problems encountered during data extraction, rather than catalogue every possible issue.

Type of Error	Description
Rule violation	Use of extended ASCII characters (e.g., non-breaking spaces, Em dashes, micro symbol µ). For those interested in diving deeper, this online article provides an overview of the ASCII issues with AGS files (https://digitalgeotechnical.com/2024/06/ags-data-the-perils-of-the-extended-ascii-character-set/)
	Use of non-ASCII characters (e.g., macrons in Māori place names or terminology)
	Duplicate key fields (e.g., repeated CPT depths).
	Data reported with incorrect precision (e.g., coordinate values).
	Custom groups not defined in the *DICT* group.
	Required groups such as *DICT* and *ABBR* not provided.
Contextual Error	Custom or new groups used unnecessarily.
	Detailed geological desxcriptions (*DETL_DESC*) incorrectly stored in *GEOL_DESC*.
	Standard penetration tests (*ISPT*) and vane shear tests (*IVAN*) results stored in sampling information (*SAMP*).
	ISPT_NPEN reported as main penetration only, instead of total (seating + main).
	Data labelled in units of MPa but incorrectly reported in kPa (e.g., local unit side friction resistance and shoulder porewater pressure for static cone penetration tests – *SCPT*).
	Unit conversions applied to cone penetration test error codes (e.g., code of data loss, -7777, reported as -7.777).

However, despite its simplicity and potential, data quality issues persist, both in newly submitted files and in legacy data stored on the New Zealand Geotechnical Database (NZGD). If these issues are not addressed, there is a risk that AGS4 data may become too unreliable for analysis, undermining its role as a data exchange format.

Some recommendations to consider:

- When preparing and submitting AGS files, validation is essential.
- The official AGS4 NZ guidance should be referenced when configuring geotechnical data management software. Additional resources are also available through the AGS UK portal, although full access may require membership.
- There is a strong case to establish a NZ-specific abbreviation (ABBR) list. The NZGD has already taken steps by providing a list of accepted abbreviations for fields such as coordinate systems and investigation types while AGS UK maintains a similar list. Additional standardisation could be extended to other commonly used fields. However, for such standardisation to be effective, industry-wide buy-in and consensus are crucial. Without broad support, enforced rules may discourage data submission.

By continuing to improve data quality, promote standardisation, and support best practices, the New Zealand geotechnical community can ensure the AGS data format remains relevant and useful to practitioners and not risk it becoming just another deliverable to satisfy contractual obligations.

DISCLAIMER

This article reflects the author's personal experience based on the software tools and data available to them at the time of writing. Certain details in the example AGS file have been omitted or modified to protect confidentiality. No warranties are provided for the tools and software mentioned herein, and their inclusion does not imply endorsement. Users should verify licensing terms and current functionality before use. The author is not responsible for any outcomes resulting from the use of these tools.

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CHALLENGE ACCEPTED.

If you drove to work today, chances are we helped engineer the roads. The office building you're in might carry the hallmarks of our construction team. And when you grabbed your morning espresso, you can bet our drilling expertise helped supply the water.

Our pleasure.

For over 50 years, Griffiths has played a key role in some of New Zealand's most significant — and most technically demanding — geotechnical drilling, civil, and construction projects. You've brought us your challenges, and you've trusted our name to deliver. Again and again.

And for that — we say thank you.

International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) Report, Dec 2025

Rolando Orense, Graham Scholey, Meenakshi Patel

Rolando Orense

Rolando Orense is a Professor at the Department of Civil & Environmental Engineering, University of Auckland. He is currently the NZGS representative to the ISSMGE.

Graham Scholey

Graham Scholey is a Technical Director at WSP Australia. He is currently the ISSMGE Vice President for Australasia.

Meenakshi Patel

Meenakshi Patel is a Geotechnical Engineer from ENGEO. She is currently the NZGS YGP ISSMGE Representative.

THE ISSMGE (https://www.issmge.org/) is the pre-eminent professional body representing the interests and activities of Engineers, Academics, and Contractors all over the world who actively participate in geotechnical engineering. The ISSMGE is a global organisation that provides a focus for professional leadership to some 90 Member Societies and over 21,000 individual members. In addition, there are currently 46 Corporate Associates.

The current Vice-President of ISSMGE for Australasia is Graham Scholey. There are only two societies in our region, NZGS and the Australian Geomechanics Society (AGS), but we have the largest per capita membership of any of the regions. The two societies are highly active, offering our members opportunities to attend lectures by eminent local and overseas speakers, attend high-quality training courses, access well-regarded journals, and attend conferences.

INCORPORATION OF ISSMGE

As of 1 May 2025, the ISSMGE has been incorporated as a legal entity under the name International Society for Soil Mechanics and Geotechnical Engineering Ltd., registered in the United Kingdom. The Society remains internationally recognised by its long-established name, the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE).

This incorporation strengthens the Society's governance and provides a formal legal foundation for its global activities. For those who are interested, you can find the new Governing Documents of the incorporated ISSMGE Ltd, specifically the (1) Articles of Association; and (2) Regulations, through this link: https://www.issmge.org/the-society/governing-documents

These documents replace the former Statutes and Bylaws.

21ST ICSMGE 2026 IN VIENNA, AUSTRIA

The 21st International Conference on Soil Mechanics and Geotechnical Engineering (21 ICSMGE 2026), with the theme "Geotechnical Challenges in a Changing Environment", will be held in Vienna, Austria, on 14–19 June 2026. The call for abstracts has been closed, and NZGS members submitted seven (7) full papers, which are currently undergoing review. Vienna is where Dr Karl Terzaghi published the book "Erdbaumechanik auf bodenphysikalischer Grundlage" in 1925, and the 21st ICSMGE will be a celebration of the 100th anniversary of this milestone.

For further details about the conference, please visit: https://www.icsmge2026.org/en/.

Preceding the ICSMGE 2026, the 8th International Young Geotechnical Engineers Conference (8iYGEC) will be held in Graz, Austria, from 11-14 June 2026. Further details about the conference are available here: https://www.tugraz.at/institute/ibg/events/8iygec

HERITAGE TIME CAPSULE PROJECT

ISSMGE was formed nearly a century ago. The ISSMGE Heritage Time Capsule (HTC) project is setting the strategy framework for the second 100 years of geotechnical engineering, including the creation of a dedicated HTC website https://htc.issmge.org/, where a large number of contributions have been placed by members of the various ISSMGE cohorts, and the planned sealing of a physical time capsule in 2026, to be opened in one hundred years, in 2126.

As part of this, the HTC project

leaders are seeking Discovery Reports. Individual members or teams are invited to prepare a brief report to shine a spotlight on a particular Contribution or Contributions to the HTC for sharing with our members. The discoverers' report can be a brief note, video, audio, or other form that can be stored on the HTC website and shared online.

