NEW ZEALAND GEOTECHNICAL SOCIETY INC

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NZ GEOMECHANICS NEWS

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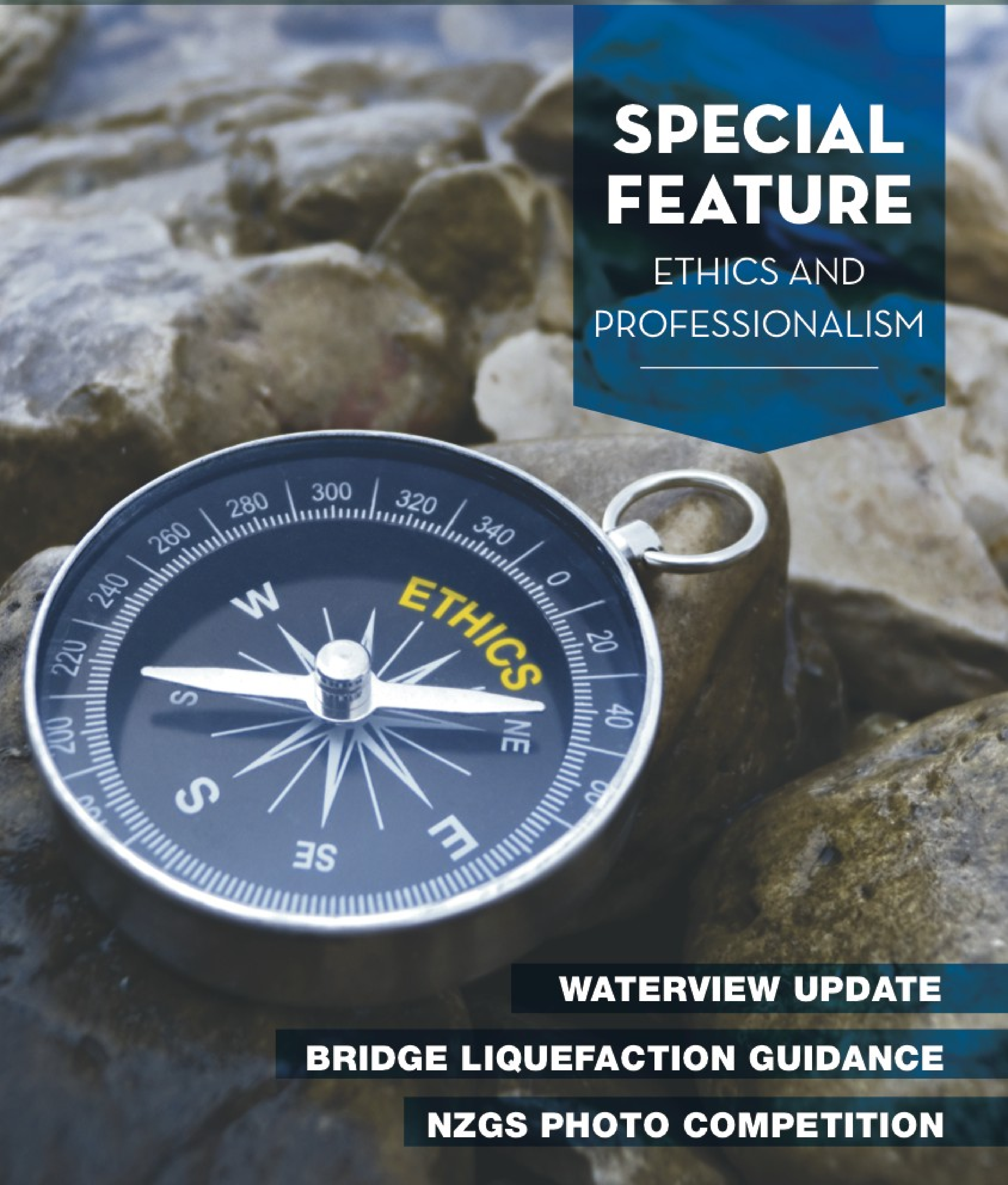

SPECIAL FEATURE

ETHICS AND PROFESSIONALISM

WATERVIEW UPDATE

BRIDGE LIQUEFACTION GUIDANCE

NZGS PHOTO COMPETITION

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Value Engineering eliminates any unnecessary use of product, without compromising on quality.
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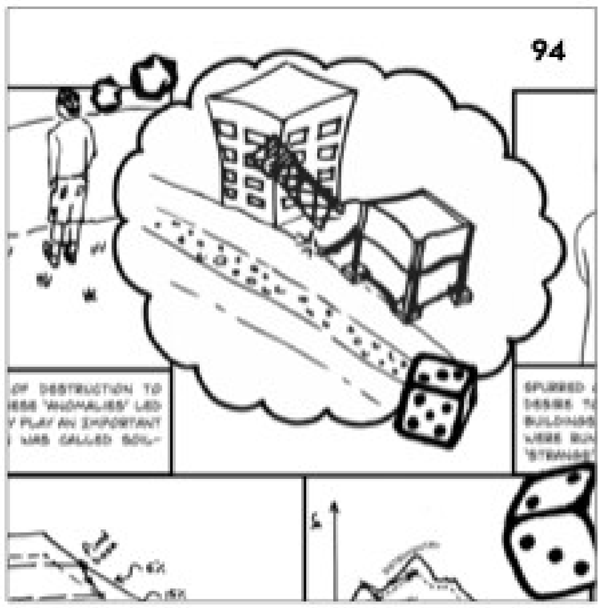
TURNING AROUND
Alice
OWAIRAKA
OUR BIGGEST CHALLENGE
Alice is a HUGE machine: The shield alone weighs 2400t, has a diameter of 14.46m and is 12.4m long.
14.46M
HOW WILL IT BE DONE?
24



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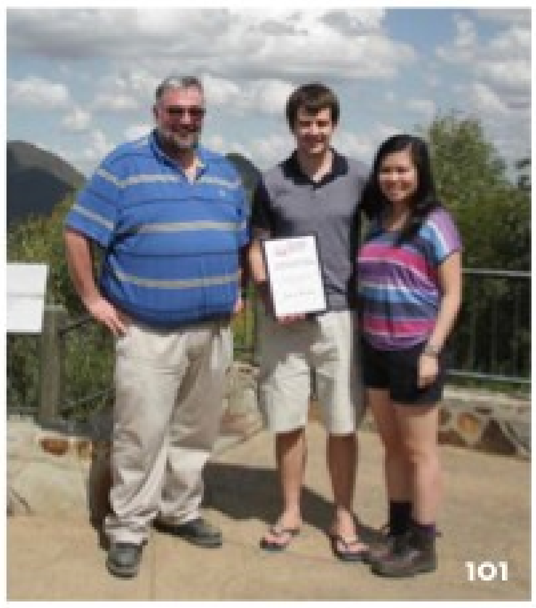

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VOTE NOW
FOR THE WINNER OF THE 2014 NZGS PHOTO COMPETITION BEFORE 02/2/2015
AT HTTPS://WWW.SURVEYMONKEY.COM/S/NZGS2014

GO TO PAGE 14

Gavin has specialized in geotechnical engineering since graduating from Auckland University in the mid 1980's. Following graduation, he spent seven years with Arup Geotechnics in the UK and Australia working on large building and infrastructure projects. In that period Gavin spent a year at Imperial College, London and was awarded an MSc and DIC in Soil Mechanics and Engineering Seismology.

Gavin Alexander
Chair, Management Committee

AS I NOTED last time, considerable effort continues to be put into the running of our Society and into contributions to the wider profession. The workload of your Committee and many other volunteers continues to exceed the time that people are able to make available, and I would like to thank all who provide a great deal of their discretionary time to contribute to our Society. I see no easing of the demand for a long time to come. There has been a lot happening, and I'd like to draw your attention to a number of key developments over the past six months.

CONFERENCES

The 10th Australia New Zealand YGP Conference was held on the Sunshine Coast, Queensland, in September, and from what I've heard was a great success. It's our turn to organize the next one, and I encourage our younger members to think about participating in it.

The next joint ANZ conference to be held in Wellington in early 2015 is nearly here. Around 170 papers have been accepted, and the programme is being finalised. There will be 31 general sessions spread over four parallel streams, so there's something for everyone. There are still a number of sponsorship spots available, and I encourage you to make the most of this opportunity to showcase your organisation to a truly Australasian audience. It's time to think about registration too – early-bird registration will have closed by the time you read this but do what you can to get to this once-in-twelve-year's event. I'm sure it will build positively on our successful Queenstown event of a year ago.

Later in 2015, the 6th International Conference in Earthquake Geotechnical Engineering is also receiving a lot of support from us. Abstract submission closed in October, and detailed planning is well underway. Judging by the number of abstracts received we expect numbers to be high for this conference, from both NZ and overseas.

SEISMIC GUIDELINES

I was very pleased to be able to formalise a partnership agreement with the Ministry of Business, Innovation and Employment (MBIE) to prepare guidance on specialist geotechnical subject areas where out of date, limited or no guidance exists. This will help us finalise the first three modules of the Geotechnical Earthquake Engineering Practice series (review of the existing liquefaction module, and writing of guidelines on foundation and retaining wall design), and, with EQC support as well, has seen work commence on guidance for geotechnical investigation for liquefaction assessment (Module 4) and a specification for ground improvement for residential development (Module 5A). Work is also underway on rockfall mitigation under the same agreement. I trust that the combination of Ministry funding and their oversight will help these projects get the focus they deserve, and get them completed and in use in the early part of 2015. There are a few other initiatives waiting in the wings, so we are in for a period of rapid development of our practice.

INDUSTRY ENGAGEMENT

The Society recently sought the views of members on aspects of an MBIE consultation document on the occupational regulation of professional engineers. With the assistance of Ross Roberts, we distributed an online survey which made the collation of your feedback much simpler. I'm not sure whether it was due to the subject matter or the online approach to collecting your responses, but we received considerably more feedback than we usually do – around 50 responses this time. Thank you to those who took the time to submit, it is important that we have a mandate when formally responding on behalf of the Society. While better than previous efforts, that still only represents less than 10% of our engineer members though, so there remains room to improve. Our submission is available on our website. It is clear that the professional engineers amongst us are in for some changes as the government response to the Canterbury Earthquakes Royal Commission works its way through the regulatory system.

I continue to represent the NZGS on the Engineering and Design Reference Group established by MBIE to overview building

and construction policy and operational developments. The sixth meeting of the ERG occurred in August, when we reviewed progress over the previous year or so and agreed short and medium term priorities. I've just attended the November meeting, where matters ranging from auditing of building consent design submissions, joint and several liability, the current health and safety reforms, occupational regulation, collaboration between engineers and architects, and a comprehensive review of Standards were all discussed. Keep an eye out for developments in all these areas! And finally.

This is my final report to you as I come to the end of a very full, demanding and satisfying two years as Chair. There have been many highlights over that time – in particular receiving the ISSMGE Outstanding Member Society award, our extremely successful Symposium in Queenstown last year, implementing PEngGeol, seeing Geomechanics News entering the digital age, signing the agreement with MBIE to work together on preparation of earthquake geotechnical engineering guidance, and establishing closer relationships with our sister societies NZSEE and SESOC. And not to mention raising my profile on our LinkedIn page!

Our Society's success relies on the hard work of many people and I'd like to acknowledge the considerable efforts of Amanda, our Management Secretary, Charlie Price, our Vice-Chair and Treasurer, the Management Committee and co-opted members, our editors, our numerous working groups, the symposium and conference committees and, last but certainly not least, our branch reps. Our Society thrives as a result of all of your efforts, and I thank you for that.

Thank you for the opportunity to lead our Society. I'll be passing the role of Chair to Charlie Price at the AGM in Christchurch in March, and am confident that he and Amanda will continue the great work of the recent years. I look forward to continuing to assist them as Immediate Past-Chair. Cheers

Gavin Alexander

Ross is an Engineering Geologist with Jacobs in Auckland. He trained in the UK at Edinburgh and Newcastle, and has since worked on projects ranging from motorways and railways to geothermal power stations and wharf structures. He has a particular interest in geohazard assessment, investigation and remediation. He has worked in the UK, Ireland, Australia, Java, Sumatra and New Zealand.
NZ Geomechanics News co-editor

Kelly is an Engineering Geologist with Opus International Consultants in Christchurch. She completed her BSc (Hons) in Geology at the University of Hertfordshire in 2001 and went on to work on a range of projects across the UK and Australia before settling in New Zealand in 2008. She has a particular interest in rockfall investigation, assessment and remediation.
NZ Geomechanics News co-editor

WELCOME TO THE December 2014 edition of NZ Geomechanics News, a heady mix of technical, professional and news articles which we hope will keep you informed and entertained.

Civil engineering has been in the media more than usual recently, and not always positively. Perceived and actual failures in Canterbury, as reported by the Canterbury Earthquakes Royal Commission, have driven the Ministry of Business, Industry and Employment (MBIE) to propose the occupational regulation of professional engineers. It has also encouraged IPENZ to make changes to its Code of Ethics; this is particularly relevant given that the NZGS rules bind all members, regardless of experience or affiliation, to abide by this Code. Our profession will be subject to a number of significant changes which will affect us all.

When the NZGS requested input to their response to MBIEs proposal, there were fifty responses – less than 5% of the membership. Although the responses received were of high quality, it is somewhat disappointing that the number was so low. We had hoped that the geotechnical community would want take an active part in steering this debate. Life is busy, and work naturally keeps people away from their non-fee earning professional interests, but such low participation on something so important suggests either a concerning level of apathy or a lack of understanding.

In this edition we explore a number of aspects of professionalism and ethics relating to the practice of geotechnics in New Zealand. We are keen to spark a debate, so if you have an opinion on this matter please write a letter or email to the editor.

Change is also a feature of our branch teams at this time of year. Pierre Malan and Doug Mason are stepping down from their roles as regional coordinators for the Auckland and Wellington Branch events respectively. Pierre and Doug have been pivotal in securing presenters and sponsors; without them and their teams our society would be much less valuable. Their voluntary work is fundamental to the continued development of our profession, and I hope you will join us in thanking them for the significant effort they've put in over the last few years. We wish them well for the future. Eric Torvelainen has volunteered to assist Luke and Kim in Auckland, Andrew Holland has stepped up in the Waikato Branch, as has Dolan Hewitt in Wellington.

Returning to the central theme of professionalism, by one definition a professional is someone who is motivated by more than personal gain, who voluntarily gives some their time to the development of their profession and their society. There can be no doubt that on this basis our branch co-ordinators are true professionals.

FOR MORE CONTENT check out our digital edition at www.nzgs.org.nz or search NZGS in your app store

Tell us about your project, news, opinions, or submit a technical article. We welcome all submissions, including:

- technical papers
- technical notes of any length
- feedback on papers and articles
- news or technical descriptions of geotechnical projects
- letters to the NZ Geotechnical Society or the Editor
- reports of events and personalities
- industry news
- opinion pieces

Please contact the editors (editor@nzgs.org) if you need any advice about the format or suitability of your material.

News – In Brief

OPUS MISSES OUT

Opus has announced the loss on an undisclosed project will cost it $5.5 million. The loss on the large project, which was in "a non-traditional discipline with unique and unusual complexities", would weigh on its New Zealand earnings, the Wellington-based company said in a statement. First-half margins were squeezed by the loss on a project and bidding costs of $1.9 million, and those costs were expected to rise to $5.5 million in the full financial year.

"Despite strong work in hand and five successful wins with our contracting partners on long-term NZ Transport Agency network outcomes contracts, the New Zealand result has been further impacted by the substantial one-off project loss," chief executive David Prentice said. "A full review is underway to identify possible areas to reduce the expected loss."

CALL FOR NOMINATIONS

Positions on the NZGS management committee, a dedicated team of ten, will be up for election in the new year. If you have enthusiasm for developing the profession and time to make a difference please consider standing. You'll learn a lot about the industry, make some great contacts, and enjoy the satisfaction of having made a difference. Look out for the call for nominations coming out in the very near future.

ANZ2015

The 12th Australia New Zealand Conference on Geomechanics (ANZ 2015) will be held in Wellington this February. Held every four years, and alternating between Australia and New Zealand, this promises to be the technical highlight of the decade for New Zealand geotechnics. Registrations are open at www.anz2015.com.

The NZGS will be co-hosting a stand with the Australian Geomechanics Society, manned by our own Amanda Blakey and Peter Robinson from the AGS. Head over to the stand and introduce yourselves to Amanda and Peter to be in the running for a mystery prize.

AUCKLAND ON THE MOVE

Auckland Transport has presented two transport budgets for capital works over the next 30 years. The basic programme lists what we can afford using current revenue streams. The second would use alternative funding to build the Auckland Plan transport network.

The additional funding would allow the completion of Penlink (an alternative route between Redvale and Whangaparaoa in the north of Auckland), the Northwestern Busway and park-and-ride facilities in Swanson, Westgate and Henderson, upgrade the Parnell and Newmarket train stations and the Tamaki Drive/Ngapipi Road intersection, build a third line on the Southern Line, build a park-and-ride at Pukekohe, upgrade the Half Moon Bay ferry terminal, deliver an additional $1 billion in arterial road improvements, and significantly improve the public transport network and the region's walking and cycling networks.

Aucklanders will be consulted in January on options for funding the additional developments.

EUROCK2015: 1ST ANNOUNCEMENT AND CALL FOR PAPERS

The ISRM Regional Symposium EUROCK 2015 "Future Development of Rock Mechanics" will be held in conjunction with the 64th Geomechanics Colloquium in Salzburg, 7-10 October 2015.

AMERICAN Wedding

Geoscience Consulting Limited, based in Wellington, Christchurch and Auckland, have recently agreed a merger with long-term partner ENGEO in December this year.

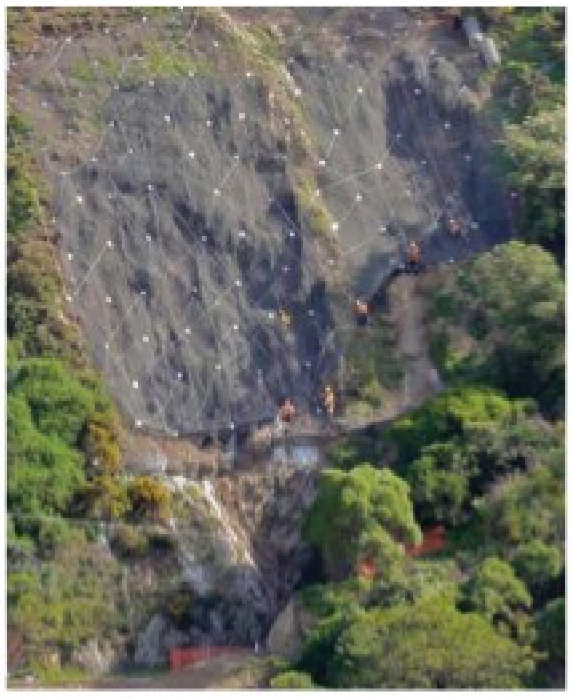
Geoscience has just been named in the Deloitte Fast 50 Index for 2014, making it one of the fifty fastest growing businesses in New Zealand over the last three years.

Founded in 1971, ENGEO is an employee-owned firm of geotechnical and civil engineers, geologists, hydrologists, environmental scientists and construction services technicians based in California. ENGEO began relocating professionals to New Zealand in 2012 as part of their support of Geosciences targeted growth. From the 1st of December 2014 the combined company with around 60 NZ staff will be known as ENGEO.

Guy Cassidy, ENGEO Associate, attributed the company's success to "a culture of service, collaboration, optimism, passion and excitement, with the firm committed to a close-knit atmosphere and a stimulating work environment". Guy also assures that "for both staff and the clients we serve it is business as usual with no changes to how we operate or changes to our employee ownership".

MALAN and MASON step down

Pierre Malan and Doug Mason are stepping down from their roles as regional coordinators for the Auckland and Wellington Branch events respectively. Over the past three years they both have been pivotal in securing presenters and sponsors; without them and their teams our society would be much less valuable. We thank them for their dedication and wish them well for the future.









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Above: Fletcher Building Chief Operating Officer Steve Evans and Building and Housing Minister Dr Nick Smith at the announcement event.

FLETCHER BUILDING has announced that it will build 237 new homes in Christchurch over the next 2 years, after the Government appointed the company as preferred development partner for the Awatea/Carrs Rd housing development.

Fletcher Building chief executive & managing director Mark Adamson said: "Our recent developments clearly show Fletcher Residential has the expertise and experience to provide affordable homes in a well designed environment that creates a community where people want to live"

The first Awatea/Carrs Rd houses will go on the market next year. The site is Crown land and the Government is expecting to see houses built at a rate of 10/month.

The development includes stand-alone & terrace homes available at both open market and affordable prices. 89 will be priced below $400,000 and 50 of those are likely to be developed on a shared-equity basis through the NZ Housing Foundation.

NZGS ONLINE

The NZGS website now hosts a huge collection of information including all the back issues of NZ Geomechanics News (free to all) and NZGS conference proceedings (in the member's area). Both are fully searchable to help you find relevant papers, and can be downloaded to peruse offline.

The new Member's Area also includes a list of NZGS members to help you contact your colleagues, and notes from the management committee meetings to help inform you about the work being undertaken on your behalf.

Next year we are considering a major upgrade to the website. If you have any suggestions for improvements or requests for content please contact editor@nzgs.org.nz.

STUDENT MEMBERS

Amanda Blakey (NZGS Secretary) requests that student members let her know when you have completed your studies and wish to become a full member of NZGS.

WORLD ACADEMY OF SCIENCE AND TECHNOLOGY

Members are advised to be very wary of the so-called World Academy of Science and Technology's circuit of International Conferences on Soil Mechanics and Geotechnical Engineering (ICSMGE), which appear to be imitating the ISSMGE events. In contrast to the ISSMGE events it appears that the organisation is likely to be a predatory publisher, allowing seemingly any article to appear in one of its journals so long as a fee is paid.

NEW LOOK

Hawkins Group Ltd announced in October that it was rebranding itself as simply Hawkins. The group has been repositioning itself for growth and wants to lift annual revenue from this year's projected $700 million to $1 billion through a mix of domestic and offshore projects. It is owned by the McConnell family through McConnell Ltd, and is New Zealand's largest privately owned construction and infrastructure business.

The group comprises 4 businesses – Hawkins Construction, Hawkins Infrastructure, Harker Underground Construction and project management arm the Canterbury Recovery Project, which will be combined to build on their strengths rather than isolate them in silos. Chief executive Geoff Hunt said in a release: "We think success is about making the right social, economic and environmental choices today, to build better communities tomorrow. People & society are expecting more from companies these days and we are responding by building a modern, high performing organisation in step with a rapidly changing world." Mr Hunt said Hawkins was investing in technology as a critical enabler to improve productivity and continued to focus on health & safety to improve performance.

FULTON HOGAN, the privately held construction firm, lifted annual earnings 43 per cent as it cut its costs in the face of falling Australian revenue, and is looking to grow its water, rail and airport units. Managing director Nick Miller said he was encouraged by the NZ government's plans for regional road projects.

Part of the gain came from Fulton Hogan expanding its new units in *water, rail* and *airports* beyond its traditional road business, and they now account for between 10 and 15 per cent of the group's annual revenue.

Mr Miller noted that, "We've made significant traction around the diversification of our brand and our activities this year, particularly into the rail, which we'd commenced last year, water, and airports space again. The driver there strategically is whilst roading will always be at the heart of our DNA, building a broader capability is critical in terms of our long-term future growth."

NZGS Rules

It has been a considerable number of years since the NZGS rules have been reviewed, and it has become apparent that there are some improvements that can be made to reflect our organisation and committee structure. Changes to the Rules have been debated by the management committee and some of these were introduced at this years' AGM. An updated set will now be put to members at the next AGM. The changes include numerous improvements to the use of English and therefore the clarity of the rules, as well as provision for life membership, inclusion of the treasurer role (which has existed in practice for many years, but is not properly defined in the rules), and giving the Management Committee the right to suspend or remove members for serious breaches of the rules.

THE MEGOLITH IS FORMED

AECOM agreed in July to buy rival URS for US$4 billion, creating a new leader in the industry that will be much larger than its closest competitor.

The move follows a turbulent period for shares in URS, which have dropped from a peak of over $55 last September to below $45 this June, before starting to rise sharply at the end of the month as speculation about a deal started to mount. Michael Burke (AECOM's chief executive) will lead the combined group, suggested that the deal would give the company the ability to reach "more clients in more industry end markets" with a wider range of services.

AECOM and URS are respectively the world's largest and third-largest engineering design companies by revenues, according to Engineering News-Record magazine (2013 figures). The deal would create a company with roughly double the turnover of WorleyParsons of Australia, the second largest in the industry by design revenue.

San Francisco-based URS, which has 50,000 employees worldwide, offers a range of project management, engineering and construction services, but has a particular expertise in energy. Its revenues were $11bn in 2013, roughly unchanged on 2012, but its earnings per share were down 21 per cent. Los Angeles-based AECOM has about 45,000 employees worldwide, offering a similar range of services to URS.

Our latest estimates from 2013 showed AECOM employing 36 NZGS members, and URS employing ten. The combined company would be the fourth largest geotechnical consultancy in the New Zealand market by employee numbers after Tonkin & Taylor, Opus and Beca.

Geomechanics LECTURE

NZGS would like to thank John Wood for his recent tour of NZ and presentation of the prestigious NZGS Geomechanics Lecture. Many members attended these presentations, and the feedback received has been spectacular. For those that missed out, we remind you that John will also be delivering his revised, definitive Lecture at the ANZ2015 Conference in Wellington in February.

WAITING FOR THE BOOM

THE BUILDING AND Construction Productivity Partnership Construction Pipeline Report issued in October shows that construction activity continues to grow strongly in New Zealand. This report is based on a compilation of known construction projects and economic forecasts of building and construction.

The value of all building and construction nationally is forecast to reach unprecedented levels, with a sustained rate of growth that has not been seen in 40 years. A peak 29% higher than current (and 28% higher than the previous peak in 2008) is anticipated within a few years.

Auckland continues to dominate the national demand for building and construction, accounting for over a third of all building and construction work, by value, from 2012 to 2019. The forecast value of all building and construction in Auckland steadily increases to a peak in 2017 ($13.1b) before reducing in 2018 and 2019.

Residential activity continues to drive growth in Auckland – forecast to more than double in value from 2012 ($3.5b) to the peak in 2017 ($7.3b). The number of multi-unit dwellings consented in Auckland each year is forecast to rapidly increase. Nearly 4,800 multi-unit dwelling consents are estimated in 2019 – nearly five times as many as in 2012 (just over 960).

Canterbury's rebuild activity increases in 2014 through 2016. The value of all building and construction in Canterbury peaks at $7.4b in 2015 and then steadily declines.

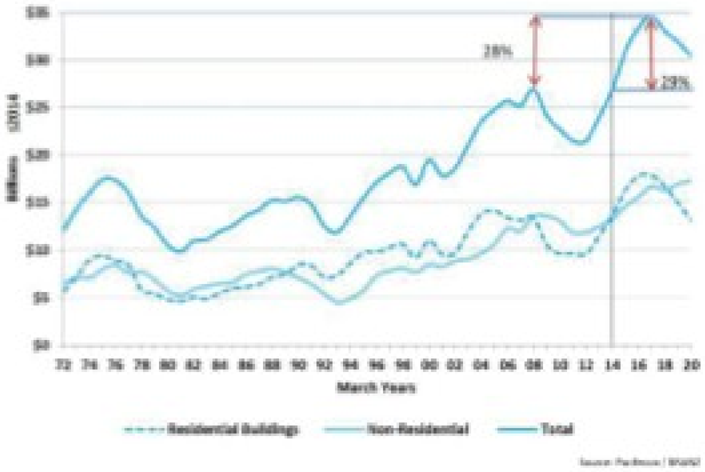


FIG 1: Value of all construction in NZ, historic and forecast

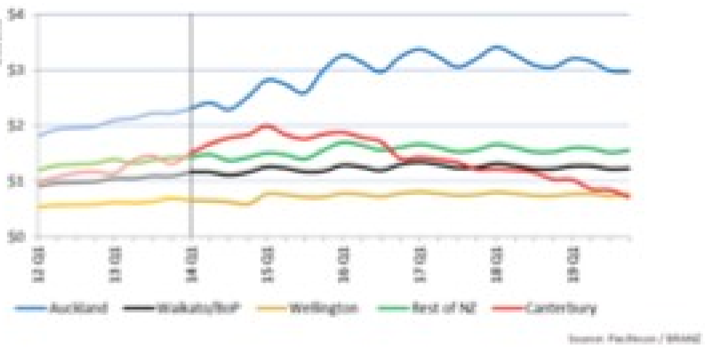


FIG 2: Projected value all building and construction by region (per quarter)

6ICEGE on a roll – World's premier Earthquake Geotechnical Engineering Conference comes to New Zealand

THERE HAS BEEN an excellent response to the call for abstracts for the 6ICEGE, with nearly 700 abstracts received from local and international practitioners and researchers. Because of the large volume of submissions, the final paper selection process will be rigorous and competitive, thus ensuring top quality presentations at the conference.

Along with abstract submission, we have also confirmed more than 10 mini-symposia that will focus on a number of geotechnical earthquake engineering subject areas. The organisers of each mini-symposia will work with potential presenters to develop each of these sessions.

The organising committee is pleased to announce that three prestigious named lectures have been confirmed as part of the conference program:

- 5th Ishihara Lecture
- 2nd Schofield Lecture
- TC203 Young Researcher Lecture

These have been organized under the auspices of the ISSMGE Technical Committees on Earthquake Geotechnical Engineering and related Matters (TC203) and Physical Modelling (TC104).

SIMSG ISSMGE

The program will include about 30 keynote and theme lectures by leading experts in the field, details of which will be announced in due course. The program will include a special session on the Christchurch earthquakes and another special session on recent major earthquakes (since the last ICEGE in Santiago in January 2011). A couple of discussion sessions on topics of great interest will also be organised with invited panelists and moderators.

WE THANK THE FOLLOWING SPONSORS FOR THEIR GENEROUS SUPPORT OF THE CONFERENCE

CONFERENCE PARTNERS	PLATINUM SPONSORS	GOLD SPONSORS
EQC Earthquake Commission Kōmihana Rūwhenua	Golder Associates	Beca
New Zealand Geotechnical Society Inc www.nzgs.org	Tonkin & Taylor	Ministry of Business, Innovation & Employment Hīkina Whakatutuki
UC Quake Centre		MWH. Building a Better World

We would also like to invite further sponsors to join this great conference event. Sponsorship at Silver and Bronze levels is particularly invited and encouraged. Don't miss this great opportunity to participate and contribute to an exceptional event, and showcase New Zealand's expertise in earthquake geotechnical engineering.

Waterview turning around

Alice's arrival at Waterview marks the completion of the first (the southbound) of the two Waterview tunnels. She is now halfway through her 4.8km underground journey between Owairaka and Waterview.

At Waterview she is being turned around, ready to bore southward in 2015 to build the northbound tunnel.

Turning Alice, the world's 10th largest-diameter tunnel boring machine, is a major undertaking. It's taking international engineering expertise and experience and a good measure of Kiwi ingenuity.

What is the Waterview Connection Project?

The Waterview tunnels are being built as part of the NZ Transport Agency's Waterview Connection project. The project involves construction of five kilometres of six lane motorway between Owairaka and Waterview, connecting the Southwestern (State Highway 20) and Northwestern (State Highway 16) motorways, therefore completing Auckland's 47km Western Ring Route.

Who is building it?

The Well-Connected Alliance comprising:

Fletcher

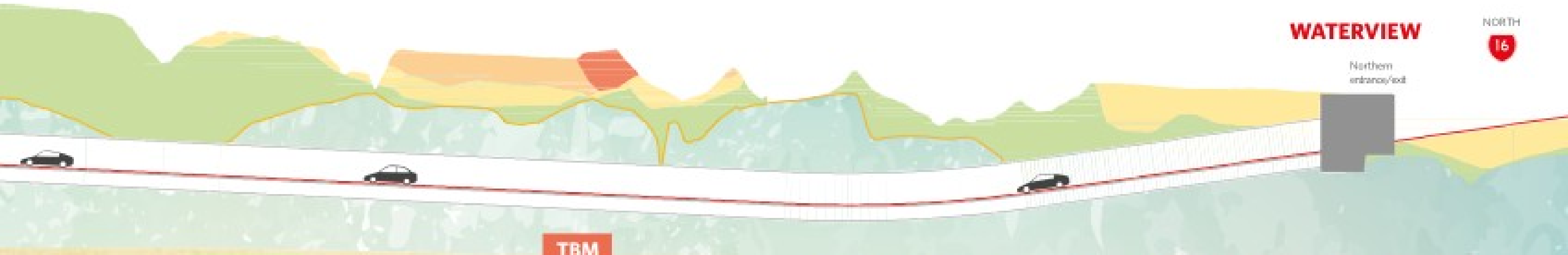
Beca

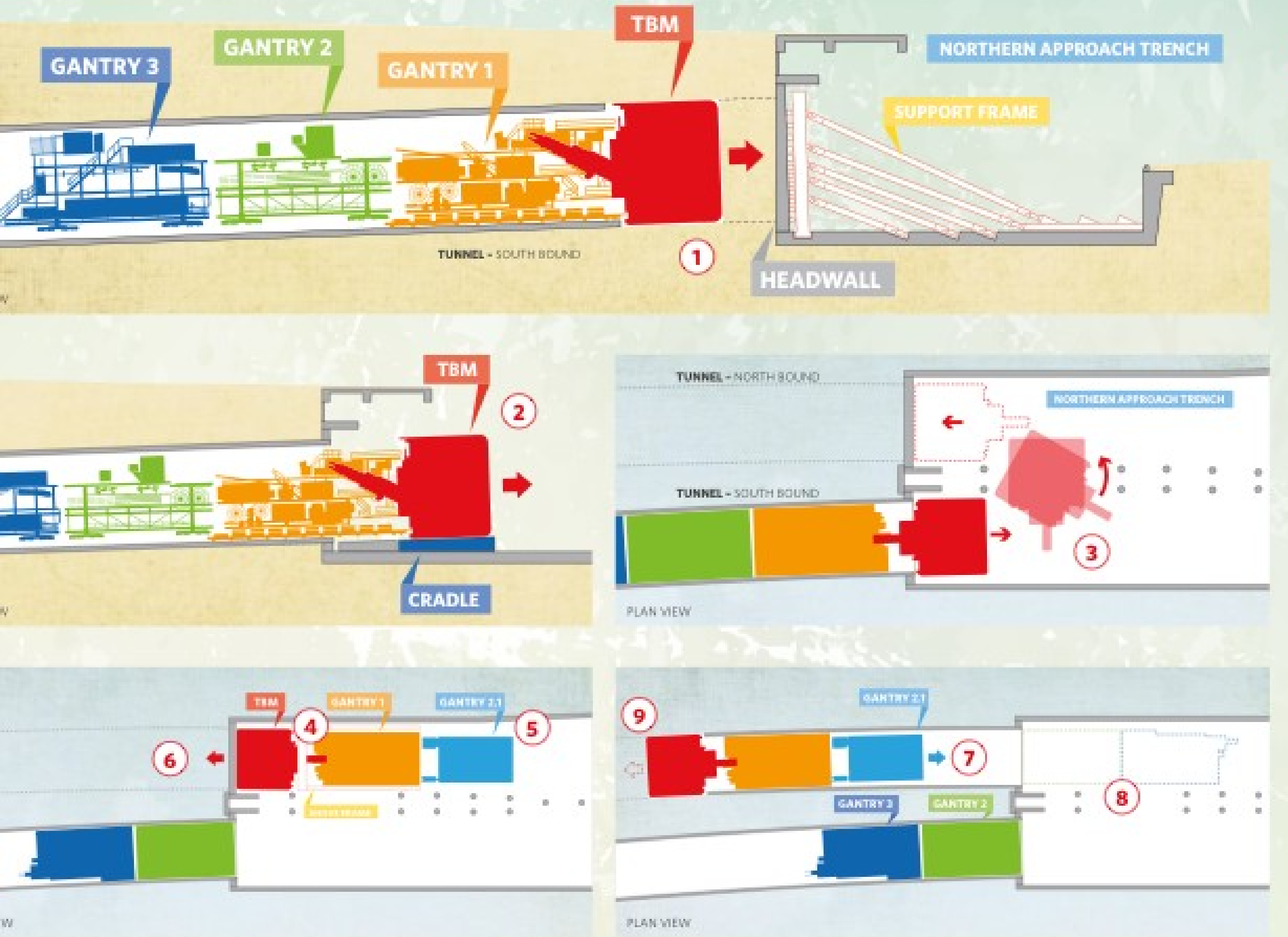
WELL-CONNECTED ALLIANCE PROUDLY DELIVERING

Waterview Connection

NZ TRANSPORT AGENCY

If you want to learn more about New Zealand's biggest infrastructure project to date, visit: www.nzta.govt.nz/projects/waterviewconnection and visit our Facebook page to receive regular updates on Alice: facebook.com/AliceTBM





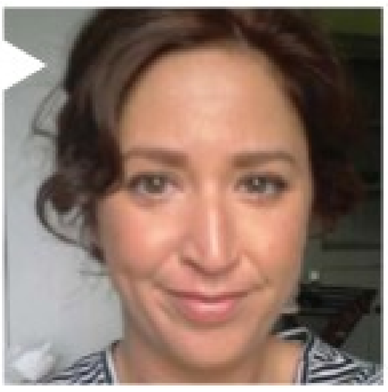
THE TURN IN DETAIL

1. As Alice approaches Waterview, a large steel frame is installed against the headwall of the Northern Approach Trench. The frame will support the wall as Alice breaks through.
2. Alice breaks through the headwall.

   The headwall support frame is removed, and a large concrete receiving cradle is installed just under Alice.

   Alice moves out onto the concrete cradle, and then onto a second cradle, this one made of steel.
3. Helped by hydraulic rams, the steel cradle is then slid across the cleaned and greased steel floor of the tunnel approach sump, turned and jacked up to her new start-up position.
4. The process is repeated for Gantry 1, which is positioned behind the shield and reconnected.
5. At this point, a smaller and lighter temporary gantry – called Gantry 2.1 – is introduced and positioned behind and connected to Gantry 1. (This gantry is needed because there's not enough space behind Alice for the full-sized Gantry 2.)
6. Alice is re-launched using the same shove frame from her initial launch at Owairaka and builds the first 300m of the second tunnel.
7. Gantry 2.1 is removed from the tunnel.
8. Gantries 2 and 3 are removed from the first tunnel, turned, and installed in position behind Alice.
9. Alice resumes tunnelling.
10. The culvert gantry is turned to begin installing the services culvert in the second tunnel.

What are Ethics?



Kelly Walker

Kelly is an Engineering Geologist with Opus International Consultants in Christchurch. She completed her BSc (Hons) in Geology at the University of Hertfordshire in 2001 and went on to work on a range of projects across the UK and Australia before settling in New Zealand in 2008. She has a particular interest in rockfall investigation, assessment and remediation.

RECENTLY, I HELPED facilitate a workshop on ethics. The group comprised a broad cross-section of junior and senior staff members across the disciplines of geotechnical, structural, mechanical, and environmental engineering along with administration staff.

The first question posed to the group; what are ethics? Initially, the attendees struggled to articulate their responses, however the eventual consensus was 'the right way to do things, in a professional manner'. When pressed on how they knew the right way to do things in a professional manner and whether they had read the code of ethics relevant to their disciplines many admitted it was a gut response to a situation i.e. it felt 'bad' or 'wrong'. Few were familiar with their respective codes of ethics or had not read them at all, which is concerning.

Throughout my career as an Engineering Geologist spanning twelve years across the UK, Australia and New Zealand I can recall one incidence of a senior staff member discussing ethics with me and giving me a copy of the IPENZ code of ethics to read during a particularly challenging job, which was two years ago. I'm not suggesting that my lack of familiarity with a code of ethics up to that point is the fault or responsibility of anyone else. However, I realised that I too had been operating on a gut feeling of 'this doesn't feel right' throughout my career.

I worked on a job, the memory of which still makes me feel incredibly uncomfortable. It became apparent during the tender evaluation stage that one of the contractors bidding for the job had missed an important part of the tender involving a significant cost. This resulted in their estimate being considerably lower than their competitors. The issue was raised and escalated, however the response from senior staff members was; "it's their responsibility to read the documents properly, accept the tender."

In black and white terms the contractor should have read the scope of the tender properly. The ethical dilemma; should we have accepted the tender, saving the client money or should this have been raised with the contractor prior to tender acceptance, thus giving them a chance to price accordingly? I felt this was the fair course of action, however I proceeded as instructed by more senior staff members. Quite simply, I didn't know what else to do and didn't feel confident to speak up.

The end result of this course of action resulted in the breakdown of a previously good relationship with a contractor, who was forced to cut corners at every chance to reduce the losses they made. At times this also resulted in questionable health and safety practices.

I believe ethical behaviour and professionalism should underpin the work we do as professionals no matter our chosen discipline. There will always be grey areas, but we need to be encouraged to discuss the role of ethics openly. We all have a responsibility to familiarise ourselves with our code of ethics and to speak up when faced with a situation that challenges them. As senior professionals and leaders we need to help guide junior staff members so that they feel empowered to proceed in a professional manner and pass on those skills as they progress in their careers.

NZGS ETHICS

All NZGS members are bound by the rules of the organisation to abide by the IPENZ Code of Ethics. I would recommend you take time to familiarise yourself with them if you haven't already.

http://www.ipenz.org.nz/ipenz/about_us/IPENZ_Code_of_Ethics.cfm

The Scala Penetrometer a Friend or Foe?

I FIND MYSELF writing this short, personal and probably provocative piece after posting a minor rant on a LinkedIn forum that was discussing the proposition 'The DCP – Essential Tool or Dangerous Relic'. That debate had been initiated by a friend and former colleague, of a well-known company and picked up by the AGS forum on LinkedIn. My colleague went on to chair an evening debate in Melbourne on the topic and has been good enough to supply me with some notes of the discussion that ensued.

My rant had been generated by my perception of the technical use and wide abuse of the Scala, but the debate also goes further with the health and safety of Scala users also coming to the fore. However I will limit this short piece to the technical application and (mis)use of the Scala.

Coming to New Zealand some years ago I had never previously heard of the Scala penetrometer. The closest similar tool I had ever used was a Mackintosh Probe, which, to the best of my knowledge has never been 'calibrated' against any geotechnical design parameter and was only ever used to determine 'the depth of soft', primarily peat and organic materials. So I have to confess coming to the Scala with a fairly jaundiced eye to start with.

Nonetheless, over the years I've come to recognise that the Scala is a useful tool. It definitely has a role in assessing soil conditions. It is a quick and cheap method of obtaining information that can be used in forming engineering judgements on how the soil/ground may behave. It was not long after arrival that I was introduced to Stockwell's paper on foundation design and Scala tests with its useful charts and figures for a rapid assessment of allowable bearing capacity – but wait a minute, what about all those nuisance factors like soil types, shape factors, slopes, eccentric loads, who reads the small print? Not many I argue.

I believe that key shortcomings of the test have been (and are still being) frequently overlooked in many reports I have seen over the years. The lack of understanding and inappropriate use of the data by some is worrying. I was recently asked what CBR equated to a design bearing pressure of 40kPa? The engineer concerned only had Scala results reported as CBR against depth. There was no soil description provided, no reference to the structure (a gravity retaining wall) and no acknowledgement of the steep downward slope next to the site. That was all the site investigation had comprised.

The test standard frequently referred to in NZ is NZS4402:1988 Test 6.5.2. This allows for reporting the blows per 50mm, 300mm penetration or the cumulative penetration per blow. So why do we see records giving bearing capacity or CBR plotted against depth - often with no reference to the correlations used in deriving these plots.

Why in reports do we so rarely see any allowance for, or comment on, skin friction build up on the rod sides where Scala's' have been hammered to depth with no accompanying widening by auger?

The shear ease of the test, its widespread use both inside and in particular outside the geotechnical community, and I believe above all its cheapness, has led to it being the primary choice of many as an investigative technique when it should at best be simply supplementing a more rigorous level of investigation.

I believe that we geotechnical engineers have done ourselves no favours in this as many Clients now expect and accept a cheap, and sometimes poor level of investigation as the norm and consider anything else to be 'gold plating'.

If treated well the Scala can be a good friend, but it can rapidly become a foe if misused.

The outcome of that Aussie debate - 'The DCP – Essential Tool or Dangerous Relic' – a dangerous relic.

What is professionalism?