The HTC project is particularly active in the lead-up to the ICSMGE 2026 conference. The ISSMGE has made available a cash pool of £3,000 for the HTC Discoverer Report Competition in 2025. There are prizes on offer for individual members who upload a Discoverer Report that meets certain criteria, as assessed by a pool of HTC Subcommittee judges. Further details on the Discoverer Reports, as well as access to Discoverers' Reports already uploaded, can be found on https://htc.issmge.org/discovery.

TECHNICAL COMMITTEES

Many members of the New Zealand Geotechnical Society are involved with the ISSMGE Technical Committees (TC) (https://www.issmge.org/committees/technical-committees). While a call was made in October for those interested to join any of the TCs, it's not too late if you want to be involved. You can apply to be a nominated member, but only two candidates per member society are permitted. Otherwise, you can be a corresponding member (and hope that a position becomes vacant). If you are interested, please contact the NZGS Secretary and ensure you are a member of both NZGS and ISSMGE. Nominations can only be made by Member Societies, not by individuals.

For more information about the Technical Committees and how to get involved, please visit the ISSMGE website (https://www.issmge.org/committees/technical-committees).

We extend our thanks to our local representatives and contributing members of the Technical Committees.

ISSMGE ACTIVITIES

Visit the ISSMGE website (http://www.issmge.org) for full details of all ISSMGE activities, as well as the wealth of resources available to members.

Prepared by:

Graham Scholey
VP Australasia

Rolando Orense
NZGS ISSMGE Representative

Meenakshi Patel
NZGS YGP ISSMGE Representative

International Association for Engineering Geology and the Environment (IAEG), June 2025

Report for New Zealand, Ross Roberts

Ross Roberts
Ross is Chief Engineer at Auckland Council. He is a chartered geotechnical engineer and professional engineering geologist with over twenty years' experience. Ross a permanent member of the New Zealand Landslides National Advisory Group, a steering committee member of the AGS Landslide Risk Management guidelines project, a past Chair of the New Zealand Geotechnical Society, and the New Zealand representative on the IAEG Council.

Lauren Foote
Lauren is an Engineering Geologist at consultancy WSP. She is New Zealand's representative on the IAEG Young Professionals Group and is a Professional Engineering Geologist who has been involved with land damage assessments following the 2010-2011 Canterbury Earthquake Sequence and the 2016 Kaikoura Earthquake. She specialises in hazard assessment and mitigation, with a particular focus on landslides.

Ann Williams
Ann is a technical specialist in the fields of engineering geology and hydrogeology. As a manager of some 630 people, a Board Member of Engineering New Zealand, past Chair and Life member of the New Zealand Geotechnical Society Inc., and past Vice President of the IAEG, Ann has significant first-hand experience of the opportunities for women in the discipline and is somewhat dismayed at the number of firsts still to be had for women in Engineering Geology in 2024.

1 WHAT IS IAEG AND HOW DO WE FIT?

All NZGS members also join one (or more) of the three international societies that NZGS represents in New Zealand; the International Association for Engineering Geology and the Environment (IAEG), the International Society for Rock Mechanics and Rock Engineering (ISRM), and the International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE).

NZGS is represented on the IAEG Board (Executive Committee) by Anthony Bowden, IAEG Vice President for Australasia (one of six regions). Each country that is part of a regional group has an independent vote in Council meetings, and I carry this vote on behalf of the NZGS.

2 IAEG WORLD CONGRESS – DELFT 2026

The next big event in the IAEG Calendar is the World Congress. This is the highlight of the circuit for engineering geologists, so anyone who can make it to Europe in late October 2026 should seriously consider making the journey. More information can be found on the conference website (https://www.iaeg2026.org/150970/home).

3 IAEG MANAGEMENT UPDATE

The most recent executive board meeting was held in September 2025 (where we were represented by Anthony Bowden and Ann Williams). Key items from this meeting include:

- Membership fees: Individual and Associate Membership fees have not changed since 2016 and will likely need to be increased at the time of the next Council meeting in 2026.
- IAEG are expanding their professional development training to members through their National Groups. To support this initiative, international sponsorship is being obtained to cover the significant costs of implementing events around the world and particularly in lower-income countries. The support and assistance of National Groups, particularly in high-income countries would be appreciated.
- A new Commission on Dams and Levees has been formed and is open to new contributors.
- Members are encouraged to complete their online digital profiles on the IAEG website to get best value from it. Each IAEG affiliated member should have received a token to allow initial access. Contact us if you haven't received this.
- Queenstown will host an Executive Committee meeting in April 2026 as part of the Landslide Geo-education and Risk conference.

4 YOUNG ENGINEERING GEOLOGISTS

The Young Engineering Geologists Group of IAEG remains very active, and all members are encouraged to participate. Lauren Foote is the IAEG YGP representative within NZGS. Young Engineering Geologists (anyone under 40) should contact Lauren, check out the IAEG YEG website (https://iaeg.info/yegs/) and for the most current activity, follow their great webinars on YouTube (https://www.youtube.com/@iaegyeg), articles in the IAEG Connector and posts on LinkedIn (https://www.linkedin.com/company/international-association-of-engineering-geology-and-the-environment).

5 WOMEN IN ENGINEERING GEOLOGY

The IAEG is committed to increasing the involvement and inclusion of women in the activities and opportunities of the Association. This is part of a wider drive to build diversity in the organisation, and to give equal opportunity to all members. The Women in Engineering Geology Group (WEG), representing the interests of Women in the field of Engineering Geology, is open to participation by any member of the Association, not just women. It is administered by a Women in Engineering Geology Committee (WEGC), led by our own Ann Williams. Find out more about the group on the IAEG website (https://iaeg.info/weg/info/).

6 REGISTER NOW FOR THE WEBSITE & JOURNAL

All NZGS members who have affiliated to IAEG are eligible to access resources on the IAEG website, including free access to the highly regarded Bulletin of Engineering Geology and the Environment, the official journal of the IAEG. It's ranked as one of the top global journals in our discipline, so is well worth keeping up to date with.

All affiliated members should have received an email (in July) with the subject line "Welcome to IAEG members Area" from membership@iaeg.info giving you a username and password. You will need to follow these instructions to access the membership benefits of IAEG including the journal. If you're struggling, please contact me or email membership@iaeg.info.

International Society for Rock Mechanics and Rock Engineering (ISRM)

Report for New Zealand – December 2025

Eleni Gkeli

Eleni is a Technical Director for Engineering Geology with 27 years of experience in the geotechnical profession, specialised in rock slope engineering and tunnels. Eleni's experience was gained in large infrastructure projects in Greece and in New Zealand. Eleni has been working in New Zealand since 2012, initially with WSP (former Opus) and more recently with Stantec. She has been involved in a range of projects in the transportation, water and land development sectors many of these involving design of infrastructure in rock formations. Eleni has been involved in the NZGS since 2016 in a variety of roles. She was the NZGS Chair for the term 2021 to 2023 and has just recently been appointed as the New Zealand liaison for ISRM.