Ross is an Engineering Geologist with Jacobs in Auckland. He trained in the UK at Edinburgh and Newcastle, and has since worked on projects ranging from motorways and railways to geothermal power stations and wharf structures. He has a particular interest in geohazard assessment, investigation and remediation. He has worked in the UK, Ireland, Australia, Java, Sumatra and New Zealand.
NZ Geomechanics News co-editor

This article draws heavily on the work of George Beaton, and his excellent essay is recommended further reading.

THE CONCEPT OF "professionalism" is commonly defined to include skills, knowledge and expertise, but also the virtues of trustworthiness and altruism (Beaton, 2010). However, defining it accurately has proven problematic.

MODERN DEFINITIONS

There have been numerous attempts to define professionalism. In common use, the term 'Professional athlete' suggests that anyone who gets paid for their work is a professional.

Abbott (1988) suggested that "a firm definition of profession is both unnecessary and dangerous," and proposed a loose definition "professions are somewhat exclusive groups of individuals applying somewhat abstract knowledge to particular cases".

Cheetham and Chivers (2005) compiled a list of a profession's characteristics, while admitting that the list was neither exhaustive nor definitive of all professions. A profession, they say:

- confers status within society
- organises itself into some sort of professional body
- is learned—i.e., requires prolonged and specialised training and education
- is altruistic (orientated towards service rather than profit)
- offers autonomy within the job role
- is informed by an ethical code of some kind
- is non-commercial
- has collective influence within society
- is self-regulatory
- is collegial
- is client-focused.

Professions Australia (1997) provided an alternative lying between these extremes: *"A profession is a disciplined group of individuals who adhere to ethical standards and hold themselves out as, and are accepted by the public as possessing special knowledge and skills in a widely recognised body of learning derived from research, education and training at a high level, and who are prepared to apply this knowledge and exercise these skills in the interest of others.*

It is inherent in the definition of a profession that a code of ethics governs the activities of each profession. Such codes require behaviour and practice beyond the personal moral obligations of an individual. They define and demand high standards of behaviour in respect to the services provided to the public and in dealing with professional colleagues. Further, these codes are enforced by the profession and are acknowledged and accepted by the community."

LITERAL MEANING

Bowen (2011) looked deeper into the meaning of the word, and surmised the following: If you break down the word professional, the Random House definition of the first part of the word "*profess*" (in the context of professions) is "*to declare oneself skilled or expert in; claim to have good knowledge as well as declaring that the profession exists*". The second part of the word "*ion*" denotes action on the part of the individual or conditions the individual is experiencing. The last part of the word "*ism*" denotes principles and/or doctrine that are adhered to.

When combining the parts of the word professionalism many perceptions can be made which may not align with the typical dictionary definition of the term. An alternative definition states "to lay claim to often insincerely", and discourse on the topic of professionalism often focuses on the insincerity of many professionals and professional organizations/institutions.

It should be clarified that professionalism and expertise are not the same. Expertise is defined by Random House as expert skill or knowledge which is based on opinion. Unlike the definition of professionalism there is no reference to the professional's character, spirit, or methods used to most effectively implement skills or knowledge. When only comparing the definitions it is clear that professionalism promotes a collective voice with a strong altruistic and ethical stance and expertise focuses on the skills of the individual professional.

HISTORICAL COMPARATORS

Trade guilds and unions provide a useful case history. Guilds took responsibility for the punishment or banning of members who were dishonest in their dealings, and oversaw training and certifying of apprentices. When a boy was finished with his apprenticeship, he became an employee of his master teacher and, in time, was called upon to produce a "masterpiece" to demonstrate his skill in his craft. Then he could become a master himself and set up his own shop, being granted autonomy by the guild. Conferring mastery or status within society, being organised into a body by occupation, requiring prolonged and specialised training and education, offering autonomy within job roles, having collective influence within society and being self-regulatory—all characteristics of the guilds—are some of the hallmarks of professionalism.

These early bodies inform aspects of the behaviour of today's professional associations and trade unions. This raises the question of what, if anything, distinguishes a profession from a craft. Is a profession simply a body of specialised knowledge, which associations and institutions impart to aspirants and then give them a certificate or degree to prove that they have it? Being an electrician, which takes several years of specialised training in apprenticeship programs and requires certification by recognised regulatory bodies, might be a profession by that definition.

Friedson (2001) gives a robust differentiator. He defines professionalism as follows: "*Professionalism may be said to exist when an organized occupation gains the power to determine who is qualified to perform a defined set of tasks, to prevent all others from performing that work, and to control the criteria by which to evaluate performance*". This in itself does not differentiate from guilds and trades. However, Friedson goes on to say that ethics is the very soul of professionalism:

"*Professionalism is not only a skills set in a given occupation; it is an ineffable something that the person exudes in manner, dress, speech and standards of practice that is palpably powerful: standards like honesty, due diligence, perseverance, willingness to listen and learn, creative thinking within a framework of training, and other qualities most people would be hard put to describe but which they expect in the professionals with whom they engage. Another word for these standards is "virtues" and the hard-to-describe something exuded is "trustworthiness": the sum total of these virtues.*

The doctor is trusted to have a patient's best interests in mind—an expectation prevails that he or she will do no harm and is sufficiently expert at least to do some good. The degree displayed on the wall certifies this, but so do the intangibles gleaned, depending on the community's culture, from the white coat, the clean fingernails, and the manner and the choice of words. Through these, the professional conveys intangible standards that win our trust that he or she is there to serve interests larger than him- or herself.

Wrapped up in our expectations of the professional—whether doctor, lawyer, pastor or other—is that he or she has our best interests at heart."

CONCLUSION

A profession is more than an advanced trade. There are personal and public expectations of the professional based on the notion that a profession is motivated by something other than raw gain. Professionals express their altruism through putting the needs of their clients and society at large before their own, and by serving in networks of responsibility within their profession, often on a volunteer basis. A certain degree of altruism or selfless service is expected. This applies equally to business professionals, whose explicit raison d'être is to make a profit. There is nothing inherently wrong with professionals wanting profit – as long as profit does not trump ethics or altruism.

Professionalism is most important in areas where there is a differential in knowledge between the client and the professional practitioner. Knowledge is power, and this power must be used ethically. We must act in our clients best interests by providing what our clients and society at large actually need, even if they have not explicitly asked for it.

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An All Too Familiar Situation?

RECENT EXPERIENCE IN the Wellington Region suggests to me, that despite extensive media coverage of the Canterbury and Seddon earthquake events, many people working in the building industry do not want to believe that "it can happen to us". This rather sad conclusion has been drawn following my interactions with a number of structural engineers and Council consenting officers who are comfortable working on and consenting projects in well documented areas of high liquefaction potential, with little or no geotechnical input. These areas are clearly identified on the freely available Regional Hazard Maps, a project jointly funded by several local Councils and the Regional Council.

It appears some structural engineers take the view the new building project they are working on is merely replacing those older buildings that stand around it (and have stood there for many decades) and that detailed geotechnical investigation, including liquefaction analysis, is not required. A view I expect was shared by many in Christchurch before 2010. Other structural engineers take the route of leaving ground anchor design (for seismic retrofit projects) up to the contractor – who often call on me for technical support. When we ask if there is a geotechnical investigation report for the site we are almost certain to hear laughter, followed by a resounding "no".

An example of this occurred last year when we were engaged by an anchor installation contractor. An investigation borehole was ordered from the contractor and based on the borehole data and site topography it was concluded that both a significant amount of liquefaction would occur and the site may spread laterally into the adjacent river during a code level seismic event. Several meetings and large amounts of time and money later, the Client and structural engineer agreed on an alternative foundation solution for the site. This additional cost could have been avoided with an appropriate geotechnical investigation conducted at the correct time in the project life cycle. Moral of the story: You pay for a geotechnical investigation whether you have one or not. When I enquired with the local Council if peer reviews are requested on similar geotechnical projects the response was "not really, landowners don't like to spend the extra money". Stunned silence.

Positive steps are being taken by MBIE and the NZGS in this regard, who are collaborating on a National ground investigation guidance document for liquefaction assessment to educate and inform practitioners around the country on current best practice in this field, which is a positive step. This effort should also be supported to educate Council consenting teams on this subject to enable them to push back where the level of geotechnical investigation is disproportionate to the scale of the project or known risks in the area.

Finally, I would support a nationwide amnesty day whereby all structural engineers may anonymously hand in their Scala probes and shear vanes. I acknowledge the fact that many structural engineers have a degree in civil engineering, hence some geotechnical knowledge. However most do not specialise in geotechnical investigation and design. You do not see many geotechnical engineers designing beams and columns. As set out in the IPENZ code of ethics, we need to operate within our areas of knowledge, training and expertise!

Guy Cassidy

Guy is an Associate Engineering Geologist working for ENGEO Ltd in Wellington.

Guy has completed a BSc(Hons) and MSc in Engineering Geology at Cardiff, University of Wales and worked in the UK for three years before moving to New Zealand in 2005. After a brief working trip back to London in 2008/09, Guy returned to New Zealand and currently enjoys the diversity of work available in such a geologically young and tectonically active country. Guy is currently an elected member of the NZGS National Management Committee and Conference Chair for the ANZ2015 Geomechanics Conference to be held in Wellington at the end of February 2015.

Specialist interests include landslide remediation, rock slope stability analysis, evaluation of geological risk around linear assets and the application of geophysics in the field of geotechnics and engineering geology.

Do I Trust this Data?

Ryan Milligan

Ryan Milligan is a Laboratory Manager for Geotechnics' Tauranga branch. He holds a Bachelor of Science (earth science) from Waikato University. Ryan started his career in civil/geotechnical testing in 2007 and throughout his career has worked on projects from Southland to Auckland and offshore. He is a member of the CETANZ committee and technical working group and has a proclivity for geotechnical instrumentation.

AS A PROFESSIONAL engineer it's possible that you've asked yourself this question when reviewing test results for your project. When you're carrying responsibility for a project, having confidence in the data you receive from third parties is an integral part of managing and minimising your professional and reputational risk.

WANT LESS RISK?

It's fair to assert that any reputable organisation or engineer is eager to minimise business risks wherever practical. Fortunately, professionals in the engineering industry have an excellent avenue to minimise their exposure to risk from third parties. Using IANZ accredited testing organisations for engineering testing purposes helps engineers make decisions on the basis of accurate and reliable results from organisations with verified competence.

WHAT DOES IANZ MEAN? WHO ARE THEY?

International Accreditation New Zealand or IANZ is an independent, third-party assessor of technical and scientific competence for professional organisations. A government entity whose primary purpose is to facilitate and maintain technical integrity of testing organisations for the "greater-good" of society rather than for commercial motives. Accreditation of a testing organisation by IANZ is assurance that the organisation has an internationally comparable level of technical integrity for their industry. IANZ itself gains further credibility by participating in regular evaluation of their processes by multilateral arrangement partners to ensure that they are assessing New Zealand testing organisations to the same standard as our international peers.

WHAT DO IANZ ACTUALLY DO?

In relation to civil/geotechnical engineering you want be confident that your chosen testing organisation is performing testing in accordance with valid and internationally recognised business process. In New Zealand this means operating an organisation in accordance with *NZS ISO/IEC 17025 General Requirements for the Competence of Testing and Calibration Laboratories*. This guideline shares many similarities to the ubiquitous ISO9001 but is geared towards laboratory based testing businesses.

To assess the technical integrity and business processes of an organisation; IANZ performs annual auditing using specialist assessors and industry technical experts. Together these groups make an assessment of accredited organisations to verify that:

A. **Personnel** are appropriately qualified, trained and have experience commensurate to their position and duties. A technical expert may request to observe actual work tasks to evaluate individual staff expertise during an audit.

B. The organisation's **accommodation and environment** is appropriate for the type of testing that they perform.

C. The organisation is using **test methods** that are validated and known to produce reliable results.

D. The organisation is using testing and measuring **equipment** that is compliant with the test methods, is calibrated and well maintained. Traceability of calibration is also critical for certain test equipment.

E. The organisation's **records and reports** are satisfactorily organised, maintained and meet international criteria for reporting.

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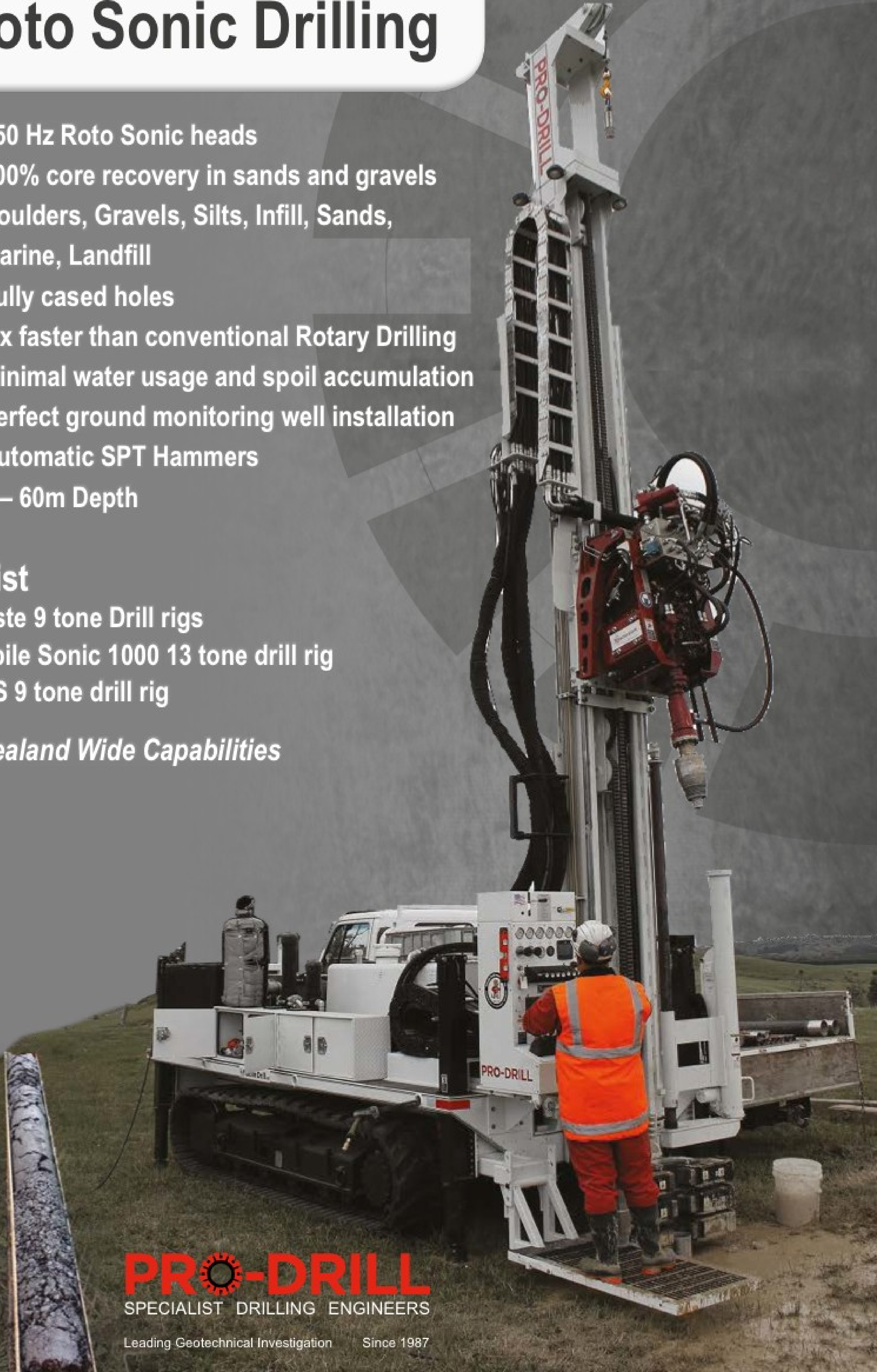

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G. The organisation takes part in intra-industry proficiency testing. Proficiency testing involves an identical sample being sent to multiple laboratories to be tested using the same methods. Results are compared and shared between participants. Significant deviations from the result consensuses may result in corrective action for the participants. IANZ plays a key role in ensuring results are properly reviewed and effective corrective actions pursued. Proficiency testing is an essential part of ensuring validity of test methods and parity of results between testing organisations in New Zealand. Non IANZ accredited testing organisations do not take part in this valuable process.

HOW DO I KNOW IF I'M GETTING ACCREDITED TESTING?

IANZ accredited organisations are encouraged by IANZ to reference their accreditation in reports and results by including the IANZ logo and/or a combination of words within the report/results stating "IANZ", "IANZ accredited", "Accredited Organisation" or similar statements.

A HIGH LEVEL OF QUALITY ASSURANCE THEN?

If you're getting your testing performed by an IANZ accredited organisation you can be confident that your quality assurance testing will be performed by an organisation following good procedures, comparable with industry peers. IANZ Accredited test results allow you to proceed with confidence in your data.

WHERE CAN I FIND IANZ TESTING FOR MY PROJECT?

All IANZ accredited organisations have their scope of accreditation publicly available on IANZ's website, www.ianz.govt.nz. Visit and search for your desired test to find providers across New Zealand.

WHAT IS CETANZ'S VIEW?

The Civil Engineering Testing Association of New Zealand (CETANZ) is largely comprised of IANZ accredited testing organisations that provide testing services to the civil/geotechnical engineering industry. Each year CETANZ members invest considerable time and financial resources into maintaining their IANZ accredited status. This investment creates and maintains the high level of data integrity that the civil/geotechnical engineering industry relies upon.

CETANZ has recently become aware of numerous projects that don't stipulate the use of IANZ accredited testing. An example may be when a contractor without IANZ accreditation is asked to perform compaction testing on a project under their control, which may be the most time/cost efficient way to perform some degree of quality assurance. However, scenarios like this are likely to increase the risk of uncontrolled variables as testing will be unlikely to realise the quality benefits of the aforementioned IANZ auditing process.

CETANZ believes quality assurance testing should be performed exclusively by organisations audited against a quality standard ensuring a means to gauge their performance against industry peers. CETANZ believes the use of an independent third party to regulate the quality process and avoid departure from accepted best practice is required. IANZ accreditation is endorsed by CETANZ to create and maintain high quality testing in New Zealand.

NEW ZEALAND GEOTECHNICAL SOCIETY INC

A Collaborating Technical Society of The Institution of Professional Engineers New Zealand

New Zealand Geotechnical Society

Submission on proposals to improve the occupational regulation of professional engineers in New Zealand

Thursday 30th October 2014

The New Zealand Geotechnical Society (NZGS) has over 1000 members who practice across the fields of geotechnical engineering, engineering geology and rock mechanics. We are a Collaborating Technical Society (CTS) of IPENZ. While our membership includes professional engineers and engineering geologists, and people working towards those qualifications, it also includes many others who work in different areas and at different levels. At April this year, our membership of just over 1000 included 357 Members or higher of IPENZ, 278 with CPEng registration and 15 with PEngGeol. A further 240 members held other grades of IPENZ membership. Consequently, around 60% of our members are formally aligned with the engineering profession and are bound by the IPENZ Code of Ethics and governed by its disciplinary procedures.

A significant issue for the occupational regulation of geotechnical engineers stems from the wide range of levels of geotechnical practice across our industry. The level of competence required to provide geotechnical advice for a house site in an already prepared residential subdivision is orders of magnitude less than that required for, say, a new multi-storey office building or underground rail line through an existing city centre. Our membership covers the full spectrum, with many at the lower end potentially (and reasonably) including "Geotechnical" as a practice field in their CPEng assessment documentation.

Many civil or structural engineers feel competent to undertake simple geotechnical investigations and to provide foundation design recommendations for "straight-forward" sites and projects, sometimes with a lack of awareness of hazards or geotechnical considerations that might come to light during construction or affect the long term performance of the development. It is currently very difficult to control these practitioners when they miss geotechnical hazards that would be obvious to a specialist. Consequently, the NZGS endorses efforts to improve the occupational regulation of engineers.

Our initial review of these documents has identified some proposals that warrant a formal view from the NZGS. To help determine that view, we posed a series of questions which are set out below, with our response, by individual proposal. While this is necessarily targeted to inform the NZGS submission, we have also encouraged our members to submit directly to MBIE and to IPENZ on the entire proposal. This response has been prepared by the Management Committee and reflects the feedback from our members, with an emphasis on the responses from those who hold CPEng registration.

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Proposal 1 – A chartered professional engineer (registered within an appropriate practice field – e.g. structural, geotechnical or fire) would be required to certify the structural integrity of commercial and multi-unit/multi-storey residential buildings requiring consent.

1.1 Is greater public assurance around the structural integrity of buildings likely to be gained by allowing CPEng registered geotechnical engineers to sign them off?

At face value, this proposal could be read as implying that a CPEng registered in geotechnical or fire engineering would be able to certify the structural engineering of a building. In the vast majority of cases, in our view, geotechnical engineers do not have sufficiently broad understanding of the related disciplines to take overall responsibility for building design. The Society's view is that, unless a CPEng engineer has been formally assessed in the appropriate area of structural engineering, then he/she should not certify the structural integrity of commercial and multi-unit/multi-storey residential buildings requiring consent or any other complex structure.

1.2 Should geotechnical aspects of these buildings be separately signed off by a CPEng registered geotechnical engineer?

a) Design
b) Construction monitoring

The NZGS strongly supports the provision of separate certification of geotechnical aspects of design and construction by a CPEng registered geotechnical engineer. Geotechnical engineers need to be involved through the design process, to check that what they have recommended is properly interpreted and implemented in the design. It is also crucial that geotechnical engineers make inspections during construction to ensure that what was expected based on initial investigations is what is actually unearthed during construction.

Requiring separate certification by each discipline would give the public far greater assurance that all aspects of building design have been properly considered. In requiring such separate certification, it is also important that it is clear who is taking overall responsibility for the design. Collaboration between the various disciplines is essential for successful design, and it seems logical that the lead designer of a structure is a CPEng registered structural engineer. Building developments are becoming increasingly complex and include specialists in construction activities and material supplies. These can be of significant importance to the building design and consideration should be made to requiring their certification by CPEng engineers with appropriate experience in those specialisms.

1.3 Should PEngGeol be an acceptable alternative qualification?

PEngGeol registered engineering geologists have a vital role to play in characterising the site conditions for design and in then confirming that what is exposed during construction is consistent with design expectations. In some cases, PEngGeol registrants have sufficient engineering knowledge and experience to provide the "full" geotechnical certification, but this is more likely to be the exception rather than the rule.

In summary, then, the NZGS considers that the overall design of commercial and multi-unit/multi-storey residential buildings requiring consent should be certified by an appropriately qualified lead CPEng, and that separate certification of the design and construction of other elements should also be sought. Such certification of each element should extend to construction monitoring.