Mohamud Hassan

Mohamud Hassan graduated from Canterbury with a professional master's degree in engineering Geology. He currently works as an engineering geologist at Bathurst Resources Limited, working at Stockton Mine, one of New Zealand's largest open-cast mines. His responsibilities include geotechnical risk management, highwall slope stability analysis, and hydrogeological modelling, utilising advanced software like Vulcan and Rocscience. He oversees multiple active pits, dealing with extreme weather conditions and the challenges of rock mass variability. He is passionate about fostering connections among young professionals, raising awareness about ISRM's initiatives, and nurturing future leaders in rock mechanics. Outside of his professional life, Mohamud enjoys hiking, tramping, reading literature, playing rugby/soccer, and socializing.

ISRM BOARD AND COUNCIL MEETINGS, 16 AND 17 JUNE 2025 – PRESIDENTIAL ELECTION

The 2025 ISRM Board and Council meeting took place at the Eurock 2025 Conference in Trondheim, Norway. Eurock 2025 served as ISRM's international symposium for that year. The election for the ISRM President for the 2027-2031 term was held on 16 June 2025 during the Council meeting. Eleni Gkeli represented the New Zealand Geotechnical Society at the Council meeting.

The candidates for the ISRM Presidency were Pinnaduwa Kulatilake, nominated by Sri Lanka, and Sergio Fontoura, nominated by Brazil, Argentina, Mexico, and Paraguay. Both candidates had significant credentials and qualifications, and the competition was strong. The New Zealand Geotechnical Society, following discussion in the management committee, choose to support the candidacy of Sérgio Fontoura as the ISRM next President.

The successful candidate in the election to be the next President of ISRM was Sérgio Fontoura. The elected candidate will join the Board as President-elect immediately following the meeting and officially assumed the role of President after the ISRM International Congress in 2027, which was held in Seoul.

17TH ISRM INTERNATIONAL CONGRESS – THE BID OF THE NEW ZEALAND GEOTECHNICAL SOCIETY

The NZGS has submitted a proposal to host the 17th International Congress of the ISRM in Christchurch, scheduled for September 2031. We are grateful for the strong support from Tourism New Zealand and Christchurch NZ in this endeavour and extend our sincere appreciation.

Eleni Gkeli, Romy Ridl, and Christoph Kraus—members of the committee bidding for the 2031 ISRM Congress—actively supported the NZGS proposal by delivering a well-received oral presentation at the 2025 Council meeting in Trondheim (see Figure 1). On this occasion, our only competitor was Mumbai, India, whose representatives also presented their bid to the Council.

The Council members responded positively to our presentation, providing encouraging feedback and expressing interest in supporting the Christchurch bid. In addition to highlighting the achievements and ongoing initiatives of NZGS, we showcased the NZGS Slope Stability guidance series developed and published by NZGS. The NZGS guidance generated considerable interest among Council members, leading to further discussions and requests for details over the subsequent days of the conference.

NZGS had also an exhibition booth which became a vibrant hub for networking, engagement and collaboration with the Council members and members of the national groups over the days of the conference. Throughout the event, the booth attracted the attention of Council members and representatives of national groups, serving as a focal point for showcasing NZGS's initiatives and achievements. The display featured the MBIE/NZGS Geotechnical Earthquake Engineering Series, the Slope Stability guidance series, sparking discussions and follow-up queries from delegates. This interactive presence not only elevated the profile of NZGS within the international rock mechanics community but also fostered valuable connections and partnerships that are expected to benefit future projects and the ongoing bid to host the 2031 ISRM Congress in Christchurch.

The final decision was scheduled to be made at the ISRM Council meeting in 2026 in Japan.

Figure 1: Eleni, Romy and Christoph present on the NZGS Bid for the ISRM 2031 at the Council meeting in Trondheim.

HOSTING INTERNATIONAL ROCK MECHANICS EXPERTS IN NEW ZEALAND IN 2026

Our plans over the coming year include hosting international rock mechanics experts in New Zealand, to enhance the engagement of the NZGS membership with the ISRM community but also to showcase our current local practice in Rock Mechanics and Rock Engineering.

We are arranging a visit for ISRM Vice President for Europe, Prof. Muriel Gasc-Barbier (France), with the kind and generous support from ChristchurchNZ. Muriel will speak at the International Conference on Geomorphology in Christchurch from 2 to 6 February 2026, followed by an NZGS-organised talk tour across New Zealand centres.

We are also arranging in collaboration with the Australasian Vice President Qianbing Zhang from Australia and the Australian Geomechanics Society for the current ISRM President Professor Seokwon Jeon from the Republic of Korea and Professor Leandro Rafael Alejano Monge from Spain to collaborate for the development of lectures and workshops in New Zealand and Australia. Please watch this space, detailed announcements will be made over the coming months.

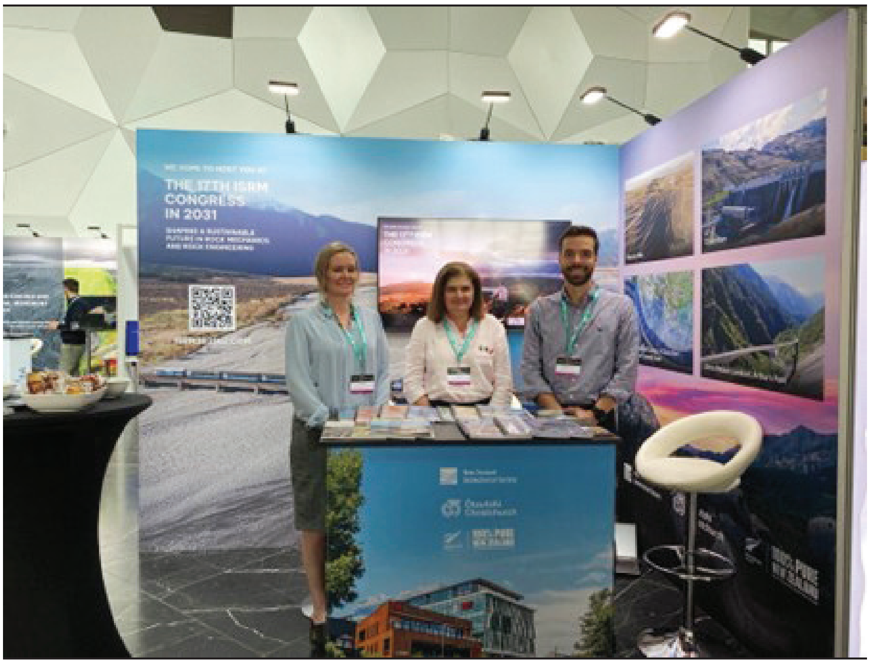

Figure 2: The NZGS exhibition booth in Eurock 2025 in Trondheim.