Proposal 3 – The Registration Authority would publish more detailed information about professional engineers and their competency levels.

3.1 Do you support the publishing of more detailed information about professional engineers and their competency levels?

The NZGS strongly supports the publishing of more detailed information about professional engineers, their areas of practice and their competency levels. Currently there is no differentiation between a CPEng with a few years' experience on low rise residential projects and one with 20 years' experience on complex projects. Good definition of roles and publically available information will improve the ability of the public to select the right advisor and assist regulators assessing the reliability of designs that have been put forward.

3.2 The current CPEng registration process requires applicants to describe their Practice Area. Do you think this information, which is endorsed by an applicant's referees and is checked by the staff and practice area assessors, is sufficiently reliable and consistent to satisfy Proposal 3?

The NZGS considers that the existing system is generally reliable, but that does depend very much on the capability and experience of the staff and practice area assessors.

3.3 Do you think it is practical to standardise the description of Practice Area for geotechnical engineers?

There are many specialist sub-disciplines in geotechnical engineering, but the NZGS believes that a system could be developed to cover the majority of cases. We are keen to assist in the development of a standardised system.

3.4 If so, how would it be subdivided?

Subdivision would likely be by market area (commercial building, residential development, local and major infrastructure, industrial projects, for example) and by scale/cost/complexity. There may also need to be other subdivisions around the various geotechnical aspects such as site characterisation, foundations, retaining walls, slope stability, and design versus design management.

3.5 In either case (status quo or a standardised approach), should the current moderation by referees and assessors be tightened up, or is it adequate?

There is anecdotal evidence of isolated cases where referees appear to have a vested interest in the outcome and may be undermining the integrity of the registration assessment process. Overall, the current system of moderation is considered by the NZGS to provide a reasonable balance between rigour and efficiency.

In summary, the NZGS strongly supports the publishing of more detailed information about professional engineers and their competency levels. The currently collected practice area descriptions provide a reasonably robust starting point, but could be standardised to some degree to provide more consistent guidance to the public and to regulators.

Proposal 7 – The requirements for CPEng for structural, geotechnical and fire engineers are independently reviewed to ensure that they are set at an appropriate quality level and that the assessment process is rigorous and consistent for all applicants.

7.1 Do you think the requirements for CPEng for geotechnical engineers are set out at an appropriate level?

Overall, the NZGS considers that the current requirements for CPEng registered geotechnical engineers are set at an appropriate levels for new registrants. In isolated cases, there may be an erosion of standards with time through the continued registration assessment (CRA) process as some practitioners move into management roles but maintain the geotechnical practice field.

7.2 Do you think the assessment process is rigorous and consistent for all applicants?

Our members report a wide range of experience and hence views on the consistency and rigour of the current system, both as applicants and as peers of CPEng registered geotechnical engineers.

7.3 Do you think a higher tier of registration is required for geotechnical engineers involved in particular areas of design? (Note: we consulted on this last year, and while there was some support for a BGA style tiered structure, many felt that a more consistent application of the CPEng requirements would be sufficient).

The NZGS generally supports the development of a higher tier of registration. More critical or complex projects should be designed by more highly qualified engineers, and these engineers need to demonstrate that they are, in fact, suitably qualified. Such a higher tier of registration may well be easily identifiable from a more structured practice area description process as discussed in our response to Proposal 3 above.

In summary, the NZGS considers that the current initial assessment process is generally consistent and suitably rigorous. In some instances, the rigour may be eroded over time, though. A more structured system for defining an individual's practice area would help to retain that consistency and rigour through successive continued registration assessments.

Proposal 11 – Introduce powers for MBIE to obtain design and construction information from engineers about buildings that they have designed or certified.

11.1 At present, TLA's hold design and construction sign-off documentation for buildings and other structures. Do you think it is necessary for MBIE to have powers to obtain design and construction information from engineers?

The NZGS considers that the relevant TLA/BCA should be required to hold sufficient information about the design and construction of buildings. This may require documentation submission requirements to be standardised across the various TLA's. Any separate requirement to hold data will result in unnecessary costs (archiving, for example) and duplication.

Typically, the building owner also owns the design and should have complete design and construction records. Many design companies are only required to hold their design records for a limited period of time, and often much less than the operational life of the building. Design companies also hold the design records rather than individual CPEng design engineers. The company may consider the design to be propriety and hold intellectual rights. Therefore, individual CPEng design engineers may not keep complete design and construction records for all their projects.

11.2 If so, should there be a time limit on the period that such information is held?

The NZGS considers that this information should be held for the period that the building or structure remains standing. It may need to be referred to at any time, not just up to the end of its notional design life.

In summary, the NZGS considers that sufficient data should be obtained by the TLA/BCA at the time of consenting and again at the issue of a code compliance certificate to allow future investigation and assessment of building performance. This data should be held securely, and ideally with an off-site copy to avoid the challenges faced in Christchurch immediately after the February 2011 earthquake where Council records were not able to be accessed.

In closing, the NZGS appreciates this opportunity to participate in the re-shaping of occupational regulation of engineers in New Zealand, and is keen to contribute the development of initiatives to bring more consistency and rigour into geotechnical engineering activities. We also endorse the IPENZ submission in particular in relation to occupational regulation of engineers beyond the building industry.

The NZGS appreciates the opportunity to make this submission on behalf of our 1000 members, and is happy to provide further information if requested.

Gavin Alexander
Chair, NZGS Management Committee

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The development of design guidance for bridges in New Zealand for liquefaction and lateral spreading effects – NZ TRANSPORT AGENCY RESEARCH REPORT 553 – Alexei Murashev, Opus International Consultants Ltd

INTRODUCTION

New Zealand Transport Agency's Research Report No 553 titled "The development of design guidance for bridges in New Zealand for liquefaction and lateral spreading effects" was published on the New Zealand Transport Agency's website early this year. The report is a result of an NZTA research project towards the development of design guidelines for design of bridges on sites prone to liquefaction and lateral spreading in New Zealand. The research project included a review of seismic behaviour of bridges on sites prone to liquefaction and lateral spreading in New Zealand and overseas, review of available design methods for bridges against liquefaction and lateral spreading effects as well as detailed consideration of the bridge design framework in New Zealand and development of appropriate design methodology for New Zealand conditions.

The purpose of the project was to identify a clear set of available procedures for analysis and design that are based on observed seismic behaviour of bridges (reviewed case studies) and on the most recent research findings. The report summarises these methods and provides references to supporting materials (where such references are available).

The project was led by Opus International Consultants Limited (Opus). The research team comprised Opus (Dr Alexei Murashev, Messrs Donald Kirkcaldie and Campbell Keepa), the University of Canterbury (Dr Misko Cubrinovski) and the University of Auckland (Dr Rolando Orense).

A brief summary of the report is given below.

Dr Alexei Murashev

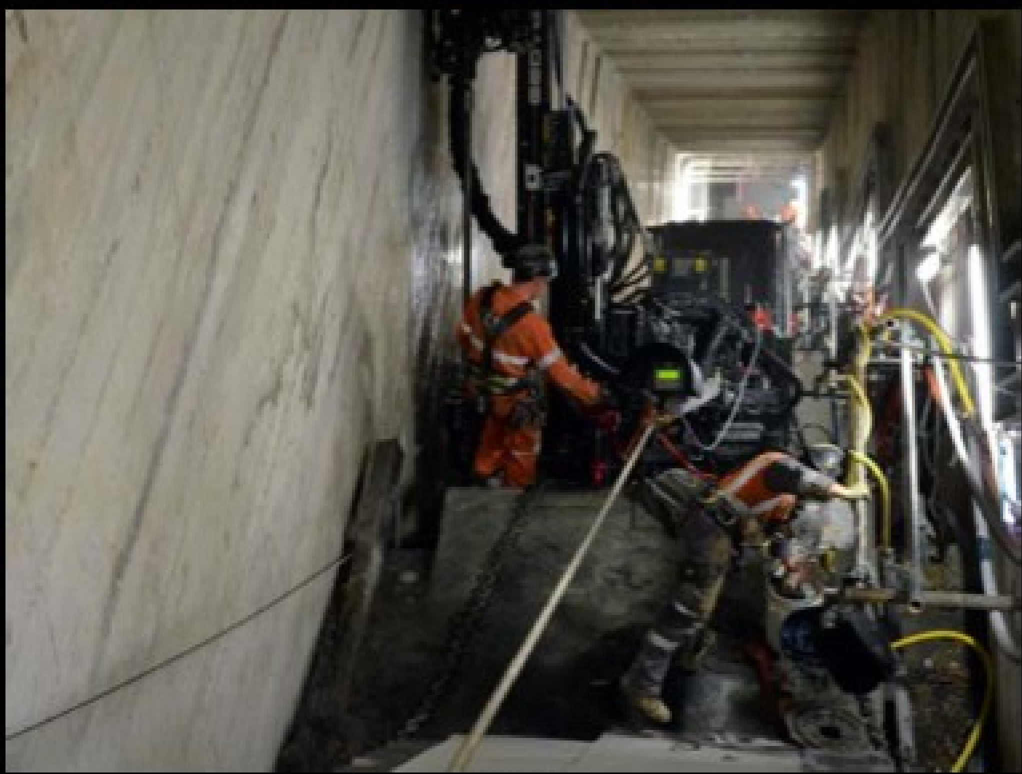
Alexei is a Technical Principal, Work Group Manager – Geotechnical Engineering & Risk and Opus Partner. He leads a Wellington-based team of geotechnical engineers and engineering geologists specialising in geotechnical investigations and design of bridges, retaining walls, roads, buildings and other structures. Alexei's team has particular strength in terms of assessment of soil potential for liquefaction and lateral spreading, design of structural foundations on liquefiable sites, ground improvement and numerical analysis of soil-structure interaction using the most advanced finite element software. Alexei has led a large number of research, bridge, roading and other infrastructure projects as well as earthquake risk studies. He published more than 40 technical papers on various aspects of geotechnical engineering. Alexei has been involved in the preparation of a number of the NZ Transport Agency's standards and guidelines for the design of roads and bridges, including the 3rd edition of the NZ Transport Agency's Bridge Manual.

LIQUEFACTION RELATED DESIGN ISSUES

Design of bridges for liquefaction and lateral spreading effects is a complex technical problem. The key considerations and issues that are important for the design of bridges on sites prone to liquefaction and lateral spreading include:

- Evaluating the liquefaction potential at the site. Key questions that need to be answered are:
  - At what level will liquefaction be triggered?
  - What is the extent and nature of liquefaction in a design earthquake event?
- Assessing the effects of liquefaction including magnitude of lateral spreading and ground subsidence, negative skin friction, bending of piles from soil inertia and displacements, as well as stresses induced in the superstructure
- Selecting the best value structural form and/or ground improvement concept to ensure that the magnitude of damage to the bridge and loss or reduction of service for the route is within acceptable limits

The quality of the developed design solution, the seismic behaviour of bridges and the ability of the bridge and geotechnical engineers to assess the post-earthquake



condition of the bridge are highly dependent on whether all of these issues have been adequately addressed.

PRELIMINARY CONSIDERATIONS

The preliminary considerations for the design of bridges on sites which are susceptible to liquefaction and lateral spreading include the following:

- Compliance with the structure performance requirements and standards given in clause 5.1.1 of the 3rd edition of NZ Transport Agency's Bridge Manual
- Liquefaction effects on structures including total and differential settlement of structure, down-drag loading on foundation piles, effects on services carried on the bridge, reduction of bearing and pull out capacity of piles, reduction in soil stiffnesses and overall stiffness of the bridge structure
- Lateral spreading effects on structures (loads from liquefied soils and from non-liquefiable crust, the effect of lateral spreading on the foundations, and the other way around, reduced strength properties for laterally spreading liquefied material, form of likely damage to piles, the effect of soil confinement on increasing pile post-elastic curvature capacity and foundation system pinning effect)
- Geotechnical pile capacity following liquefaction (generation of excess pore water pressure in liquefied layers and its effect on the bearing capacity, settlement and structural capacity of piles)

A preliminary geotechnical appraisal is carried out initially to make a preliminary assessment of the liquefaction and lateral spread hazards at the bridge site (as well as other geo-hazards) and to scope site investigations for detailed assessment. The preliminary appraisal is typically based on surface site observations and a desktop study. The depth of this assessment depends of the information available. At some sites, previous investigations or observations of performance in past earthquakes enable the geotechnical engineer to carry out a detailed assessment in this preliminary stage. As a minimum, the preliminary appraisal should establish:

- The geology of the site including probable soil types and the nature of deposition, the geologic age of the deposits, groundwater conditions, geomorphology and topography
- The seismicity of the site – proximity and orientation of active faults, fault rupture probability
- Likely site response characteristics (site subsoil class).
- The scope of site investigations required for detailed assessment of liquefaction and its effects on the bridge

GEOTECHNICAL INVESTIGATIONS

If the preliminary geotechnical appraisal indicated the presence of liquefiable materials, site specific geotechnical investigations may be required. A programme of geotechnical investigation should be developed to obtain sufficient information for the assessment of liquefaction potential of the site soils and for seismic analysis.

The geotechnical investigation to assess the liquefaction and lateral spreading hazard at a site can include:

- Cone penetrometer tests (CPT)
- Boreholes with standard penetration tests
- In-situ measurements of shear wave velocities
- Laboratory testing including particle size analysis, Atterberg Limits and triaxial testing (cyclic and monotonic)
- In-situ testing of seismic behaviour of soils using truck-based vibration machines or explosives
- Ground improvement field trials, if appropriate

The geotechnical investigation report should outline the methodology used in the interpretation of the collected information.

EVALUATION OF LIQUEFACTION TRIGGERING AND LATERAL SPREADING

The commonly accepted empirical stress based approach originally proposed by Seed and Idriss is recommended for liquefaction triggering analysis.

The seismic demand of the bridge site should be evaluated using the NZTA Bridge Manual (3rd Edition) where the unweighted PGA and the effective earthquake magnitude to be applied are derived for the relevant return period.

It should be noted that different methods for calculating CRR (cyclic resistance ratio) and CSR (cyclic stress ratio) must not be inter-mixed in assessing the liquefaction potential for a particular set of investigation data.

METHODS OF ANALYSING LATERAL RESPONSE OF PILES IN LIQUEFIED SOILS

Phases of Response

When evaluating the effects of liquefaction and lateral spreading on the performance of the bridge pile foundations using equivalent static analyses, it is necessary to conduct separate analyses for different phases of the response. The following analyses corresponding to different phases of the response should be carried out:

- Cyclic analysis without liquefaction, in which inertial loads that would occur in the absence of liquefaction are considered
- Cyclic liquefaction analysis, estimating the potential and consequences of liquefaction, and considering

simultaneous kinematic loads (due to cyclic ground displacements) and structural inertial loads while accounting for stiffness and strength degradation due to excess pore water pressures

- Lateral spreading analysis, estimating the potential for liquefaction and consequences of lateral spreading including large stiffness and strength degradation and kinematic loads due to large displacements associated with lateral spreading. Inertial loads may be considered in this analysis, but such loads are of secondary importance in the spreading phase, and can be ignored in many cases

Classification of Design Methods

Methods of analysis for bridges and piles in liquefied soils range from simplified methods using an equivalent static analysis approach to a rigorous time history analysis based on the effective stress principle. These analysis methods can be classified into three different categories:

- Pseudo-static analysis (PSA) or equivalent static analysis
- Direct dynamic time history analysis: (a) - effective stress analysis (ESA) considering effects of excess pore pressures and liquefaction through detailed constitutive modelling, and (b) - total stress analysis (TSA), dynamic analysis using total stresses or equivalent stresses (either ignoring excess pore pressures or considering them in a simplified manner). In the direct method of analysis (ESA and TSA), the response of the soil-pile-pier-abutment-deck system can be considered over the entire period of time from the initiation of shaking to the final stage of post-earthquake equilibrium and residual deformation of the bridge in a single analysis
- Substructure analysis methods which use some features of PSA, ESA or TSA but are essentially hybrid approaches tailored to address specific aspects in the performance assessment. For example, the well-known Newmark-type analysis (which is a simplified, user-defined time history analysis) would be a typical representative of this group of methods.
  The substructure method uses a set of separate but related analyses to assess the performance of a bridge subsystem

The effect of liquefaction and lateral spreading can be evaluated by considering a single member (e.g. a single pile), a subsystem of the bridge (pile group, pier piles or piled abutment), or the whole bridge. Each of these models is acceptable in the evaluation of the bridge performance provided that proper boundary conditions and modelling assumptions are used, and that there is a clear understanding of the analysis objectives and limitations. The cyclic response should be considered both in the transverse and longitudinal directions, whereas the lateral spreading response is commonly considered in the longitudinal direction of the bridge.

Pseudo-static Analysis (PSA)

In the PSA approach, a relatively simple beam-spring model is used for the soil-pile-bridge system to perform a nonlinear equivalent static analysis. This approach is also referred to as the beam on Winkler foundation or static pushover analysis.

Based on our review of available PSA methods, the following design methods would be appropriate for design and assessment of bridges in New Zealand and are described in the report:

- Cubrinovski method
- Pacific Earthquake Engineering Research Centre (PEER) method

In addition to the above PSA methods, the report provides a summary of the following two Japanese design methods:

- Architectural Institute of Japan (AIJ) Method
- Japan Road Association (JRA) Method

Time History Analysis

The dynamic response of the bridge is highly nonlinear and changes dramatically as it goes through the different transient stages from the initiation of strong shaking, through rapid build-up of excess pore pressures and consequent reduction in stiffness and strength of soils, development of liquefaction and post-liquefaction large ground deformation associated with earthquake-induced but gravity driven spreading. A non-linear time history analysis allows investigation of the dynamic response of the bridge while accounting for the complex soil-pile-pier-abutment-deck interaction or response of the bridge system in liquefying and laterally spreading soils. A sound numerical analysis that is well calibrated and executed provides the most realistic simulation of the actual bridge behaviour. Two types of such analysis are commonly carried out:

- Effective stress analysis (ESA) which permits evaluation of seismic soil-pile interaction while considering the effect of excess pore pressure and eventual soil liquefaction on the pile response
- Total stress dynamic analysis that has the same attributes as ESA but cannot simulate effects of the excess pore pressures and liquefaction with the same level of accuracy as ESA

Substructure Analysis

A design procedure proposed in MCEER/ATC-49-1 report is considered to be appropriate for the substructure analysis and is described in the research report.

STRUCTURAL MITIGATION APPROACHES

Two main approaches for mitigation of liquefaction induced lateral spreading are described:

- Minimising the effect of lateral spreading on the most vulnerable part of the structure and utilisation of the superstructure to support the abutments and piers
- Resisting lateral spread loads at the abutments

Either of these two options provides an opportunity to pre-determine the location of damage that is in either abutment or pier piles.

Soil structure interaction should be considered in the design of foundations under the effect of lateral spreading loads.

METHODS OF GROUND IMPROVEMENT

Ground improvement methods can be categorised into densification, solidification, drainage, reinforcement and replacement of liquefiable soils as well as increase of in-situ stresses by either surcharging the soils or lowering the groundwater table.

The report summarises the most common liquefaction countermeasures and gives brief descriptions of the principles behind the improvement mechanism associated with each category of the ground improvement methods along with some references detailing the design of ground treatment schemes and methods to evaluate the effectiveness of soil improvement.

The design and type of ground improvement measures depend on the site soil conditions. The installation process of mitigation measures can cause vibrations and settlements which can affect adjacent structures and buildings. It is therefore important to consider the installation process in design.

The main design issues which need to be considered in the design of ground improvement include the following:

- Size, location and type of treated zone
- Pore pressure migration due to differences in the density of the treated zone and the surrounding soil
- Possible increase of the ground motion amplification due to the increase in stiffness of the treated soil, which could be detrimental to the structure on top
- Dynamic fluid pressure
- Inertia forces acting on the improved ground and supported structure
- Influence of the structure on the stress state and seismic response of the improved ground zone
- Forces exerted by laterally spreading soil (the load

exerted by laterally moving ground itself and the force exerted by the unliquefied surficial crust riding on top of the liquefied ground)

CONCLUSIONS

Available design procedures that are based on observed seismic behaviour of bridges and on the most recent research findings have been summarised and design methods appropriate for New Zealand conditions have been identified. A large number of issues associated with liquefaction and lateral spreading and their effect on bridge structures have been considered and requirements for geotechnical investigations, evaluation techniques for liquefaction and lateral spreading, methods of ground improvements, methods of liquefaction analysis, structural mitigation solutions, construction and monitoring issues have been summarised. It was not possible to address all design issues in detail within the scope of the project, therefore the report also identifies areas where supporting information is not available and further research work is required. Design of bridges for liquefaction and lateral spreading effects is a complex technical problem and a large number of issues associated with the design process require further research and refinement. Additional work is currently being carried out to summarise the findings in a form of a technical document that will be later incorporated into the NZ Transport Agency's Bridge Manual and disseminated to the wider New Zealand engineering community.

ACKNOWLEDGEMENTS

The funding for the project was provided by the NZ Transport Agency. Dr John Wood of John Wood Consulting and Dr C Y Chin of URS are thanked for their detailed review of the research report. Messrs Nigel Lloyd, John Reynolds and Barry Wright of the NZ Transport Agency are gratefully acknowledged for the valuable comments, advice and support they provided to the research team through the course of the project. Members of the Project Steering Group Messrs Rob Jury of Beca and Lloyd Greenfield of the Christchurch City Council and thanked for their valuable input.

The Assessment of LIQUEFACTION and Lateral Spreading Effects on Bridges

– A. K. Murashev, D. K. Kirkcaldie & C. Keepa, Opus International Consultants, New Zealand; R. Orense, University of Auckland, New Zealand; M. Cubrinovski, University of Canterbury, New Zealand; J. N. Lloyd, New Zealand Transport Agency, New Zealand

ABSTRACT

This paper presents a summary of the outcomes from a research project commissioned by the NZ Transport Agency to develop design guidance for bridges in New Zealand for liquefaction and lateral spreading effects. The purpose of this research is to prepare design guidelines that will give clear design methodology and enable cost-efficient design of bridges, reducing the need for costly ground improvements on sites subject to these hazards. Once finalised, the proposed design requirements and guidelines will be incorporated in the NZ Transport Agency's Bridge Manual and disseminated to the wider New Zealand engineering community.

LIQUEFACTION RELATED DESIGN ISSUES

Bridges and highway structures located on sites with shallow groundwater tables or close to bodies of water can be susceptible to earthquake damage. Liquefaction of saturated sand-like materials (sands, sandy gravels, non-plastic silts), cyclic softening or cyclic failure of clay-like materials (plastic silts and clays) and lateral spreading can result in significant damage to bridges and highway structures. Earthquake damage to bridge abutment slopes on sites prone to liquefaction and lateral spreading may include ground failures, excessive lateral displacements and settlements. A large number of recorded cases of damage to bridge foundations due to the lateral displacements and settlements associated with liquefaction have been reported worldwide. There are also recent examples of earthquake damage to bridge structures caused by liquefaction and lateral spreading in Christchurch as a result of the 2010 Darfield earthquake and 2011 Christchurch earthquakes.

Soil deformation caused by liquefaction and lateral spreading can result in damage to bridge abutment and pier foundations and structural elements of bridges. A detailed report on the performance of highway structures during the Darfield and Christchurch earthquakes of 4 September 2010 and 22 February 2011 has been prepared by Wood et al. (2012). Conservative design approach quite often results in the need for costly ground improvement to fully mitigate against liquefaction, cyclic softening and lateral spreading at bridge sites. Regular review and

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improvement of design methods for bridges located on sites prone to liquefaction and lateral spreading is critical for the development of cost-effective bridge designs.

The design of bridges on sites prone to liquefaction and lateral spreading is a complex process that requires consideration of a large number of issues:

- Issues relating to geotechnical investigations: geotechnical investigation techniques for field and laboratory testing, determination of site subsoil class, development of a reliable ground model
- Issues relating to the evaluation of liquefaction and lateral spreading: determination of seismic demand, triggering of liquefaction, liquefaction-induced ground displacements, evolution of liquefaction and associated loads with time, settlement and lateral spreading including their uncertainties and outstanding issues, issues of practical concern (liquefaction of low plasticity silts, probabilistic vs. deterministic liquefaction evaluation, effect of groundwater conditions and artesian water heads, liquefaction at depth and maximum depth of liquefaction, cyclic mobility, partial liquefaction and limited shear strain), assessment of reduced bearing and pull out capacity of the foundation piles and of the effect of negative skin friction
- Issues relating to ground improvement: available methods of ground improvement, effectiveness and cost of ground improvement work, reliability and resilience of the adopted ground improvement
- Issues relating to the detailed analysis of the effect of liquefaction and lateral spreading on the bridge structure: methods of analysis, determination of model parameters, boundary conditions and pile-group effects, key uncertainties and the need for sensitivity analysis, soil - structure interaction, P-- effects, performance criteria (displacements, plastic deformation in piles and piers, need for post-earthquake repair or replacement of the bridge)
- Structural design issues: interaction between structural and geotechnical designers, definition of acceptable damage states and design for adopted damage states, structural detailing of bridges to reduce damage and increase reparability
- Construction issues: confirmation of ground conditions and design assumptions, control of ground improvement quality
- Post-construction monitoring issues: equipment / systems for monitoring post-earthquake performance of bridges (e.g. inclinometers in piles and/or ShapeAccelArrayTM that consists of a chain of sensor elements / segments joined together in such a manner that they can move in relation to each other in all directions except for twisting, each segment contains a multi axial MEMS-chip accelerometer which makes the segment act as an extremely accurate inclinometer), post-earthquake inspections and assessment

All of the above issues affect the design process, the quality of the developed design solution and finally the seismic behaviour of bridges as well as the ability of the bridge and geotechnical engineers to assess the post-earthquake condition of bridges.

In 2013 NZ Transport Agency commissioned a research project to develop design guidelines for design of bridges on sites prone to liquefaction and lateral spreading in New Zealand. The research project includes a review of seismic behaviour of bridges on sites prone to liquefaction and lateral spreading in New Zealand and overseas, review of available design methods for bridges against liquefaction and lateral spreading effects as well as detailed consideration of the bridge design framework in New Zealand and development of appropriate design methodology for New Zealand conditions (Murashev et al., 2014). It was not possible to address all of the design issues in detail within the first stage of the research project. Therefore areas where supporting information is not available and further research work is required have been identified.

PHASES OF RESPONSE

When evaluating the effects of liquefaction and lateral spreading on the performance of the bridge pile foundations using equivalent static analyses, it is necessary to conduct separate analyses for different phases of the response (Cubrinovski et al., 2009). The following analyses corresponding to different phases of the response should be carried out:

- Cyclic analysis without liquefaction, in which inertial loads that would occur in the absence of liquefaction are considered
- Cyclic liquefaction analysis, estimating the potential and consequences of liquefaction, and considering simultaneous kinematic loads (due to cyclic ground displacements) and structural inertial loads while accounting for stiffness and strength degradation due to excess pore water pressures
- Lateral spreading analysis, estimating the potential for liquefaction and consequences of lateral spreading including large stiffness and strength degradation, and kinematic loads due to large displacements associated with lateral spreading. Inertial loads may be considered in this analysis, but such loads are of secondary importance in the spreading phase, and can be ignored in many cases.

CLASSIFICATION OF DESIGN METHODS

Methods of analysis for bridges and piles in liquefied soils range from simplified methods using an equivalent static analysis approach to a rigorous time history analysis based on the effective stress principle. These analysis methods can be classified into three different categories:

- Pseudo-static analysis (PSA) or equivalent static analysis
- Direct dynamic time history analysis: (a) - effective stress analysis (ESA) considering effects of excess pore pressures and liquefaction through detailed constitutive modelling, and (b) - total stress analysis (TSA), dynamic analysis using total stresses or equivalent stresses (either ignoring excess pore pressures or considering them in a simplified manner). In the direct method of analysis (ESA and TSA), the response of the soil-pile-pier-abutment-deck system can be considered over the entire period of time from the initiation of shaking to the final stage of post-earthquake equilibrium and residual deformation of the bridge in a single analysis
- Substructure analysis methods which use some features of PSA, ESA or TSA but are essentially hybrid approaches tailored to address specific aspects in the performance assessment. For example, the well-known Newmark-type analysis (which is a simplified, user-defined time history analysis) would be a typical representative of this group of methods. The substructure method, uses a set of separate but related analyses to assess the performance of a bridge subsystem

Effect of liquefaction and lateral spreading can be evaluated by considering a single member (e.g. a single pile), a subsystem of the bridge (pile group, pier piles or piled abutment), or the whole bridge. Each of these models is acceptable in the evaluation of the bridge performance provided that proper boundary conditions and modelling assumptions are used, and that there is a clear understanding of the analysis objectives and limitations. The cyclic response should be considered both in the transverse and longitudinal directions, whereas the lateral spreading response is commonly considered in the longitudinal direction of the bridge.

PSEUDO-STATIC ANALYSIS

In the PSA approach, a relatively simple beam-spring model is used for the soil-pile-bridge system to perform a nonlinear equivalent static analysis. This approach is also referred to as the beam on Winkler foundation or static pushover analysis.

Based on our review of available PSA methods, the following design methods would be appropriate for New Zealand conditions:

- Cubrinovski method (Cubrinovski et al., 2009)
- Pacific Earthquake Engineering Research Centre (PEER) method (Ashford et al., 2011)

The main features of these methods are briefly described below.

Cubrinovski method

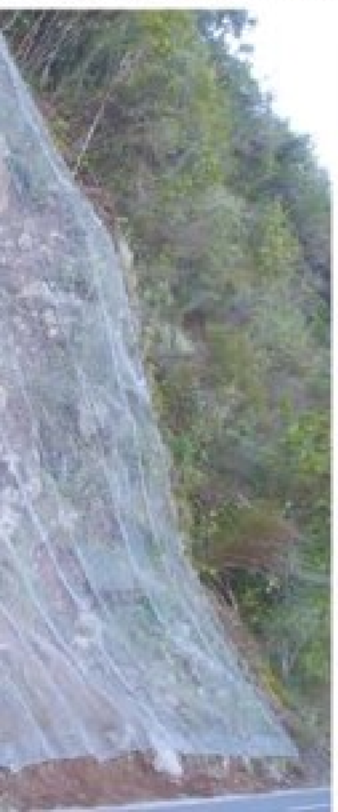
This PSA method can be applied either to a single-pile, pile group or the whole bridge. We will use the single pile model to describe the method.

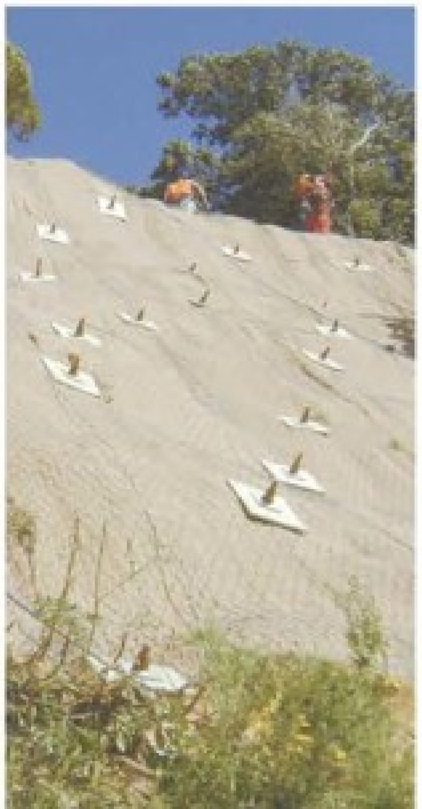
The typical beam-spring model representing the soil-pile system in the analysis is shown in Figure 1. The model can incorporate a multi-layered deposit with liquefied layers of different thickness and different depths, a crust of non-liquefiable soil at the ground surface, and deeper non-liquefiable layers including base layers at the tip of the pile. Parameters of the model are shown in Figure 1.

Given that the key requirement of the analysis is to estimate the inelastic deformation and damage to the pile, the proposed model incorporates simple but non-linear load-deformation relationships for the soil and the pile. The soil is represented by bilinear (elastic-plastic) springs, the stiffness and strength of which can be degraded to account for effects of nonlinear behaviour and liquefaction. The pile is modelled with a series of beam elements each having a tri-linear moment-curvature relationship. Commonly available software and finite element programs can be employed for the beam-spring model and analysis. Two equivalent static loads can be applied to the pile: a lateral force at the pile head (F) representing the inertial load on the pile due to vibration of the superstructure, and a horizontal ground displacement (UG) applied at the free end of the soil springs representing the kinematic load on the pile due to lateral ground movement (cyclic or spreading) in the free field. As indicated in Figure 1, it is assumed that all of the ground surface displacement is accommodated within the liquefied layer, that the non-liquefied crust at the ground surface moves as a rigid body and undergoes the same ground displacement as the top of the liquefied layer, and that the non-liquefying base layer is not moving. Deformed shapes of piles and bending moment distribution in the piles can be obtained from the analysis. If the pinning effect of the bridge deck needs to be included, a full bridge model including the bridge deck should be developed.

PEER Method

The PEER method of analysis (Ashford et al., 2011) is very similar in concept to Cubrinovski method. A beam-spring model is used (Figure 2) in which non-linear soil springs and nonlinear (tri-linear or bi-linear) moment-curvature





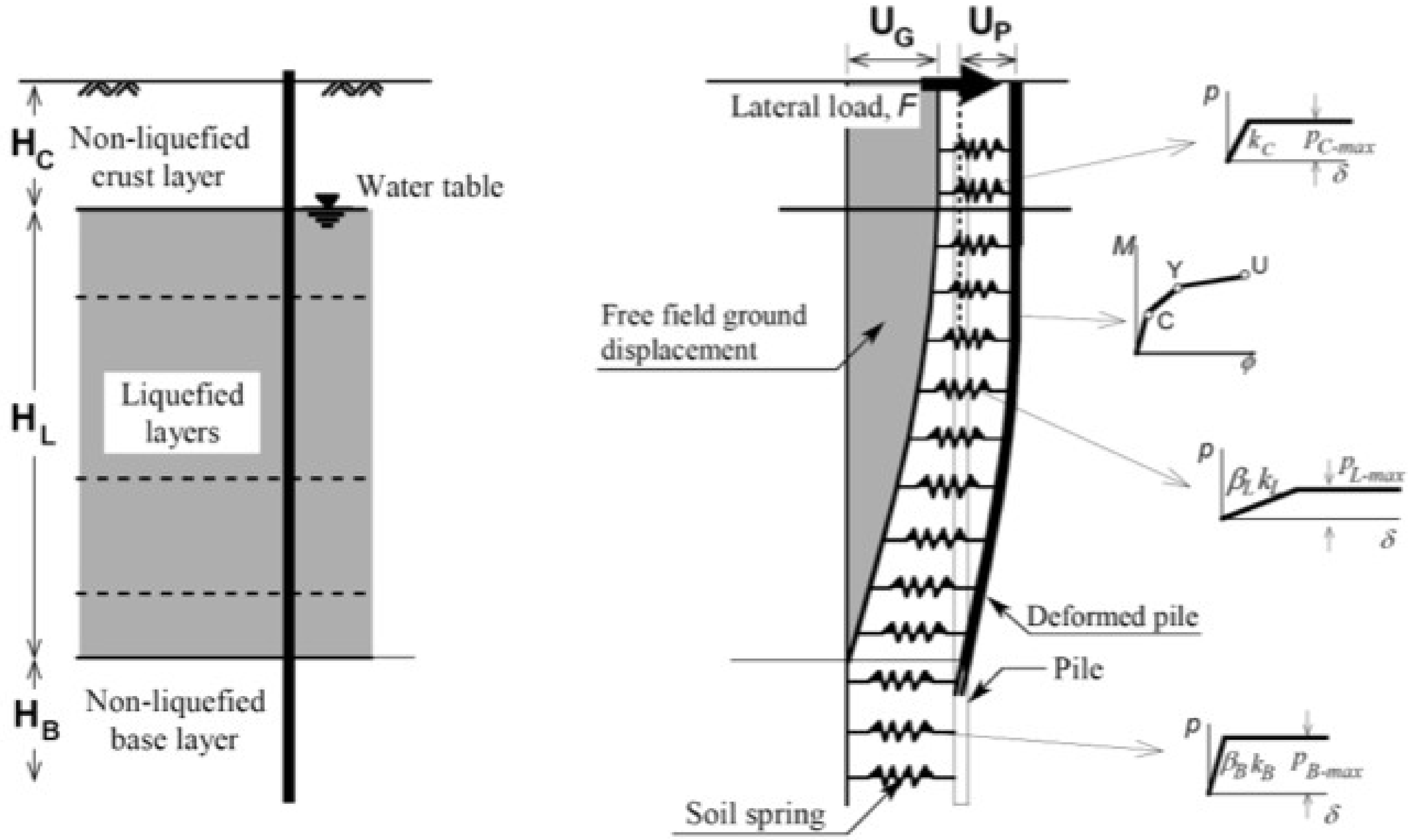


Figure 1: Beam-spring model for pseudo-static analysis of piles in liquefying soils: model parameters and characterization of nonlinear behaviour (Cubrinovski et al., 2009)

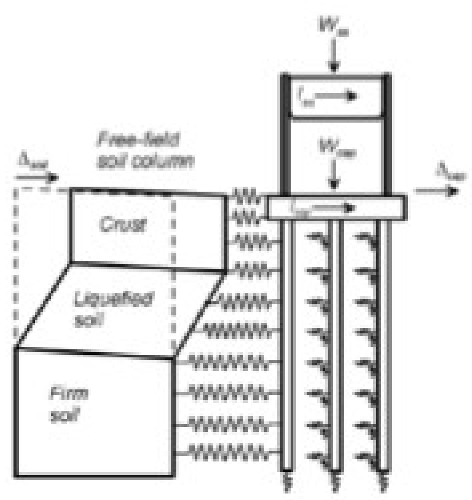


Figure 2: Pile-group model in the PEER equivalent static analysis with imposed soil displacements

relationships are employed for the soil and the pile respectively. Lateral, vertical and bearing soil springs are defined in the model, however it is stated that guidance for the vertical springs and bearing springs cannot be provided. For the liquefied soil, reduction factors (or so-called p-multipliers) are applied to the non-liquefied soil spring (p-y) resistance.

The method can be used for the analysis of a single pile, pile group or whole bridge. Analyses in both transverse and longitudinal directions, and separate analyses for non-liquefaction and liquefaction cases are recommended.

The displacement based approach is adopted as the best method for lateral spreading analysis using the equivalent static approach, in which free field ground displacements are applied at the base of the soil springs. A number of methods are recommended for estimating the free field displacements (Ashford et al., 2011). However, considering the uncertainties in estimating lateral spreading displacements, it is recommended to employ several different methods and to use a range of anticipated displacements in the analysis.

The PEER method suggests that for design purposes it is prudent to assume that sufficient lateral spreading displacement occurs during strong ground shaking, and therefore analyses should be performed in which lateral spreading displacements and inertial loads are applied simultaneously.

In addition to the above PSA methods, the report (Murashev et al., 2013) provides a summary of the following two Japanese design methods:

- Architectural Institute of Japan (AIJ) Method (Tokimatsu and Asaka, 1998)
- Japan Road Association (JRA) Method (JRA, 1996)

AIJ Method

The AIJ method was developed for the design of piles for building foundations. It is one of the most developed and widely used methods for the analysis of pile foundations in Japan. The key features of the AIJ method are as follows.

The displacement-based approach using a beam-spring model is adopted with bi-linear soil springs and

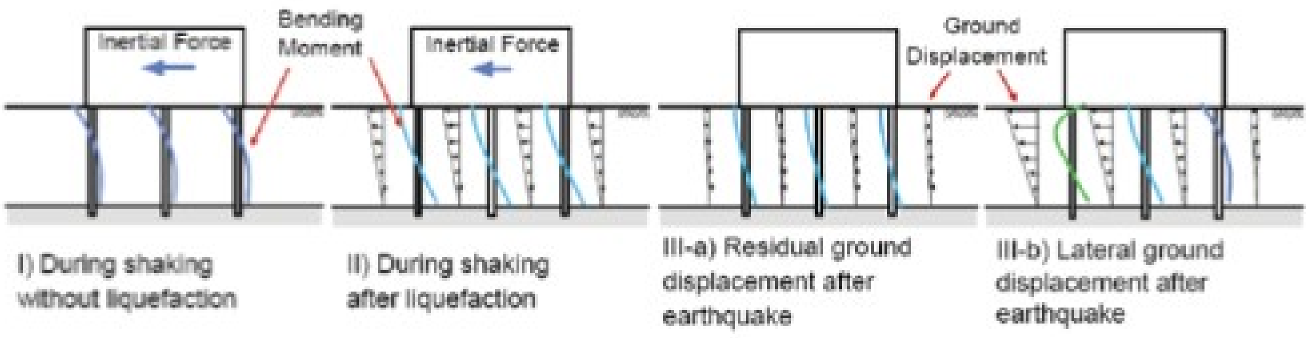


Figure 3: Separate analyses and loading conditions (inertia and kinematic loads) for three different stages of the response

tri-linear moment-curvature relationships for the pile. Free field ground displacements are applied at the base of the soil springs to simulate effects of transient ground displacement and spreading displacements. Stiffness of soil springs is defined using the subgrade reaction approach. Recommendations are given on the reduction factor for liquefied soil springs; the reduction factor is given as a function of the SPT blow count and depth. Three separate analyses are recommended for the pile foundations: no-liquefaction analysis (Case I), liquefaction analysis (Case II) and ground displacement analysis without liquefaction (Case IIIa) and with liquefaction, i.e. under lateral spreading conditions (Case IIIb) as illustrated in Figure 3 (Tokimatsu and Asaka, 1998).

Effects of ground displacements are neglected in no-liquefaction analysis (Case I) with the inertia force being the only applied load in this analysis. In the liquefaction analysis (Case II), the effects of inertia and ground displacement demands are combined.

JRA Method

The JRA method was developed specifically for highway bridges. Highway bridges in Japan are very large and massive structures and large diameter piles in a group are typically used for the bridge foundations. The piles are very stiff and strong and this is reflected in the design and analysis philosophy of the JRA Method.

The JRA method also considers three cases using separate analyses for: no-liquefaction case, liquefied soil case and laterally spreading soil case. The JRA analytical model for a pier-pile structure using a beam-spring model is shown on Figure 4. Bi-linear springs and tri-linear relationships are used for the soil and pile respectively. Effects of liquefaction are accounted for by multiplying the stiffness (subgrade reaction coefficient), ultimate soil reaction and skin friction capacity by a degradation coefficient which is defined as a function of the factor of safety against liquefaction triggering and depth.

Force-based approach (with prescribed lateral loads equal to the ultimate passive soil pressures applied to the pile) is used for lateral spreading analysis. Rankine passive pressures are specified for the non-liquefied layers, while

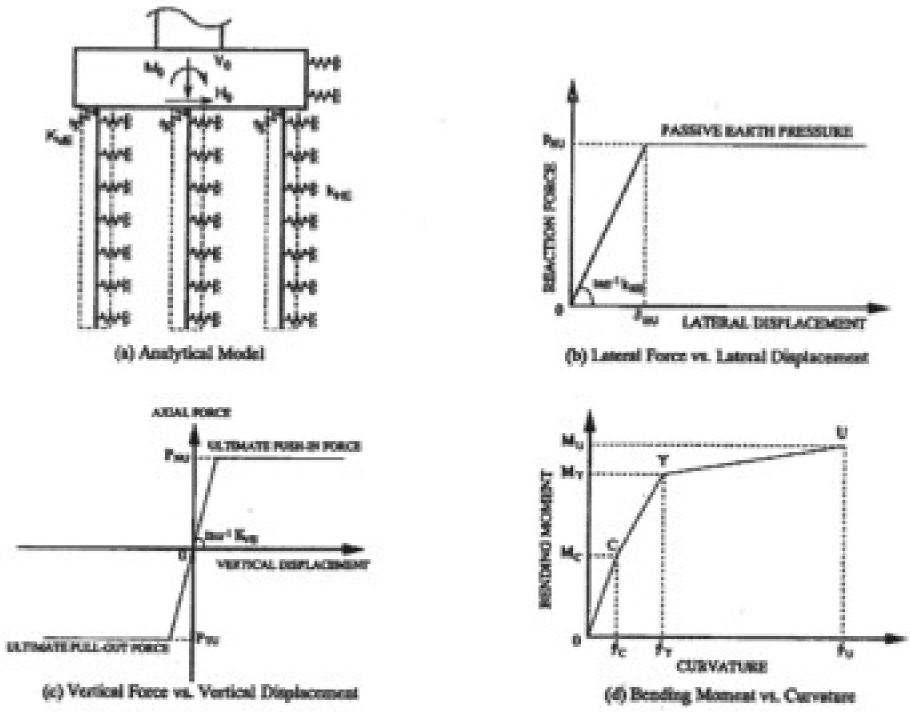


Figure 4: JRA analytical model and load deformation relationships for liquefaction analysis of a pier pile foundation

lateral pressures applied by the underlying liquefied layer are assumed to be equal to 30% of the overburden effective stress. Strictly speaking, force-based methods should be discouraged from use since they quite often produce incompatible soil loads and pile displacements.