FIRST AUSTRALIAN CONFERENCE ON ROCK MECHANICS (ACRM 2026), MELBOURNE 21-24 JULY 2026

The AGS and ISRM invite researchers and practitioners to contribute to the First Australian Conference on Rock Mechanics (ACRM), to be held in Melbourne on 21-24 July 2026. This event will bring together leading experts, researchers, and industry practitioners to exchange knowledge, foster collaboration, and showcase the latest advancements in rock mechanics and rock engineering. ACRM is set to become a key national platform for professional networking, engagement with early-career professionals, and strengthening Australia's contribution to the global rock mechanics community. The deadline for abstract submission is Monday 10 November 2025.

51ST ISRM ONLINE LECTURE

The 51st ISRM Online Lecture was given by Professor Michel Van Sint Jan from Chile. The topic of the lecture was "Rockbursts: Mechanisms, Hazards, and Engineering Implications". It was broadcast on September 11 at 10 A.M. GMT and will remain available on the Online Lecture's page. Michel Van Sint Jan began his geotechnical career in 1972 at the Pontificia Universidad Católica de Chile (PUC), where he served on the civil engineering faculty until 2010 and was appointed Full Professor in 1989. He earned his MSc (1975) and PhD (1982) in Civil Engineering from the University of Illinois. From 2011–2022 he was Managing Partner of MVA Geoconsulta, leading geotechnical consulting for major civil and mining projects in Chile and abroad. His teaching and research span rock and soil mechanics, with contributions to the behavior of tunnels and caverns in rock, rock-mass strength and fracture, seismic stability of rock slopes, and the performance of tunnel support under dynamic loading (rockbursts). He has authored more than 60 papers and book chapters and has delivered invited lectures and professional courses.

PROFESSOR EVERT HOEK'S LEGACY

Professor Charles Fairhurst delivered the keynote lecture "Evert Hoek, his Legacy and Rock Mechanics/ Engineering in the 21st Century" at the 59th US Rock Mechanics / Geomechanics Symposium (ARMA Rocks 2025) held in Santa Fe, New Mexico, in June 2025. The lecture was recorded and is available on the ISRM website.

In addition, a lecture by Evert Hoek was reconstituted by the University of Leeds and is now available on the ISRM website. This lecture was compiled from an audio tape of Professor Evert Hoek's lecture on weak rock masses in 1990 at the University of Leeds, combined with his slides. It is considered important as it sets out Professor Hoek's philosophy about what makes a rock mass weak, and, in answer to a question from Dr John Sharp he presented an early version of the Hoek-Brown strength criterion, which over time has morphed into the Geological Strength Index (GSI). As he stated, there wasn't then (and still isn't) any other tool that allows the strength of fractured, isotropic rock masses to be estimated.

Prepared by
Eleni Gkeli
ISRM NZ liaison

Young Geoprofessionals Reports

Christoph Kraus – Young Geoprofessionals Representative

Christoph Kraus
Christoph is a Professional Engineering Geologist (PEngGeol) at Beca, and the current NZGS Young Geo-Professionals Coordinator. Christoph's key interests and expertise include analysing complex geology and developing geological models, landslide risk assessments, as well as the assessment and mitigation of natural hazards. He is experienced in geological mapping and ground investigations, having conducted fieldwork in a range of different geological settings throughout New Zealand, in Samoa, Patagonia and Antarctica.

Outside of work, Christoph's interests include travel, exploring the outdoors, football, photography, and spending time with his young family.
ygp@nzgs.org

IT'S BEEN ANOTHER busy 6 months in the YGP space since the last edition of Geomechanics News. I've provided an update on some of the activities over the past few months, and plans going forward, below. As always, if you are keen to be involved or have ideas for future events and opportunities, please feel get in touch.

YGP BREAKFAST SESSION AT THE NZGS2025 SYMPOSIUM

During the recent NZGS2025 Symposium in Auckland, we hosted the YGP breakfast session where the winners from last year's mini symposia (Jerry Lei, Hamish Foy, João Pedro de Souza Oliveira, Rebecca Till, and Dion Dow) were able to present their winning presentations to the NZGS2025 Symposium delegates. All five YGPs presented excellent presentations on a variety of topics including how geotechnical engineers can drive innovation together, engineering geological models, rockfall fragmentation testing, landslide remediation, and collaborative approaches to reconnecting communities.

The event had a fantastic attendance, and it was great to see so many delegates making their way to the conference venue early to support the YGPs. I also want to thank Geo Data Solutions for their support sponsoring the session.

REGIONAL YGP MINI SYMPOSIA

This year we again hosted our annual regional YGP mini symposia. It's been seven years since we hosted the first NZGS YGP mini symposium, and it's great to see the continued success and growth of these events. This year we hosted the first mini symposium in Nelson, meaning that we had a total of five regional mini symposia across New Zealand! I really enjoyed being able to attend two of the mini symposia (Wellington and Nelson) and connect with the local YGPs and mentors. The presentations from the YGPs were excellent, and it was fantastic to hear the mentor's share their stories, insights and advice.

I want to acknowledge the local organisers of the symposia who have put in significant effort and have done a great job organising this year's events: Jerry Lei, Connor Oey, Hamish Foy (Auckland and Northland), João Pedro de Souza Oliveira, Ben McKay (Waikato and Bay of Plenty), Rebecca Till, Paul Tan (Wellington), Lauren Foote (Nelson), Dion Dow, Jayden Neven and Imogen Daysh (Christchurch).

I also want to thank all the mentors and NZGS management committee representatives who attended the mini symposia for their time, dedication and support for the next generation of geo-professionals in New Zealand: Heather Lyons, James Johnson, Rolando Orense (Auckland and Northland), Kim de Graaf, Matt Packard, Jesse Beetham (Waikato and Bay of Plenty), Eleni Gkeli, Kate Williams (Wellington), Sigfrid Dupre, Sally Hargraves (Nelson), Naomi Norris, Adrian Short, Ioannis Antonopoulos (Christchurch).

We are also extremely grateful to our sponsors Geotechnics, Redi-Rock, Geobrugg and WSP. Without your support these events wouldn't be possible.

Finally, I want to congratulate all the winners of this year's symposia: Shane Forrest (Auckland and Northland), Bala Elankumaran (Waikato and Bay of Plenty), Dani Castello (Wellington), Lucie Klimkova (Nelson), and Izzy Raziff (Christchurch). The winners of this year's mini symposia, as judged by the mentors, have all received a prize toward their ongoing professional development.