TIME HISTORY ANALYSIS

The dynamic response of the bridge is highly nonlinear and is affected by the initiation of shaking, build-up of excess pore pressures, consequent reduction in stiffness and strength of soils, development of liquefaction and ground deformation associated with lateral spreading. A non-linear time history analysis allows to investigate the dynamic response of the bridge while accounting for the complex soil-pile-pier-abutment-deck interaction or response of the bridge system in liquefying and laterally spreading soils. There is no doubt that a sound numerical analysis that is well calibrated and executed provides the most realistic simulation of the actual bridge behaviour. Two types of such analysis are commonly carried out:

- Effective stress analysis (ESA) which permits evaluation of seismic soil-pile interaction while considering the effect of excess pore pressure and eventual soil liquefaction on the pile response

- Total stress dynamic analysis that has the same attributes as ESA but cannot simulate effects of the excess pore pressures and liquefaction with the same level of accuracy as ESA.

SUBSTRUCTURE ANALYSIS

MCEER/ATC-49-1 report (MCEER, 2003; Ledzema & Bray, 2010) recommends the following design procedure:

- Identify the soil layers that are likely to liquefy
- Assign undrained residual strengths to the layers that will liquefy
- Perform pseudo static seismic stability analysis to calculate the yield coefficient ky for the critical potential sliding mass
- Estimate the maximum lateral ground displacement
- If the assessment indicates that movement of the foundation is likely to occur in concert with the soil, then the structure should be evaluated for adequacy at the maximum expected displacement. This is the mechanism illustrated in Fig. 5
- The structural remediation alternative makes use of the pinning action of the piles
- Identify the plastic mechanism in the pile that is likely to develop as the ground displaces laterally
- From an analysis of the pile response to a liquefaction induced ground displacement field, the likely shear resistance of the foundation is estimated
- This increased resistance is then incorporated into the stability analysis, which increases ky. If appropriate, this lateral shear resistance should be reduced to account for PΔ effects
- Recalculate the overall system displacement on the basis of the revised resistance levels and iterate until the resistance is consistent with the level of displacement estimated. Once a realistic displacement is calculated, the system can be assessed for this level of movement
- If necessary, additional piles can be installed or ground improvement carried out to reduce the seismic displacement further

CONCLUSION

An extensive review of the design methods for bridges located on sites prone to liquefaction and lateral spreading has been carried out. There is no consensus amongst the geotechnical engineering community on the preferred design methodology. Therefore, geotechnical engineers should choose one of the described design methods based on the consideration of the importance level of the bridge, client's requirements and available time and project budget. The design is highly dependent not only on the design methodology for the bridge, but also on

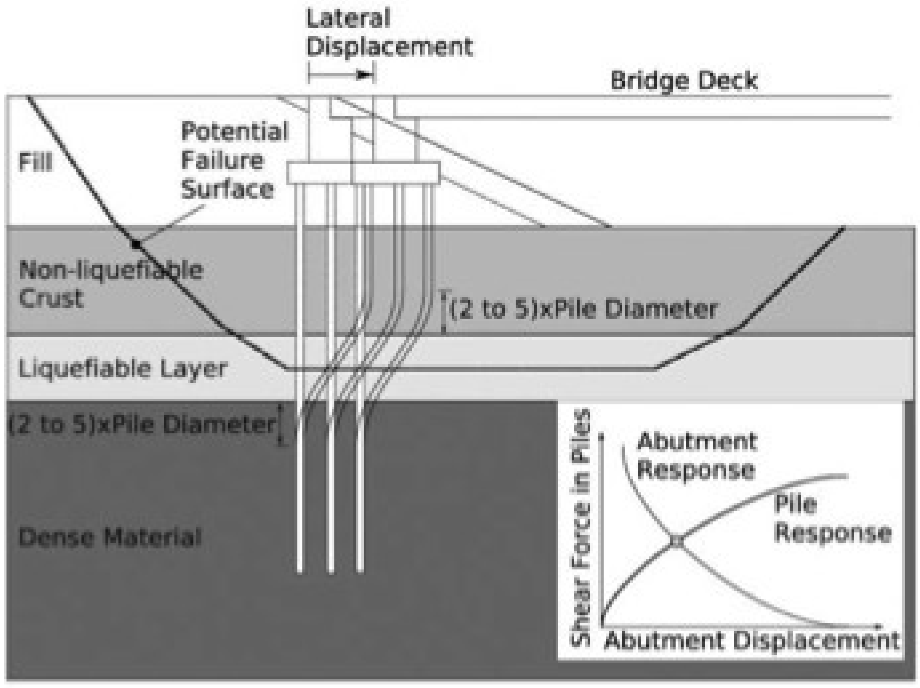


Figure 5: "Pile-pinning" effect for the case of a pile that is locked into both the soils above and below the liquefied soil layer

the quality of geotechnical investigations, assessment of seismic demand and triggering of liquefaction. It is therefore recommended that a number of different methods are used and sensitivity analyses are carried out at each stage of the design process. Irrespective of the adopted design method, it is always important to address uncertainties associated with the assessment of liquefaction potential of soils and the magnitude of lateral spreading, and evaluate the effect these uncertainties have on the predicted response / performance of the bridge structures. Additional work is currently being carried out to summarise the findings in a form of a technical document that will be later incorporated into the NZ Transport Agency's Bridge Manual and disseminated to the wider New Zealand engineering community.

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The New Zealand Soil Testing Standard (NZS4402) – Do We Need it? – Laurie Wesley and Michael Pender

READERS MAY BE aware of recent publicity regarding the New Zealand soil testing standard, namely that it is likely to come up for review by CETANZ in the not too distant future. The purpose of this article is to address two issues in relation to NZS4402. The first is: Does New Zealand require a standard of its own? And the second is: Does it need upgrading? The answer we would give to both is a definite yes.

WHY ARE NEW ZEALAND CONDITIONS SPECIAL?

The main justification for a New Zealand standard is that we have soils in New Zealand with unusual properties that are not adequately covered by the standards of other countries, in particular those of England and America. Large areas of Taranaki, the Bay of Plenty, and the North Island central plateau are covered in clay derived from the weathering of volcanic ash layers, and generally contain a large proportion of the clay mineral allophane. This gives the clay unusual properties, especially very high water contents, and a dramatic change in properties when dried. With very high allophane content the clay may lose much of its plasticity by air drying alone and become completely non-plastic after oven drying.

The group who drafted the NZS4402 Standard went to some trouble to specify procedures for performing Atterberg Limit tests that did not involve cycles of wetting and drying of the soil under test. Similar comments, related to testing for compaction curves, are discussed below.

COMPACTION CURVES

A dramatic illustration of the influence of drying is shown in Figure 1. This shows the results of standard Proctor

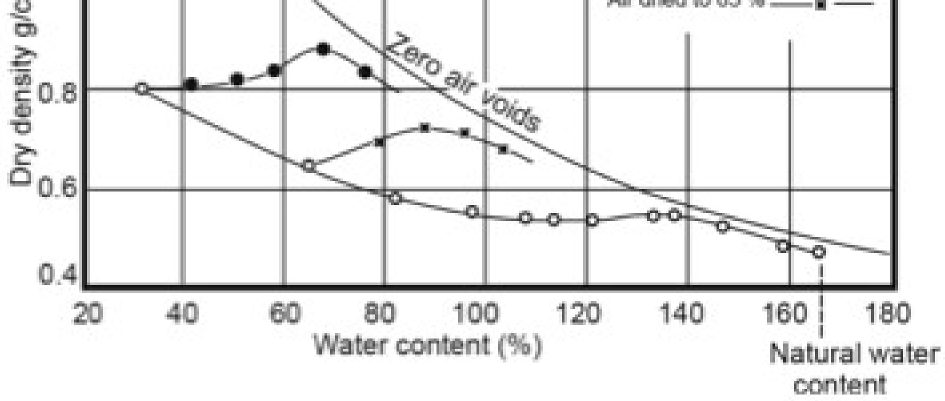

Figure 1: Standard Proctor compaction tests on an allophane clay, following a range of drying procedures from the natural water content of 167%.

compaction tests following varying degrees of drying. This figure is also Figure 44.4 of Tezaghi, Peck, and Mesri, 1996, quoting an earlier Wesley paper.

The curve of relevance to field control is the "natural" graph, where the soil has simply been progressively dried from its natural water content. The other curves are for re-wetting after varying degrees of drying, demonstrating the very large change in results caused by this re-wetting cycle. Each test in the figure was on a fresh sample of the soil. If the same sample of soil is used repeatedly then the curves will be somewhat different due to the increased degree of remoulding that occurs each time the soil is compacted. This test was on Indonesian volcanic clay with a very high allophane content. The allophane content in New Zealand soils may be less and the influence of drying less dramatic but still very significant. Following "conventional" Proctor testing has led to numerous difficulties in earthworks operations, and especially disputes between contractors and geotechnical consultants.

For good reasons therefore the present New Zealand standard does not allow drying before testing (unless there are special circumstances), and this should definitely be retained. Regarding the second question above, there are very good reasons why the standard should be upgraded. The first is that in addition to volcanic soils, New Zealand has a predominance of residual soils of many types, especially in the North Island. British and American standards have been put together by people or organisations with no experience of residual soils, and their testing standards reflect this. Apart from issues associated with volcanic soils a characteristic property of all residual soils is their variability. Specific changes that should be considered in any upgrade of the standard are the following

OEDOMETER TESTING:

(a) The compression graph should be presented using a linier scale alongside the traditional log scale. This might well seem "revolutionary" in the eyes of most of the geotechnical world, but if that is the case it shows how entrenched faulty standards have become and simply reinforces the need for a more enlightened standard. Most readers will be familiar with this issue, for any who are not Figure 2 is included. This shows oedometer tests results on Auckland (Waitemata) clay; the graph on the left with a log scale and on the right with a linear scale. The different picture given by the two plots is startling.

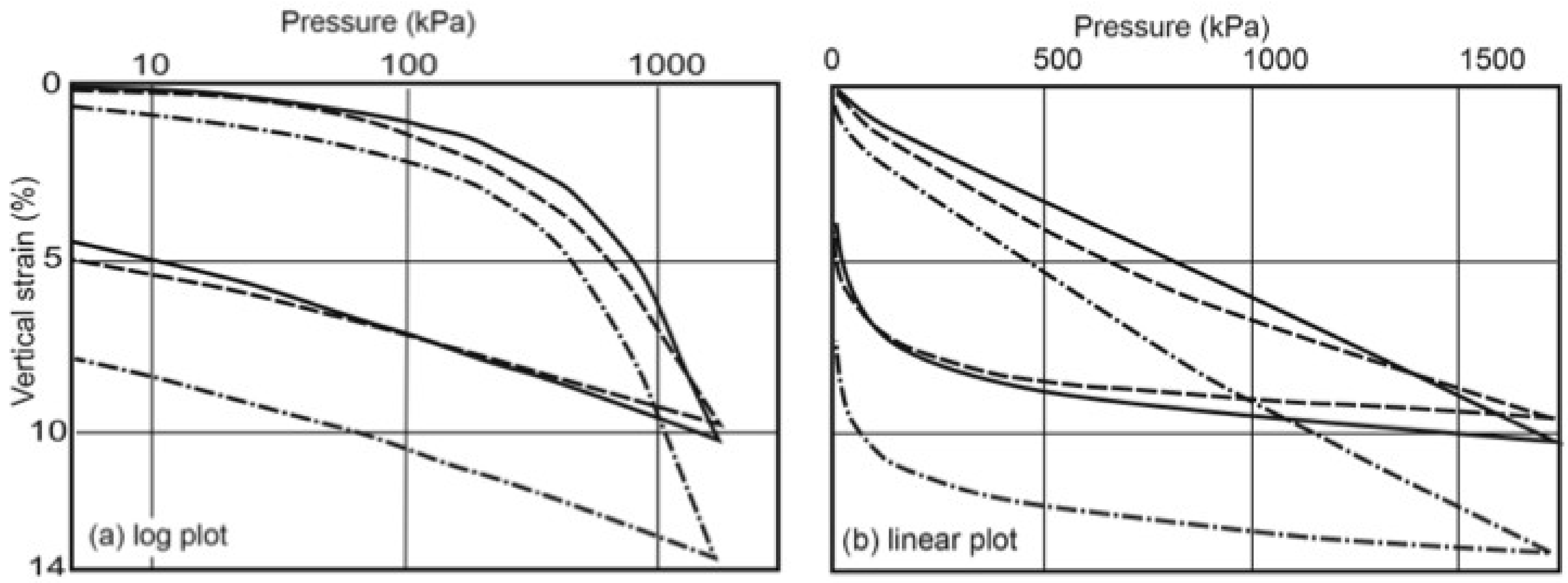

Figure 2: Oedometer tests on Auckland (Waitemata) clay using log and linear scales.

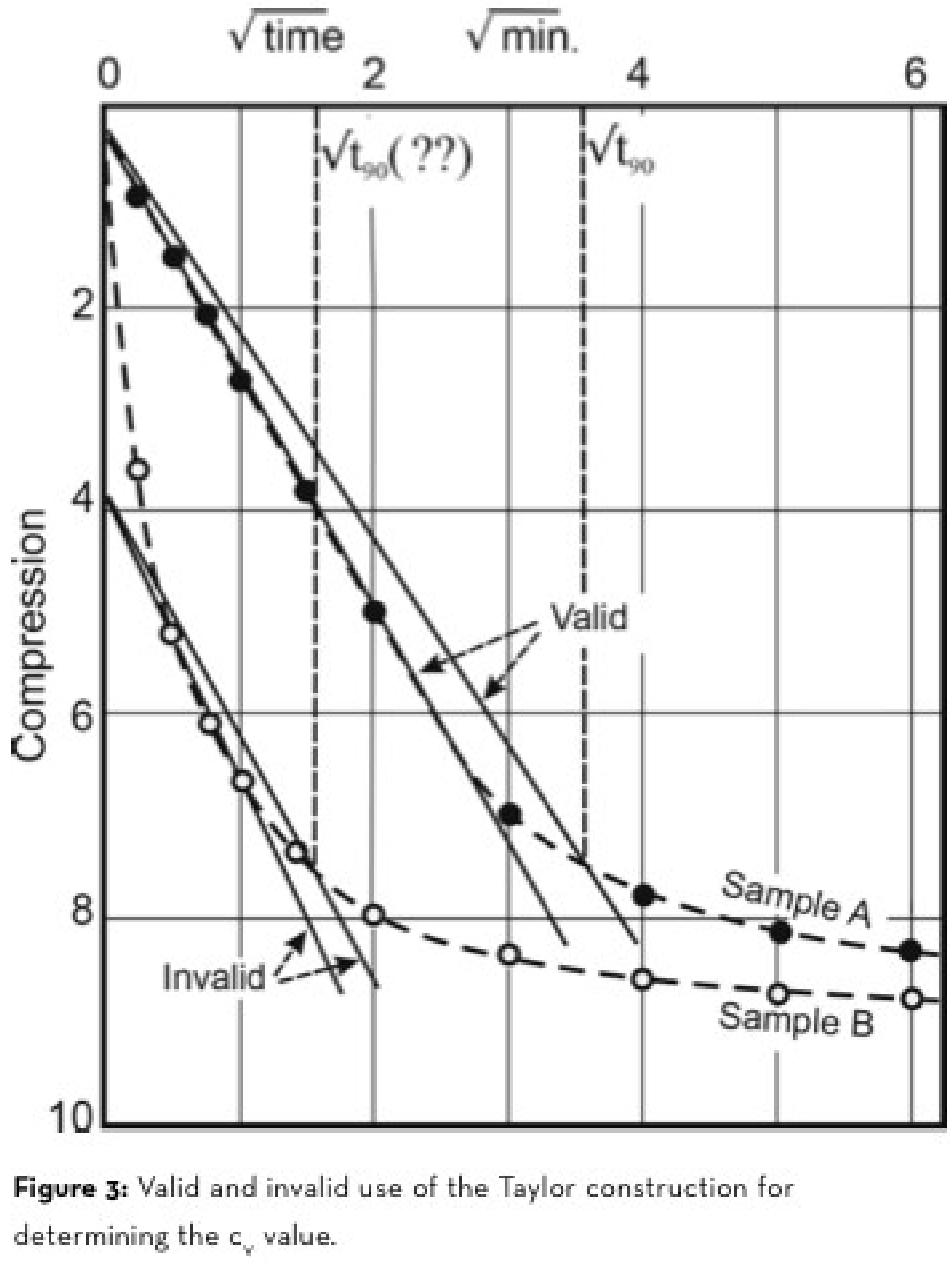

Figure 3: Valid and invalid use of the Taylor construction for determining the c_v value.

(b) The coefficient of consolidation of many residual soils is so high that its value cannot be determined from conventional oedometer tests. The pore pressure dissipates so rapidly that accurate measurements of its dissipation rate are not possible. Because of this, plotting results using the Taylor root time method does not produce a straight line. The shape of the graph is thus no longer governed by the rate of pore pressure dissipation, and the application of concepts based on the Terzaghi 1-D consolidation theory is no longer valid. This point does not appear to be recognised by the geotechnical fraternity or covered in any testing standards. The Taylor construction continues to be blindly applied and may well produce values of c_v that are much lower than the true value. Figure 3 illustrates this point. Sample A shows a valid straight line, but Sample B shows a smooth curve. In the case of Sample B the line "fitted" to the curve is arbitrary and at the whim of whoever does the test. The highest value of the coefficient of consolidation is about 0.1 m^2/day. If the Taylor graph does not produce a straight line then the test interpretation should be that the c_v value is greater than 0.1 m^2/day. Many residual soils have c_v values higher than the above limit.

TRIAXIAL TESTING

Some laboratories are still presenting the results of consolidated undrained triaxial tests in terms of both total stress and effective stress. A graph in terms of total stress provides no useful information. The c and ϕ values obtained from such a graph are not soil properties, only a reflection of the test procedure. It is surprising that total stress plots from consolidated undrained tests are still being produced; they should have disappeared about half a century ago.

COMPACTION CONTROL TESTING

The variability of residual soils makes compaction control using conventional Proctor testing very difficult and New Zealand has pioneered a very useful alternative method of control, using shear strength and air voids. This method is widely used in New Zealand, especially in the Auckland area, and most readers will be familiar with it. New Zealand consultants have also used it for projects undertaken overseas, especially in Malaysia and Indonesia. The method is slowly gaining attention overseas among those involved with residual soils, especially those of volcanic origin. It would be appropriate for New Zealand to have a "standard" for this method, though NZS4402 may or may not be the appropriate place for it since it is a method of controlling compaction quality rather than a laboratory test method.

TWO ADDITIONAL COMMENTS

(1) Some New Zealand laboratories may be following overseas testing standards in the belief that these are more up to date and superior to our own. This may well be a misguided belief, partly because of the points made above but also because overseas standards, especially those of America and England tend to be very "entrenched" and not easy to change. Those who produced these standards are often reluctant to concede that changes are necessary.

(2) There appears to be a widening gap worldwide between geotechnical engineers and geotechnical testing laboratories. This is a regrettable trend and may well result in poorer quality testing, and geotechnical engineers less able to evaluate the nature of the soil they are dealing with, or the reliability of the parameters they are using for design. There are various possible explanations for this trend. One is that consulting companies have found it difficult economically to maintain laboratory facilities of their own, and so their staff no longer have easy access to testing laboratories. Another may be that soil testing work is often seems to be regarded as a commodity item, all that is needed are the numbers and all will be well! On the contrary the purpose of soil testing is to provide understanding of the behavior of soil as an engineering material, this requires not only data from carefully and correctly performed tests but also thoughtful reflection on the significance of the results. Yet another explanation is that geotechnical engineers have become seduced by their computers into believing that sophisticated calculations are of more value than a sound understanding of testing procedures and direct observance of the soil itself.

CONCLUSION

We think that is it vital that the NZS4402 standard is retained but also that some enhancements are incorporated.

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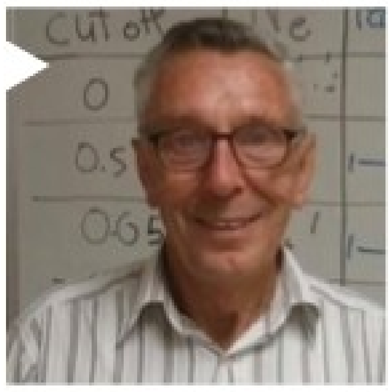
His primary areas of interest in teaching, research, and consulting are: the elucidation of the geotechnical properties of NZ materials, particularly soils of residual or volcanic origin; the earthquake resistant design of foundations; limit state design in geotechnical engineering; the engineering behavior of closely jointed rock masses.



Laurie Wesley

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

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HVSR & seismic zonation: a case study based on current NZS 1170.5 subsoil classification methods

– G. Mazzoni, A. Giannakogiorgos, M. F. Haryono, D. Sullivan

ABSTRACT

The Horizontal-to-Vertical Spectral Ratio method (HVSR) is a geophysical method based on single station ambient noise recording. Such method, first introduced in the 1970s, is currently considered to be a valid tool for seismic zonation due to its versatility and robustness. With reference to NZS 1170.5 site subsoil classifi-cation methods, HVSR can be proficiently used to enhance current engineering practice in New Zealand re-lated with seismic input definition and seismic hazard mapping.

1. INTRODUCTION

In this paper we present a case study focused on the potential use of Horizontal-to-Vertical Spectral Ratio method (HVSR or H/V) to complement the current practice in terms of seismic site classification and seismic input motion definition based on NZS 1170.5.

In May 2014 several HVSR tests were performed in the Christchurch's metro area and the results of such tests, with particular emphasis on site perods, are presented.

2. THE HVSR METHOD

The HVSR method (figure 1) consists of first recording several minutes of 3-component ambient noise vibrations, and performing a Fourier transform on each component. The ratio between the Fourier amplitude spectra of the horizontal to vertical components is then calculated. The method was first introduced by Nogoshi and Igarashi (1971) but it was later modified and developed by Yutaka Nakamura as illustrated by his paper [Nakamura, 1989].