Figure 1. YGP Breakfast Session presenters at the NZGS2025 Symposium

For more information about each symposium, please have a look at the reports in this edition of Geomechanics News.

15TH YOUNG GEOTECHNICAL PROFESSIONALS CONFERENCE (15YGPC) IN ADELAIDE, SOUTH AUSTRALIA

The proceedings of the 15th Young Geotechnical Professionals Conference (15YGPC), held in Adelaide in November 2024, have now been published on the AGS website. To view the proceedings, head to https://geomechanics.org.au/papers/proceedings-of-the-15th-young-geotechnical-professionals-conference/.

16TH YOUNG GEOTECHNICAL PROFESSIONALS CONFERENCE (16YGPC) IN CAIRNS, QUEENSLAND, AUSTRALIA

We are excited to announce that the next joint Australia and New Zealand Young Geotechnical Professionals Conference will be held from 1-4 September 2026 in Cairns, Australia.

Abstracts are due 27 February 2026, so now is a great time to start preparing your abstract! For more information head to: https://geomechanics.org.au/16ygpc

INTERNATIONAL YGP PRESENTATION

At the time of writing this report we are currently preparing to host our first international YGP presentation on 9 December. The presentation will be hosted online and may also be shown at some branch locations with opportunities for local networking before and after the presentation.

Bernhard Klampfer from ILF Consulting Engineers in Austria will present on 'Risk-Based Decision-Making in Road Tunnel Operations'. The presentation is a continuation of our ongoing work with the joint young member's Austria (J-YMA) group of the Austrian Society for Geomechanics.

COLLABORATION WITH INTERNATIONAL SOCIETIES

In addition to the international YGP presentation noted above, I have also met with the local organisers of the Canadian YGP conference to discuss similarities and differences between the ANZ and Canadian YGP conferences, and what we could learn from each other. I've also set up regular meetings with the Canadian Geotechnical Society YGP representative to discuss the initiatives that each of our societies run, share learnings, and discuss potential future collaborations.

The NZGS international society coordinators Lauren Foote (IAEG), Meenakshi Patel (ISSMGE), Mohamud Hassan (ISRM), and I continue to meet monthly to share ideas and discuss what is going on in the international societies.

LaRGE2026 IN QUEENSTOWN

As part of the first international joint workshop of JTC 1 and JTC 3 on Landslide, Risk and Geo-Education (LaRGE2026), which will be hosted in Queenstown next year, we are organising a dedicated early career researcher and YGP event (which can be attended by all conference attendees). The event will be a panel discussion on career paths, showcasing a range of diverse perspectives and highlighting the different potential career paths in our industry. The panel discussion will be followed by networking at the venue. For more info please visit: https://landsliderisk.nz/

2025 Auckland & Northland YGP Symposium

WHERE: Beca Auckland Office, 124 Halsey Street, Auckland Central
WHEN: Friday 21 November 2025
SPONSORS: Redirock and Geotechnics

Hamish Foy (ENGEO)

THE SEVENTH AUCKLAND and Northland Young Geotechnical Professional (YGP) Symposium was held on 21st November 2025 at Beca's Auckland office, with seventeen young professional delegates (twenty three geotechnical professionals in total) from various sectors, including students, consultants, and regulators. Presentations covered topics in engineering geology, geotechnical engineering, risk regulations, construction, and automation, providing a rich exchange of knowledge and networking opportunities.

Congratulations to all the presenters for their outstanding contributions – The calibre of presentations this year was top class and the top prize was hotly contested. The Mentors' Choice Award was presented to Shane Forrest (ENGEO), while Matt Cook (Tonkin & Taylor) received the People's Choice Award. Notable mentions were given to Alice Boyd (Riley Consultants), Tony Liu (Tonkin & Taylor). Special thanks to our sponsors, Geotechnics and Redi-Rock, for their ongoing support. Their representatives, Jinjutar Saisakares (Geotechnics), along with David Hepburn (Redi-Rock), attended the event. We would also like to thank Jamie Young (ENGEO) for creating such an incredible trophy for this year's event.

A big thanks also to Beca for hosting the symposium. We appreciate our mentors, Heather Lyons (ENGEO) and James Johnson (Beca), for their insightful contributions during the event. A big thank you to Professor Rolando Orense for supporting the symposium as an NZGS representative.

ABOVE All 17 delegates, facilitators, mentors and NZGS representatives for the Auckland & Northland Young Geotechnical Professionals Symposium. **LEFT** Mentors choice award kindly crafted by Jamie Young, ENGEO. **BELOW** Mentors choice award winner – Shane Forrest, ENGEO (Centre), people's choice award winner – Matt Cook, T&T (Third from left). Our mentors – Heather Lyons, ENGEO (second from the left) and James Johnson (third from the right). Our honorable mentions Tony Liu, T&T (Left), Katrina Browne, ENGEO (second from the right), and Alice Boyd, Riley Consultants (right).

2025 YGP Bay of Plenty / Waikato Regional Mini-symposia

WHERE: University of Waikato, Tauranga Campus
WHEN: 3 November 2025

João Pedro (JP) de Souza Oliveira, ENGEO

THE WAIKATO/BAY OF Plenty Young Geotechnical Professionals (YGP) Symposium was recently held at the University of Waikato's Tauranga Campus, creating an exciting platform for knowledge sharing and collaboration across YGP's of both regions. The event was a success, with a dynamic pace and featured a diverse range of topics, from regional-scale geological studies to practical case studies. A standout for the event was the first year of participation of researchers from the University of Waikato, who contributed with presentations on each of their lines of research.

Attendees had the opportunity to network, exchange ideas, and strengthen connections among local YGP's, reinforcing collaboration within the geotechnical community. This year's mini-symposium highlighted the value of bridging academic research and industry practice, fostering innovation and professional growth.

Bala Elankumaran from the University of Waikato took home the Mentor's Pick for best presentation, while Jordan Mackinnon from WSP won the People's Choice award.

A special thank you goes to our sponsors, Redi-Rock and Geotechnics, whose support made this event possible. We also extend our gratitude to our mentors, Matt Packard and Kim de Graaf, for their presentations, feedback and guidance, demonstrating the commitment to developing the next generation of geotechnical professionals. Lastly, a big thank you to all the YGP's who stepped up to collaborate and share learnings from their careers - the event would not be possible with the contribution of each of you!

ABOVE BoP/Waikato event participants.
LEFT Peoples Choice and Mentors Choice Award winners: Jordan Mackinnon & Bala Elankumaran.
BELOW Celebration dinner following the event was well attended!

2025 Wellington Young Geotechnical Professional (YGP) Mini-Symposium

WHERE: Engineering New Zealand Office, Level 6, 40 Taranaki Street, Wellington
WHEN: 11 November 2025

Paul Tan, WSP & Rebecca Till, Beca

THE 2025 WELLINGTON Young Geotechnical Professionals (YGP) Mini-Symposium was held on 11 November at the Engineering New Zealand office, bringing together a group of 14 emerging geo professionals.

Each attendee demonstrated strong technical expertise and professionalism through their presentations, covering topics such as desktop study preparation, ground modelling, design and calculations, and on-site construction monitoring and testing.

Congratulations to all participants for doing a fantastic job sharing their insights and experiences! Special recognition goes to Dani Castillo (WSP), recipient of the Mentor's Choice Award, and Theo Calkin (WSP), winner of the People's Choice Award.

We extend a massive thank you to our generous sponsors Geotechnics and RediRock for their continued support in making this event possible. We also thank NZGS for providing this valuable opportunity for young professionals, and Engineering New Zealand for kindly providing the venue and assisting with the preparation.

We also express our sincere appreciation to our mentors Kate Williams (Tonkin & Taylor) and Eleni Gkeli (Stantec), and to Christoph Kraus (Beca), representing NZGS, for their guidance, feedback, and for sharing their own stories which we will treasure throughout our careers.

Lastly, on behalf of the organising team, thank you to all the participants for making the mini-symposium a success. Your enthusiasm, insights and willingness to share knowledge turned this event into an enjoyable and memorable experience. We could not have achieved this without your contributions, and we look forward to seeing you at future events."

2025 Wellington YGP Mini-Symposium – Mentors and Attendees.

2025 Wellington YGP Mini-Symposium – Mentor's Choice Award Winner, Dani Castillo (WSP).

2025 Wellington YGP Mini-Symposium – People's Choice Award Winner, Theo Calkin (WSP).

2025 Wellington YGP Mini-Symposium – Dinner.

2025 Nelson YGP Mini-Symposium

WHEN: 6 November 2025
SPONSORS: WSP and Geotechnics

Lauren Foote, WSP

THE TOP OF the south was well represented with participants from Nelson, Tasman and Marlborough attending the first Nelson YGP Mini-Symposium. While we were a small group of five presenters, the day was a great success with interesting presentations covering the depth and breadth of projects that are underway across our region. All the presenters should be proud of their efforts in sharing projects in such an engaging and informative way. I know we all left having learnt something new.

Our people's choice award went to Simon Alder for his presentation on the Northbank Road Emergency Rock Cut (we love a project with explosives), while mentors choice went to Lucie Klimkova for her excellent presentation "Stopbanks – Curveballs in linear disguise". Congratulations to our award winners!

We have many people to thank for the success of this event – Christoph Kraus for travelling from Wellington to bring a NZGS presence and share some cool updates around things that are underway in the broader YGP space; our wonderful mentors Sally Hargraves and Sigfrid Dupre for giving up their time to connect with our young professionals; and to our sponsors WSP and Geotechnics for the financial support to bring this event to life.

I'm already looking forward to the second Nelson event in 2026.

FIGURE 1. Attendees at the first Nelson YGP mini-symposium
FIGURE 2. Enjoying some post-event networking in the Nelson sunshine.
FIGURE 3. Our award winners, Simond Alder and Lucie Klimkova

Christchurch NZGS YGP Mini Symposia

WHERE: Aurecon Office
WHEN: 12 November 2025
SPONSORS: Geobrugg and Geotechnics

Dion Dow (ENGEO), Jayden Neven (Aurecon), Imogen Daysh (ENGEO)

THE CHRISTCHURCH YOUNG Geotechnical Professionals (YGP) Symposium was held on the 12 November 2025 at the Christchurch Aurecon office. The event saw 14 brilliant young professional delegates from across the industry, including geotechnical engineering, engineering geology, and even some attendees from the University of Canterbury. All the presentations were of high quality and gave an insight into all the exciting projects that young professionals are making their mark on. The day was filled with laughs, networking and most importantly, lots of learning!

Congratulations to all the presenters for doing a wonderful job! The Judge's Award went to Izzy Raziff (KiwiRail) for his presentation on "The Digital Age of Managing Geotechnical Risk and Asset Management on New Zealand Railways". The People's Choice Award went to Caroline Birse (Pattle Delamore Partners) for her presentation on "Avoiding a Slippery Slope: Factors influencing the Distribution and Effects of Earthquake Induced Landslides in New Zealand".

We would like to thank our sponsors of the event, Geotechnics and Geobrugg, from which Stu Mason attended and provided insights on Geobrugg's products and capabilities. We would also like to thank our two mentors Naomi Norris (ENGEO) and Adrian Short (Aurecon) for providing valuable learnings and career advice to the presenters. We also extend our thanks to Ioannis Antonopoulos (NZGS Vice Chair/Stantec) for attending the event.

PRESENTATIONS

Tim Stotter – *Landfill Slope Stability*

Anna Duston – *Liner Strain Assessment for Te Waihekeora Water Storage Reservoir*

Sophie Braddick & Jasmine Niederberger – *Internal erosion of volcanic soils*

Izzy Raziff – *The Digital Age of Managing Geotechnical Risk and Asset Management on New Zealand Railways*

Hugh Charles – *O'Sullivan's Bluff Rockfall*

Amy Woermann – *Using Publicly Available Geotechnical Data in a Leapfrog Model*

Trent Williamson – *Plants as Indicators of Geology – How vegetation can reveal underlying geological features*

Lara Pieters – *Embankment design driven by site constraints: Waihoehoe Road Upgrade and Drury SH-1 Offramp*

Caroline Birse – *Avoiding a Slippery Slope: Factors influencing the Distribution and Effects of Earthquake Induced Landslides in New Zealand*

Archie Goodrick – *Cohesive strength lose in Loess – Project Examples*

Nathania Cheung – *Geotechnical Investigations for Puke Kapo Hau Wind Farm*

Kaylee Wu – *Dynamic Penetration Test (DPT)*

Ryder O'Neill – *Learnings about geology from an engineer's perspective*

Society Branch Reports

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

Auckland branch

BLAIR MATHESON

Blair is a Chartered Geotechnical Engineer with over 14 years of experience across New Zealand and Canada. Originally from Southland, Blair moved to Auckland in 2018 and has spent much of the past eight years working on major transport projects—designing bridges, highways, and rail infrastructure. He recently joined Soil and Rock Consultants, where he's now helping lead the technical delivery of commercial and residential developments across the upper North Island. Blair enjoys solving challenging ground problems, mentoring young engineers, and finding practical, down-to-earth solutions. Outside of work, you'll find him playing golf (mostly from the rough), five-a-side football, or out exploring the world with his girlfriend and their dog.

JOHN FRENGLEY

John is an Engineering Geologist at Engineering Geology Ltd. Having studied Engineering Geology in Dunedin and Auckland. He has 5 years of experience, being based in Auckland working on a range of residential, commercial and mining projects that have taken him across New Zealand.