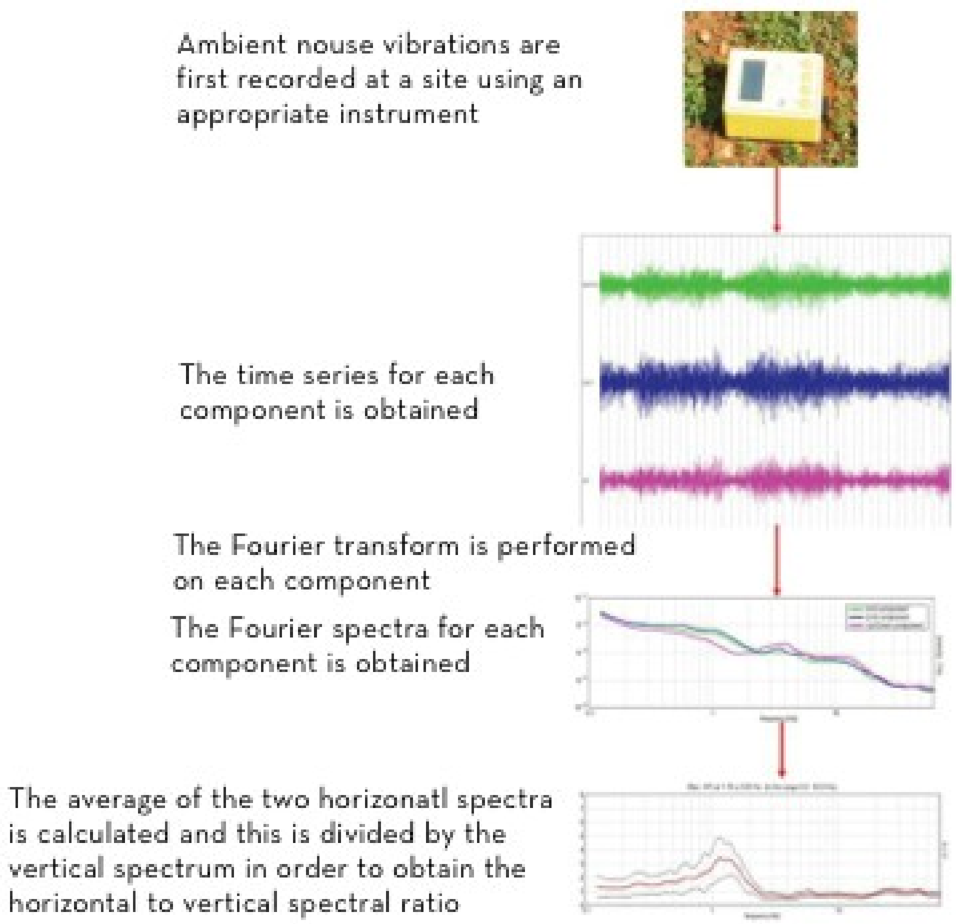
Shallow soft soils may amplify seismic motion at a site. Modifications of incoming earthquake wave characteristics result when these pass through soil deposits. Site effects are therefore dependent on the medium itself and on the stratum boundary depths or thickness of soil above the bedrock. In addition, site effect seismic wave amplification may result from topographic and boundary effects due to irregular surface geometry which focuses any incoming seismic energy at certain points leading to amplification [Bouchon, 1996].

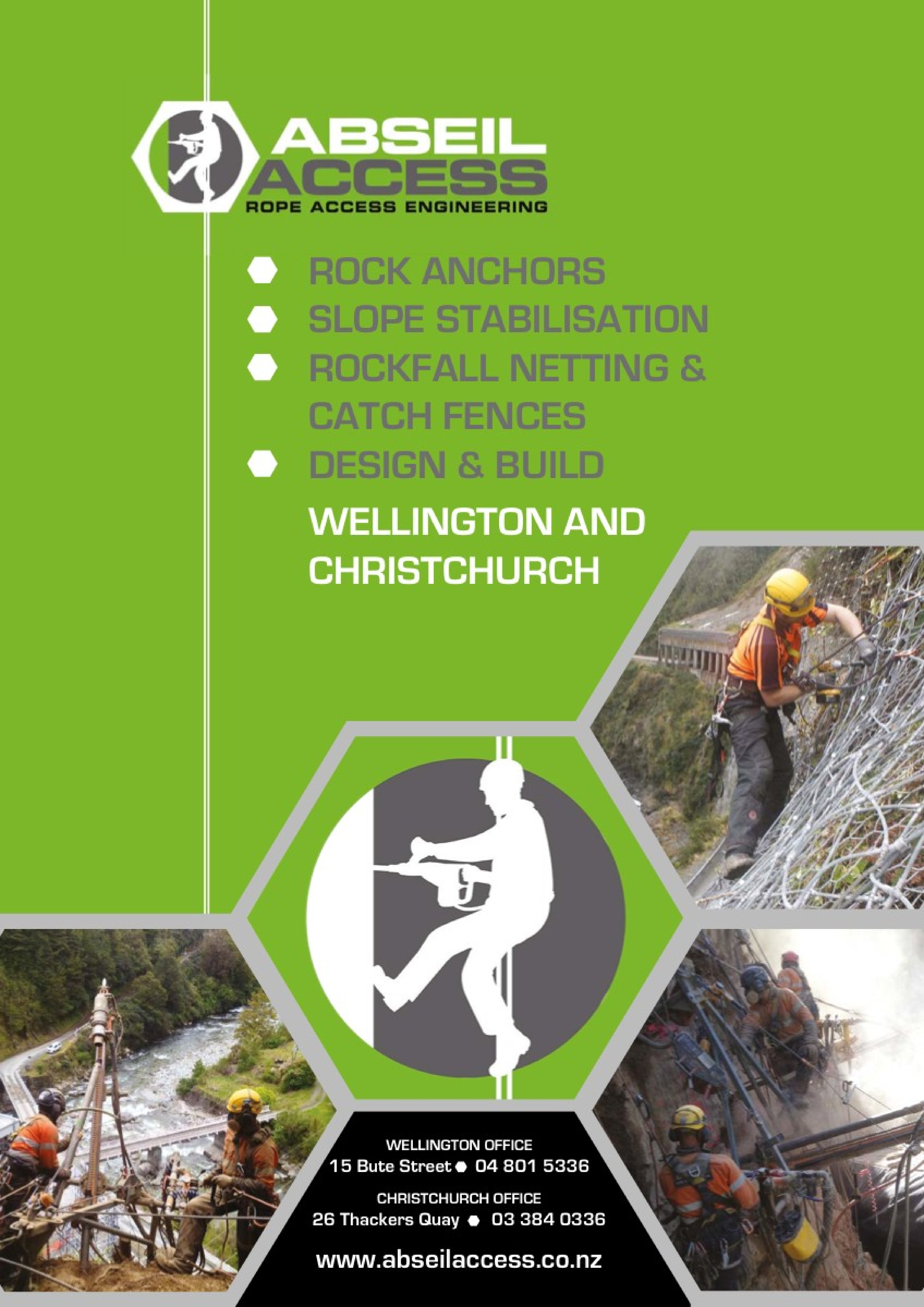
Even though sites often involve both irregular geometry and heterogeneous soil properties, topographic-induced effect on amplification is usually less than the one from heterogeneous soil properties [Chávez-García, 2007]. Amplification is a significant variable, even within small distances (few metres), therefore caution is required for more accurate seismic hazard mapping.

Andreas Giannakogiorgos

Andreas is a Geotechnical Engineering working for Coffey, with 15 years of professional experience mainly from Greece, and an MSc - DIC degree in Soil Mechanics and Engineering Seismology from Imperial College. He worked for major infrastructure projects in Greece including more than 300 design projects. He moved to New Zealand with his family in 2013, and enjoying a lifetime experience in an amazing part of the world.

Figure 1: General principles of HVSR testing

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HVSR technique is a fast and cost-effective alternative way to investigate for site effect amplifications through the use of ambient (low-frequency) seismic noise. The ground is never "at-rest" being constantly exposed to sources of rapid deformational energy resulting in triggering seismic waves [Lay and Wallace, 1995]. Ambient noise was previously regarded as a nuisance in seismograph design, but further studies have resulted in a bet-ter understanding of the noise source and charac-teristics. Following that, new applications for microtremors and different techniques making use of noise recordings, have been proposed [Bonnefoy-Claudet et al., 2006].

Ambient noise includes several noise types and triggering sources, both of artificial and natural origin and most importantly of near surface generation. This is why surface soil layers are exposed to ambient vibration of the ground caused by man-made or atmospheric disturbances, known as microseisms or microtremors.

Sources of microtremors are both natural such as sea waves, tides and atmospheric pressure changes, as well as anthropogenic (man-made) such as vehicles, industrial machinery and construction work.

Microseisms usually consist of long-period waves of periods longer than 2 seconds, and also being referred to "long-period microtremors". Microtremors usually denote higher frequency signals of short periods such as those caused by traffic, or functioning machinery. Noise generated at frequencies around 1Hz is usually due to wind effects and local meteorological conditions (atmospheric disturbances). Microtremors are normally considered to affect the uppermost sedimentary layers whilst microseisms affect layers deeper down [Atakan, 2007]. The ambient noise wave field is temporally and spatially variable, as well as being non-uniform at all frequencies. Islands such as New Zealand are constantly exposed to wave surf and oceanic standing waves and are therefore noisier than land sites distant from any coast. As well as hourly variations, the noise wave field exhibits daily and seasonal changes too, both as a result of human-induced variations in the case of microtremors and atmospheric variations such as changing atmospheric patterns in the case of microseisms.

Numerous studies have been undertaken to determine the capability and applicability of the HVSR technique, as well as its limitations [Di Giacomo et al., 2005]. Many agree about the fact that the original Nakamura's technique does not give reliable amplification values. Generally the amplification values obtained directly from the HVSR spectrum (as per Nakamura's proposed method) vary from those found using other site-response estimation techniques, hence they can only provide rough amplification level estimates in the case of relatively simple geological conditions [Lermo and Chavez-Garcia, 1994]. Although the latter represent a commonly accepted limitation, the HVSR technique allows for a reliable estima-tion of the fundamental resonance frequency (f_0) of soft deposits and, in most cases, for the robust estimation of S-wave versus depth profile.

Applying the HVSR technique is simple and it may also be performed on a wider range of sites than other techniques such as array methods. For example, seismic refraction is problematic when applied in agricultural fields, marsh areas, areas of dense vegetation and built urban environments, whereas the HVSR method is not. If explosives are required as a seismic source, there are further restrictions as well as possible damages and inconveniences incurred for underground communi-cations and drainage systems.

This is not the case for HVSR technique that makes use of single station measurements by means of simple portable instrumentation. Moreover, there is no need for a "seismic source" such as sledge hammers, since only ambient noise is used which will always be available. A significant aspect of the HVSR method, something worth to highlight in comparison with other methods such

CLASS	DESCRIPTION	DEFINITION
A	Strong Rock	UCS>50MPa & Vs30>1500m/s & not underlain by < 18 MPa or Vs 600 m/s materials
B	Rock	1<UCS<50 MPa & Vs30>360m/s & not underlain by <0.8 MPa or Vs 300m/s materials, a surface layer no more than 3 m depth (HW-CW rock/soil).
C	Shallow Soil	not class A,B or E, low amplitude natural peroid ≤ 0.6s, or depths of soils not exceeding those in Table2.
D	Deep or Soft Soil	not class A,B or E, low amplitude natural period >0.6s, or depths of soils exceeding thse in Table 2, or underlain by <10 m sols with underained shear strength <12.5 KPa, or <10m solis SPT N<6.
E	Very Soft Soil	>10m soils with undrained shear strength <12.5 KPa, or >10m soil with SPT N <6, or >10m soils with Vs ≤ 150m/s, or >10m combined depth of previous properties

Table 1: NZS1170.5:2004 Site Subsoil Classes

as the refraction, is that whilst the latter has a limited penetration depth, the HVSR technique could be better and considerably reliable at reaching larger depths.

3. SITE CLASSIFICATION: THE CURRENT PRACTICE

The influence of local geologic (site) conditions on the intensity of ground shaking and earthquake damage is well documented.

The New Zealand loading standard (NZS 1170.5:2004), takes into consideration site conditions, largely through prescribing structural design actions on the basis of site subsoil class to accommodate likely increased earthquake loadings due to shaking modification.

As in many other national and international standards, NZS 1170.5:2004 sets out five site subsoil class categories (Table 1), based on geological and geotechnical properties, which must be used in the calculation of horizontal and vertical loading.

The unrefined classification provides for subsoil classes ranging from sound rock (class A) to very soft soil (class E) to define spectral amplification of seismic acceleration.

The seismic site classification according to NZS 1170.5 is based on three fundamental parameters:

a) Shear wave velocity (Vs)
b) Site period (T_0)
c) Impedance contrasts (Z)

The introduction of site period and impedance contrasts in subsoil class definition represents a major progress in simplified seismic zonation (with reference to international codes, such as the EuroCodes) and it is in line with the most recent attitudes in seismic risk assessment procedures based on a multi-parametric approach [Castellaro, 2011].

The current standard practice in New Zealand involves the use of seismic-mechanical tests (such as the seismic Cone Penetration Test - sCPT) and general geological knowledge to determine the subsoil class, even though the HVSR method (aka Nakamura's method) is included into the list of suggested methods for seismic site classification in NZS 1170.4.

Evidently the limits of such approaches (e.g. depth limit of sCPT) imply the necessity of precautionary assumptions, especially when dealing with soft soil classes.

Moreover this procedure does not allow any direct estimation of site fundamental periods rather than via a limited simplified procedure (two-layer model) that cannot be applied in deep alluvial formations.

All these factors inevitably lead to a potential variation (under or over estimation) of amplification parameters that might lead to an unjustified increase in construction costs (overestimation) or to an increased risk (underestimation).

4. THE ROLE OF HVSR IN SITE CLASSIFICATION

To overcome such partial lack of information, the HVSR method can be used as a primary integrative tool for seismic micro-zonation. Such technique will allow, in most cases, for a direct estimation of site fundamental frequencies (hence site period) as well as S-wave velocity profiling (Vs) via suitable wavefield inversion procedures [Lunedei & Albarello, 2010].

The main advantages of the HVSR method are mostly in terms of the cost-effectiveness and versatility. Compared to other state-of-the-art methods for indirect seismic parameterization, including 1D and 2D surface wave methods and body waves tomography, the HVSR method has far fewer limitations in terms of usability (depth target, velocity inversions, etc.) and procedural costs.

Current engineering practice, especially in Europe and Japan, the HVSR method is proficiently used in seismic zonation in conjunction with mechanical tests and/or other geophysical methods, towards a robust seismic parameterization, both for simplified classification and for more advanced wave propagations analysis and ground motion predictions.

Following, a case study based on the use of HVSR technique for simplified seismic zonation according to NZS 1170.5 in several spots within Christchurch metro area is presented.

5. THE CHRISTCHURCH'S CASE STUDY

In May 2014, several HVSR tests around Christchurch's metro area have been performed, and with the use of existing geotechnical information (e.g. borelogs and CPTs data) to optimising the HVSR inversion procedures and proceed with a quick seismic subsoil classification based on NZS 1170.5.

Two case studies are reported, the first in Hagley Park and the second within in a private property in Hillsborough.

At Hagley Park we have used two MASW tests and three borelogs (Canterbury Geotechnical Da-tabase) to constrain the inversion problem of five HVSR tests carried out in a 3-hour span during daytime hours.

AT Hillsborough we have used one CPT and one borelog (Coffey's database) to constrain an equal number of HVSR tests.

The results of such case studies are presented below with reference to the mentioned simplified subsoil classification method, together with some graphical examples of test outputs and inversion procedures.

It must be noticed that any physical restraint (e.g. borelog) is strictly needed only to the first major contrast of seismic impedance, hence, for a future perspective,

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it is not necessary to actually reach great depths with mechanical tests to provide a sound seismic classification. Moreover autocorrelation with MASW proved to be a good alternative solution to mechanical tests' constraining, at least where local conditions are suitable for line array tests (e.g. Hagley Park site).

5.1 Hagley Park (HAG) test site

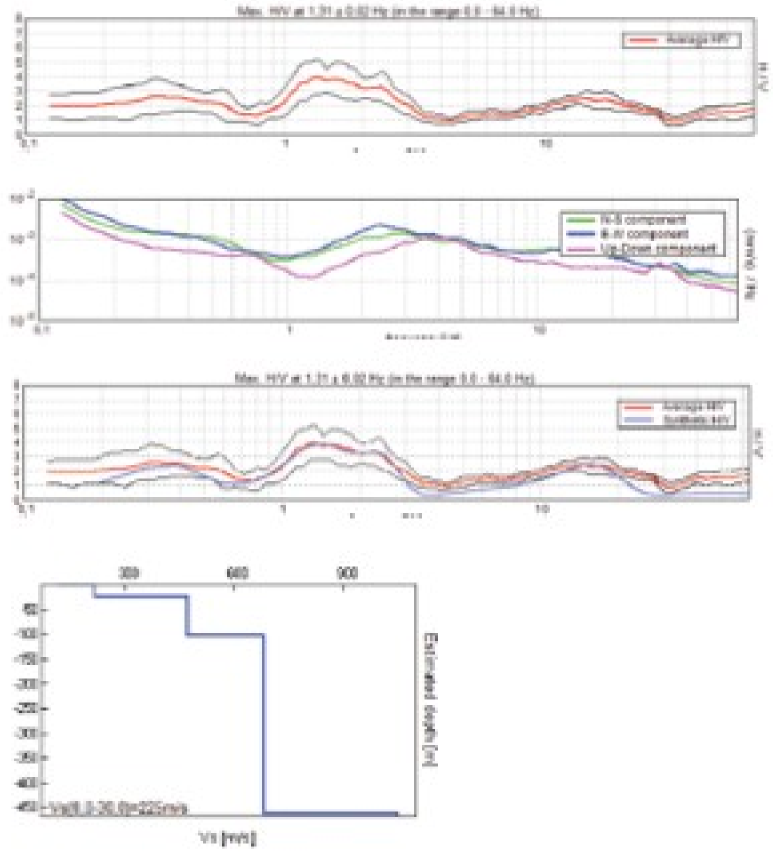

Figure 2: HVSR results and Vs30 profile

Test No.	V_{s30}	T_o	Reference test
HAG 1	225 m/s	0.76 s (0.46 s)	MASW
HAG 2	221 m/s	0.46 s (0.77 s)	MASW
HAG 3	201 m/s	0.82 s (0.53 s)	Borehole
HAG 4	221 m/s	0.76 s (0.46 s)	Borehole
HAG 5	214 m/s	0.48 s (0.77 s)	Borehole

Table1: Hagley Park (HAG) results

The period value in brackets () corresponds to the second major peak of HVSR curve. In the case of Hagley Park site such peak is very close to the dominant one, since the whole site presents 2 major peaks in the H/V spectrum (very close to the 0.6 s threshold).

This suggests that, although the site may be often classified as Class D based on V_{s30} and T_o values, a more accurate analysis based on HVSR results (e.g. 1D propagation model) may lead to amplification factors closer to a Class C soil classification.

By examining the HVSR curve, the T_o value can be estimated directly (max. H/V).

The inversion procedure has been carried out through the optimization procedure based on surface waves dominance assumption (synthetic H/V below 1.0 Hz is merely indicative). Such procedure allows estimating the V_{s30} parameter.

5.2 HILLSBOROUGH (HIL) TEST SITE

Test No.	V_{s30}	T_o	Reference test
HIL 1	260 m/s	0.33 s	CPT
HIL 2	(309 m/s)	(0.27 s)	Borehole

Table2: Hillsborough (HIL) results

The site in Hillsborough is generally classified as Class C, even though the site stratigraphic amplification reaches Class B level for the uphill zone (test HIL 2).

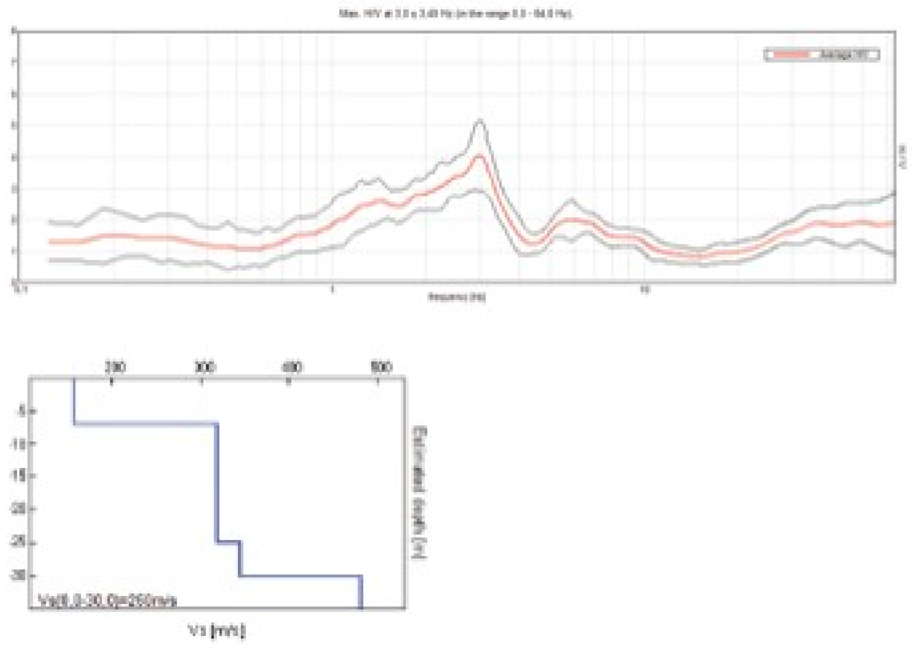

Figure 3: HVSR results and Vs30 profile for HIL1

The V_{s30} and T_o value for the uphill tests are reported in brackets () since the scattering between N-S and E-W spectral components of ambient noise suggests caution while evaluating such parameters with standard procedures from HVSR.

The limitation of HVSR method as direct estimator of To in sharply non-1D conditions is stated in many literature references and in NZS 1170.5 as well. Again, the T_o value can be estimated directly (max. H/V on test HIL 1).

6. CONCLUSIONS

The HVSR method is a valid alternative to more invasive and expensive testing procedures (eg sCPT or Down Hole) for determining the V_{s30} pa-rameter and profile. Moreover, site periods can be easily measured with the HVSR method, eliminating the need for further modelling and/or simplified precautionary assumptions.