SADEQ ASADI

Sadeq is a chartered geotechnical engineer at Jacobs, based in Auckland, with 15 years of industry and research experience in New Zealand and internationally. Throughout his career, he has held several technical and operational leadership roles, contributing to a wide range of major geotechnical projects. Sadeq completed his PhD at the University of Auckland in 2017, where his research focused on developing a method to classify crushable pumiceous soils and creating design guidelines for the liquefaction assessment of pumiceous soils.

AUCKLAND RECENTLY HOSTED the NZGS Symposium 2025 from 15–18 October under the theme "Geotechnical Horizons: Innovations and Challenges.". The event was a great success, drawing together practitioners, researchers and contractors from across Aotearoa to share experiences and advances in the field. A highlight was the strong participation from local firms and universities. The Auckland Branch extends sincere thanks to all presenters, volunteers and attendees who contributed to making the symposium a vibrant and forward-looking event.

Looking forward, the next Auckland Branch's event is the upcoming seminar "Erionite in New Zealand and Implications for Industry" on 3 December 2025 at the University of Auckland. The presentation, led by Dr Martin Brook and colleagues, will explore the occurrence of erionite in New Zealand, its identification challenges, and implications for geotechnical and construction practice. This is an excellent opportunity to stay informed on an emerging geohazard issue and continue the spirit of knowledge-sharing fostered by the symposium. This will be our final Auckland event for 2025, so I hope to see all our local members there to catch up and reflect on 2025.

Moving into 2026, we intend to have regular branch meetings and presentations on a wide range of topics. Please reach out if you have any feedback, or topics, research or case-studies you would like to share with the Branch.

Hamilton branch

BEN McKAY

Ben is a geotechnical engineer with CMW Geosciences, based in Hamilton. He has over 10 years of industry experience, with practical background experience in both construction and mining in NZ/AUS. Ben's key interests include landslide assessment, liquefaction, ground improvement and earthquake engineering. When he is not at work, he can be found rock climbing or pottering in his garden.

NEIL KUMAR

Neil Kumar is an Engineering Geologist at Beca in Hamilton, bringing 15 years of work experience to the table. He initially worked in the mineral exploration and mining industry, with the past six years dedicated to Engineering Geology. Beginning his career in Fiji, Neil relocated to New Zealand to join Beca. His experience spans a diverse range of projects and various ground investigation techniques. Outside of work he enjoys outdoor activities involving gardening and exploring nature.

THE MEMBERS OF the Waikato Branch were given the opportunity to network and learn about the interface between permanent design and construction in action for a bridge on a complex geotechnical site, thanks to Raj Ramgobin (CMW Geosciences) and Natasha Jokhan (Brian Perry Civil). There was some great banter between the presenters, and interesting discussion points raised by the audience.

The final planning stages of the shared YGP mini-symposium for Waikato/Bay of Plenty are underway at the time of writing this, with a great range of presentation topics – this is shaping up to be a great day for attendees, organized by last year's YGP peoples-choice winner Joao Oliveira (Engeo).

And finally, a call to action - we are still keen to hear from members regarding topics or ideas for future events – have you got some case studies or recent research you think the community would appreciate? Or maybe you just want to eat some pizza and look at rocks? Let us know, all ideas welcome!

Ben McKay & Neil Kumar

Tauranga branch

KIM DE GRAAF

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

MATT PACKARD

Matt works as a Geotechnical Engineer at ENGEO's Tauranga office. He has over 20 years industry experience, working primarily within the mostly sunny Auckland and Bay of Plenty regions. He has an interest in resilience based seismic design, complex retaining wall design and soft ground engineering and is currently looking after a number of challenging projects across our geologically diverse country. An NZGS Branch Co-ordinator for the Bay of Plenty in a past life, he's come back on board to help pester NZGS members into presenting more local events.

RHIANNON ROBINSON

Rhiannon is a Chartered Professional Engineer in Geotechnical Engineering with Engineering New Zealand Te Ao Rangahau. Rhiannon has worked as a Geotechnical Engineer with Beca since graduating from the University of Auckland in 2018 with a Bachelor of Civil Engineering with honours. Initially she worked for the Beca Auckland branch before transferring back to her hometown of Tauranga at the start of 2021.

Figure 1

Figure 2

IN THE SECOND half of 2025 we have had some excellent presentations. Firstly, in May we had Greg Snook (ENGEO) leading a site walkover of The Pitau, a luxury, 5 storey, retirement living complex with an 8000m^2 basement level, under construction in the heart of Mount Maunganui.

Later in May, on the 20th anniversary of the May 2005 Bay of Plenty storms, Marianne O'Halloran and Tony Cowbourne shared their experiences of the events and outcomes from the storms and landslides triggered across Tauranga (see Figure 1). This event provided an opportunity for our YGP and new geoprofessionals to learn more about some of the ongoing geotechnical issues we have in the Bay of Plenty.

In August, Berrick Fitzsimons (Beca) took members on a site visit around Takitimu Northern Link Stage 1 and presented on some of the interesting earthwork's challenges to date (see Figure 2).

In September we had Part 2 of the May 2005 Storm series, the presentation was focused on the 2005 Matatā Debris Flow and was given by Jeff Farrell (Whakatane District Council). The debris flow event occurred at the same time as the 2005 Tauranga Storm event and provided a diverse perspective for our members on policy and managed retreat.

We always welcome additional ideas from our members for presentations or site visits so do get in touch with any thoughts!

Kim De Graaf , Matt Packard, and Rhiannon Robinson

Taranaki branch

MATTHEW SULLIVAN-BROWN
Matt is a Geotechnical Engineer at BCD Group Ltd in New Plymouth. Matt Graduated from Auckland University with a BE(Hons) in 2016. He has recently taken his experience working in and around the Auckland region back to Taranaki to tackle the unique geotechnical challenges within the region. Matt's project experience ranges from smaller residential to large scale residential, commercial, and industrial developments.

LAURA JOHNSTON
Laura is a Graduate Geotechnical Engineer with HD Geo in New Plymouth and enjoys getting "hands on and hands dirty" in the field. Laura first graduated in 2010 from University of Plymouth, UK with BSc (Hons) in Geography and Ocean Science and has recently re-trained and graduated with NZDE (Civil) from Western Institute of Technology Taranaki and is continuing their academic journey with postgraduate study from University of Auckland.

Taranaki branch.

THE TARANAKI BRANCH has been busy since the last branch report.

The highly anticipated Te Ara o Te Ata - Mt Messenger bypass project site visit completed in May was well received by attendees. Geotechnical Lead, Danny Beasant (Tonkin + Taylor) did an excellent job of explaining some of the geotechnical challenges faced by the project team. The only downside – we wish we had booked in a more time to explore the site! Lesson learnt for next time. The project has recently reached a major milestone with the tunnel breakthrough. We hope this will allow for some exciting future visits to the site in 2026.

We enjoyed having Peter Fowler (Blade Pile NZ) visit the region in September for a lunch and learn to share information on the capabilities of the Blade Pile system and how blade piles can be another tool in the foundation toolbox for our local engineers.

We organised an opportunity to share project learnings and a discussion evening lead by Ben Dixon (Aurecon). Ben provided a fascinating presentation about the challenges faced by the project team as part of the Te Ahu a Turanga: Manawatū Tararua Highway project. Ben presented plenty of photos and explained some of the unique geological features and groundwater conditions encountered across the project. For us regional practitioners, it was an interesting insight to the complexities of geotechnical design and construction on an infrastructure project of national significance.

The Christmas break will be upon us before we know it. The Taranaki Branch have started planning for our 2026 branch events. Watch this space if you want to know more and keep up-to-date with recent research on Taranaki soils.

Matthew Sullivan-Brown

Wellington branch

SHIRLEY WANG

Shirley is a Geotechnical Engineer 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.

REBECCA TILL

Rebecca is a Geotechnical Engineer with Beca, based in Wellington. She is a graduate of the University of Canterbury with a BE (Hons). Rebecca's key interests include slope stability assessments, geotechnical investigations, and working on multi-disciplinary projects. Outside of work, Rebecca enjoys playing hockey and spending time outdoors.

MEENAKSHI PATEL

Meenakshi is a geotechnical engineer at ENGEO with almost five years of experience. Originally from Christchurch, she now calls Wellington home which has given her many an opportunity to get involved with slope stability and retaining designs, as well as the odd seismic assessment. Outside of work, Meenakshi loves all things creative and is an avid gardener.

THE WELLINGTON BRANCH has seen some changes in its representatives during 2025, with Rebecca Till and Meenakshi Patel joining Shirley Wang on the team. Rebecca is a geotechnical engineer with Beca, having moved to Wellington four years ago. She is excited for the opportunity to contribute to the local geotechnical community with some interesting events. Meenakshi is a geotechnical engineer with ENGEO who is keen to support and connect the Wellington geotechnical network through the committee.

Together with Shirley, they are both looking forward to running some exciting upcoming events and support the Wellington geotechnical community.

We extend our sincere thanks to outgoing members Christoph Kraus, Adam Smith and Brigitte Shepherd for their efforts over the last few years.

With the new representatives we are busy planning for some upcoming events for our members. This includes the planned streaming event from an international young geotechnical professional from Austria who will be presenting on "Risk-based decision-making in road tunnel operations" as well as a close look into Wellington's deep boreholes and what they can teach us.

We continually seek interesting presentations or workshops for our members. If you have any ideas or suggestions, please feel free to contact the local branch committee.

WE WOULD LIKE to extend our sincere appreciation to our advertisers in the *NZ Geomechanics News*. Your continued support plays a vital role in helping us bring a high-quality edition of the magazine to our readers.

Thank you for partnering with us!
Teresa Roetman

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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, PO Box 12 241, WELLINGTON 6144.

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Vice Chair	Ioannis Antonopoulos	awards@nzgs.org
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Elected Member	Liam Wotherspoon	committee@nzgs.org
Co-opted YGP Representative	Christoph Kraus	ygp@nzgs.org
Co-opted *NZ Geomechanics News* Editors	Camilla Gibbons Robert Kamuhangire	editor@nzgs.org
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IAEG NZ Representative	Ross Roberts	secretary@nzgs.org
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***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

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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/

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

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NZ Geomechanics News is published twice a year and distributed to the Society's 1000 plus members throughout New Zealand and overseas. The magazine is issued to society members who comprise professional geotechnical and civil engineers and engineering geologists from a wide range of consulting, contracting and university organisations, as well as those involved in laboratory and instrumentation services. NZGS aims to break even on publication, and is grateful for the support of advertisers in making the publication possible.

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National and International Events

2026

28-30 January
Taguig, Phillippines
2026 Southeast Asian Geotechnical Conference

25-28 March
Beirut, Lebanon
Pan Mediterranean Geotechnical Engineering Conference

9-12 April
Singapore
RocDyn-5 Fifth International Conference on Rock Dynamics and Applications

9-12 April
Tunisia
3rd International Conference on Advances in Rock Mechanics – TuniRock 2026

14-17 April
Ho Chi Minh City, Vietnam
8th International Conference on Geotechnics, Civil Engineering and Structures

28-1 May
Queenstown, New Zealand
(LaGER)Landslide Geo-Education and Risk 2026 – JT1 & JTC3 Workshop

14-19 June
Vienna, Austria
21st International Conference On Soil Mechanics and Geotechnical Engineering

6-10 August
Indore, India
12 International Symposium on Field Monitoring in Geomechanics 2026

24-26 August
Delft, Netherlands
International Conference on Advances and Innovations in Soft Soil Engineering

26-28 August
Brasilia, Brazil
LARMS2026 – X Latin American Congress on Rock Mechanics

14-19 September
EUROCK 2026
Skopje, North Macedonia

31-5 November
Delft, The Netherlands
XV IAEG World Congress

16-18 September
Athens, Greece
LARMS2026 – X Latin American Congress on Rock Mechanics – an SRM Regional Symposium

13-16 October
Graz, Austria
6th International Conference on Information Technology in Geo-Engineering

13-16 October
JTC Conference 6th International Conference on Information Technology

22-26 November
Fukuoka, Japan
ARMS14 – 14th Asian Rock Mechanics Symposium – 2026 ISRM International Symposium

2027

12-14th April
IS-GI Lyon 2027 – International Symposium on Ground Improvement Lyon, France

12-14 May
Vancouver, Canada
CPT'27: International Symposium on Cone Penetration Testing

9-12 June
Budapest, Hungary
XVIII Danube-European Conference on Geotechnical Engineering

9 -12 June
Graz, Austria
11th European Conference on Numerical Methods in Geotechnical Engineering

21-24 June
Graz, Austria
19th European Conference on Soil Mechanics and Geotechnical Engineering

2-4 September
Cairns, Australia
YGPC 2026

17-23 October
Seoul, Korea
16th ISRM International Congress on Rock Mechanics

26-27 November
Hanoi, Vietnam
The 6th International Conference on Geotechnics for Sustainable Infrastructure Development

2028

9-12 March
Chicago, USA
18th Pan American Conference on Soil Mechanics & Geotechnical Engineering & Geo Congress 2028

25-30 June
Aix-en-Provence, France
Eurock2028 – Andvances in rock mechanics and rock engineering to cope with increasingly extreme conditions

20-25 August
Istanbul, Turkey
19th European Conference on Soil Mechanics and Geotechnical Engineering

2029

1-5 September
Southampton, UK
6th International Conference on Transportation Geotechnics

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