As per NZS1170.5, the necessary parameters for seismic site classification can be easily retrieved from simple single station ambient noise recording (HVSR), provided there is an understanding of the ground profile to be used in the constraining procedure for HVSR – particularly significant for the city of Christchurch since the amount of available geotechnical data through CGD allows for a robust and virtually inexpensive constraining procedure for HVSR.

Finally, the HVSR method, due to its cost-effectiveness, can be widely used to help towards seismic subsoil classification, aiming to avoid misestimating seismic amplification factors that could undoubtedly lead to under or over designing engineering structures.

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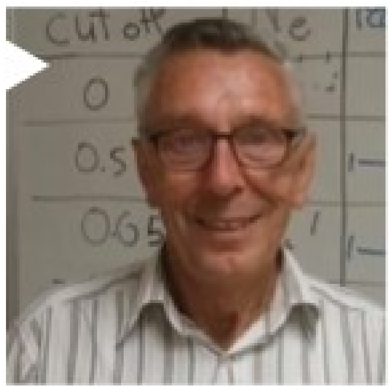
Undrained and Drained: Behaviour and Terminology



Laurie Wesley

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THE TERMS "UNDRAINED" and "drained" are not always correctly used, especially in relation to stability analysis of slopes. In particular, such analysis is often referred to as either an undrained analysis or a drained analysis. The purpose of this note is to try to bring some clarity to what the terms mean and how they should be used. My comments apply to fully saturated clays. This document is not intended to be a technical paper. Rather it is a "clarification" or a discussion article, and responses are encouraged.

UNDRAINED AND DRAINED

An undrained situation, or undrained behaviour, is when no water flows into or out of the soil. The soil water content and the volume both remain constant. Undrained behaviour occurs for two reasons:

1. The soil is enclosed within an impermeable barrier, as is the case in an undrained triaxial test.
2. There is insufficient time for water to drain out of, or into, the soil, as is the case when a foundation load is applied, (or an excavation made) rapidly to a clay of low permeability.

A drained situation (or drained behaviour) is the reverse. Water is free to flow into or out of the soil. However, as will be evident from the second condition above, we cannot be sure drained behaviour occurs simply because there are no physical barriers to flow. There must also be sufficient time for water to drain into or out of the soil. When there is sufficient time to allow unrestrained seepage flow then the term "fully drained" is appropriate. Fully drained should not be taken to mean there are no pore pressures present, only that water can freely flow into or out of the soil.

In many practical situations the behaviour during application (or reduction) of a load will be somewhere between undrained and drained.

STABILITY ANALYSIS USING TOTAL STRESS AND EFFECTIVE STRESS

As we are all well aware, we have the choice of carrying out stability analysis either using total stresses or effective stresses. A total stress analysis is based on the undrained shear strength, S_u, of the soil, and an effective stress analysis is based on the effective stress parameters, c' and ϕ'. The total stress analysis can correctly be referred to as an *undrained analysis*, and the effective stress analysis seems often to be referred to as a *drained analysis*. This might seem logical at first sight, but a little thought shows that it is not at all logical and is a misleading term.

When we do a stability analysis using the effective stress parameters c' and ϕ' we simply determine the normal stress and the pore pressure on a possible failure plane and calculate the shear strength from the effective stress Mohr-Coulomb equation:

$$s = c' + (\sigma - u) \tan \phi'$$

We are therefore analysing a static (or pseudo-static) situation, and no assumption is made about whether any potential failure will be undrained or drained, or somewhere in between. Whether failure is drained or undrained depends on the factors causing the failure. If failure occurs as a result of steadily rising pore pressures (as for example during prolonged rainfall) then behaviour up to the point of failure is drained, but once failure is initiated there is no time for flow of water and the behaviour changes to undrained. During earthquake loading it is very unlikely that there will be time for water to drain into or out of the soil so it is probable that behaviour will be undrained

We can of course carry out an effective stress analysis of an undrained situation. We may wish to do this to estimate the stability an earth dam during an earthquake. This is not easily done as it involves estimating the change in pore pressure caused by the cyclic loading of the earthquake. For this reason it is easier to do the analysis in terms of total stress using the undrained shear strength, possibly making an allowance for some loss of strength caused by the earthquake.

The choice we have in carrying out an analysis therefore

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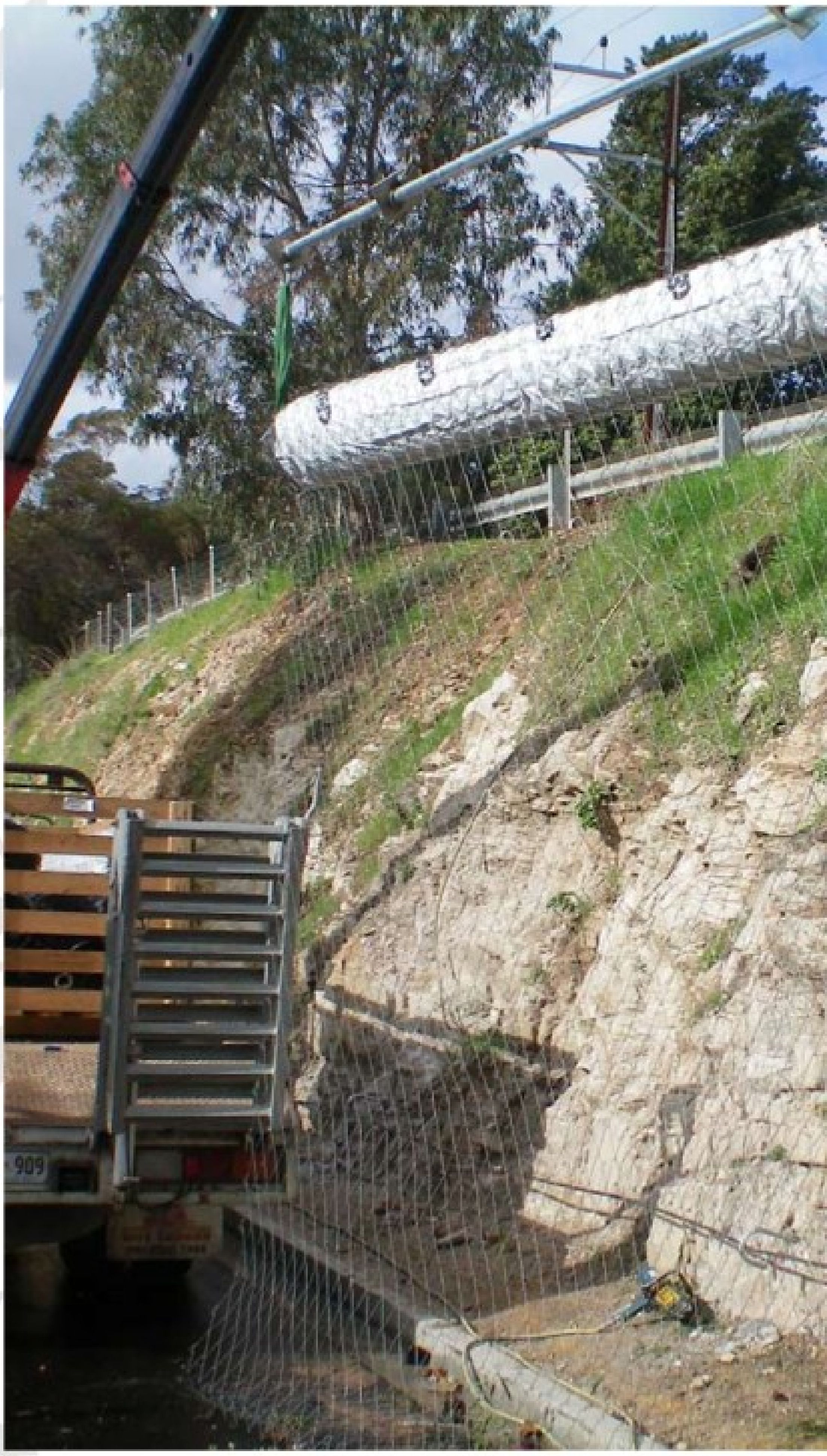

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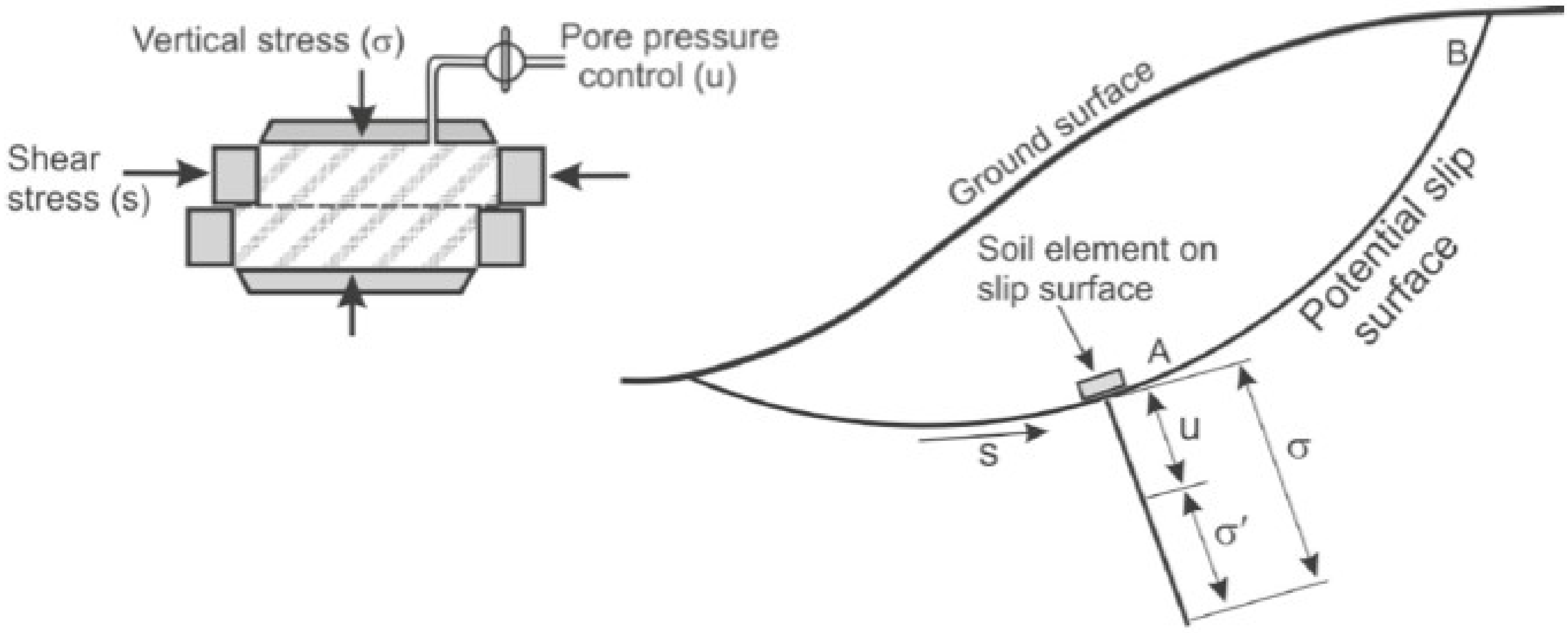

Figure 1: Measuring the undrained strength, S_u, and estimating the strength using the effective stress parameters c' and ϕ'.

is not between an undrained and a drained analysis. It is between:

(a) A total stress or undrained analysis, using the undrained shear strength.

(b) An effective stress analysis, using the parameters c ′ and ϕ ′.

SHOULD A TOTAL STRESS ANALYSIS AND AN EFFECTIVE STRESS ANALYSIS GIVE THE SAME RESULT?

I suspect this is a question that not many geotechnical engineers can give a clear answer to. I will outline several answers. Consider the soil element on the potential slip surface shown in Figure 1, and assume the effective strength parameters of the soil are c' and ϕ'. There is a total stress (σ) and a pore pressure (u) acting across the slip plane as indicated. To determine the available shear strength of this element we could take an undisturbed sample at the appropriate inclination and set it up in a theoretical shear box subject it to the exact stress state under which it exists in the field. This is a special shear box (so special that it hasn't actually been invented yet) as it makes possible both drained and undrained tests, and has a special connection to control or measure the pore pressure in the sample, as the figure shows.

If we carry out the test as a drained test with the vertical stress and pore pressure maintained exactly as they are in the field then we will measure the shear strength given by the equation:

$$s = c' + (\sigma - u) \tan \phi' \qquad 1$$

In the above test the pore pressure is connected to a constant pressure supply with value u.

We could also carry out the same test undrained, in which case the valve on the pore pressure line is closed or connected to a transducer to measure the pore pressure. When the horizontal force is applied the pore pressure will change, and the measured strength will be different. If we assume the pore pressure change is Δu, then the measured strength will be given by:

$$s = c' + (\sigma - u - \Delta u) \tan \phi' \qquad 2$$

The pore pressure change, Δu, could be positive or negative, so the strength could be greater or smaller than that according to equation 1.

This explanation may seem excessively complex or tedious but its purpose is to make clear that even with the most careful and idealistic sampling and testing, an undrained test will not normally give the same strength as that deduced from the stress state acting on the sample in the ground. There are two exceptions to this. The first is when the safety factor is unity, that is the soil is on the point of failure. Bishop made this point in relation to the stability of a vertical cut in his 1960 paper with Bjerrum. In this situation there is no additional shear stress to be applied, and thus no change in pore pressure. The second is the fortuitous situation when there is no change in pore pressure during shearing, or in other words the pore pressure parameter A is zero.

A second, and perhaps simpler, way of answering the above question is to point out that the undrained strength of a soil does not have a unique value. The test described above is highly idealised, assuming no disturbance of the sample occurs and it can be set up in the shear box in exactly its original in situ state. In practice, there is always disturbance during sampling and

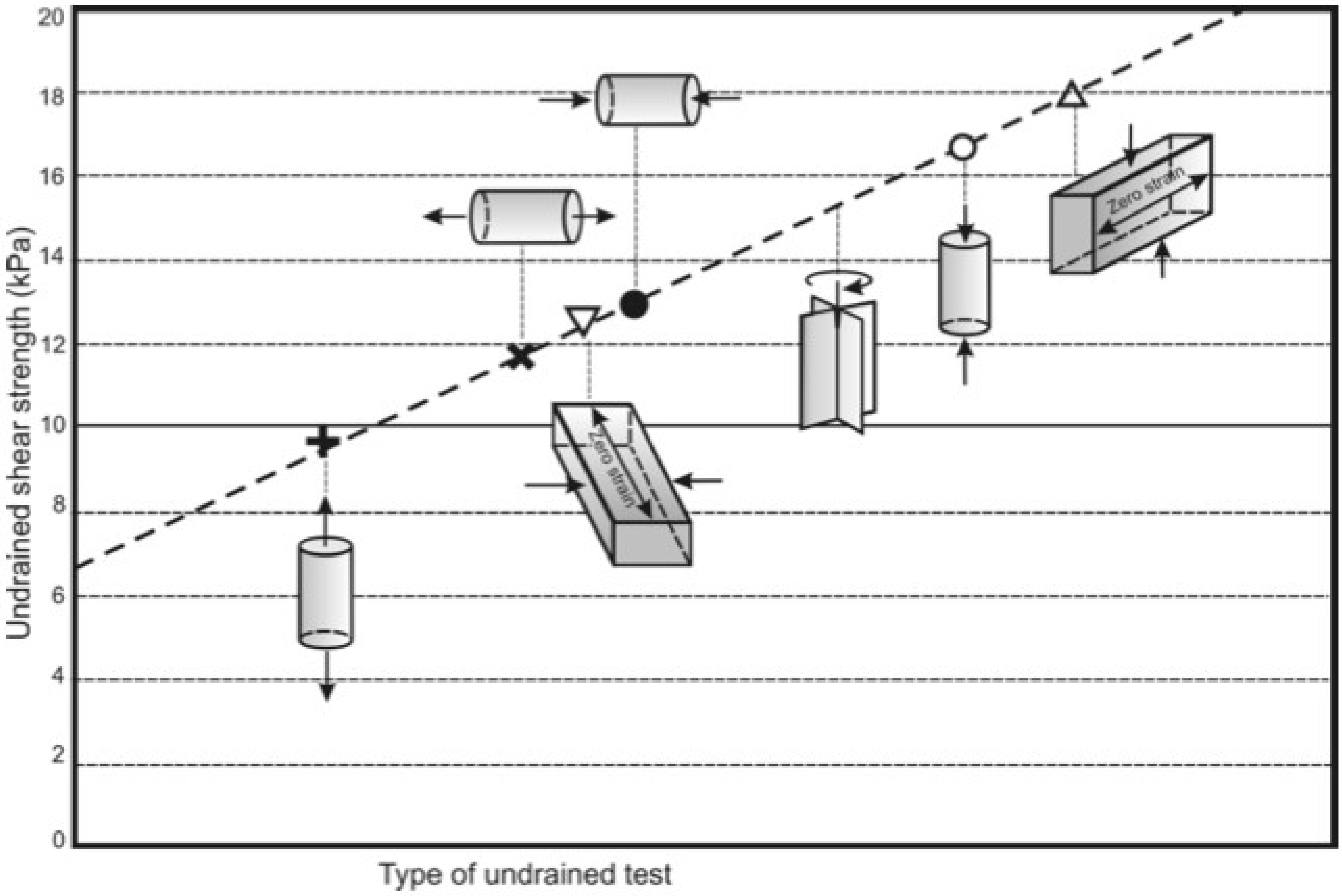

Figure 2: Undrained shear strength from a range of different types of test (from my PhD thesis).

setting up the sample in the test apparatus, so that the pore pressure at the start of the test is unlikely to be that in the ground. Normal practice with triaxial testing is to carry out compression tests on samples taken with vertical orientation. This is appropriate if the load to which the soil will be subject in practice is also vertical. However, there are numerous situations where this is not the case. The soil may be compressed horizontally, and thus fail in extension, as would be the case in the centre of a deep circular excavation. While triaxial tests are the standard method of measuring soil strength, many situations, especially soil slopes, are more likely to be plane strain than triaxial.

Figure 2 shows a range of values of undrained shear strength obtained from various tests on a soft normally consolidated clay, namely:

- triaxial tests in compression and extension on vertical samples
- triaxial test in compression and extension on horizontal samples
- plane strain tests in compression on both vertical and horizontal samples
- in situ vane tests

These laboratory tests were carried out on undistrubed samples set up under the same initial stress state as that in the field. Each test gives a different value of undrained shear strength because the pore pressure response during each test is different. The pore pressure when a sample fails in extension is higher than in a compression test, and the strength correspondingly lower. Also, soil stiffness in the horizontal direction is frequently less than in the vertical direction, resulting in higher pore pressure when subject to an increase in horizontal stress. For the above clay, the minimum value of undrained strength (9.5 kpa) is from the vertical triaxial extension test, while the maximum value (18kPa) is from the vertical plane strain compression test. In slope stability situations the action is likely to be plane strain, in which case the difference in values could be greater. Figure 2 does not contain extension plane strain tests, because a plane strain apparatus for such tests has yet to be invented; the design difficulty is preventing the soil moving away from the end platens that maintain the zero strain condition during compression tests. Note also that the above tests are from a soft clay. Tests on a firm or stiff residual clay may not show such large differences in the undrained strength.

The most important point here is that while the parameters c' and ϕ' can legitimately be regarded as unique for a particular soil, this is not the case with the undrained shear strength. The latter is variable depending on the method used for measuring it.

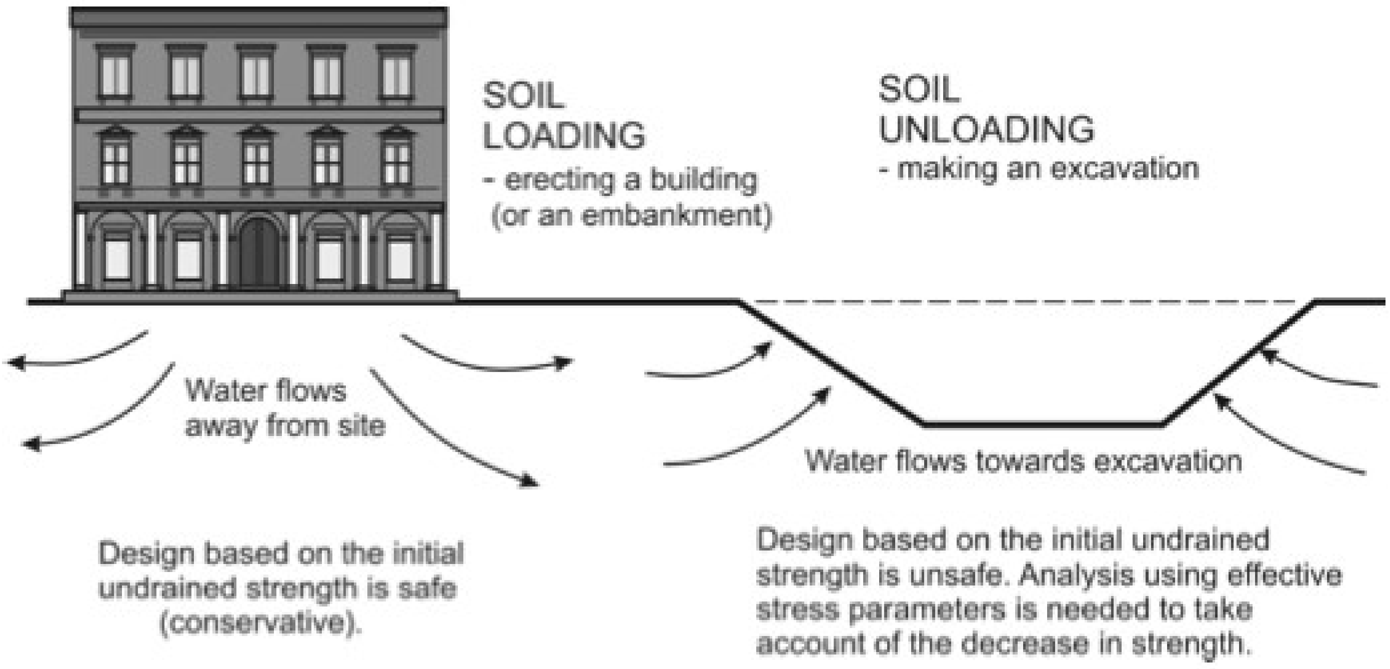

Figure 3: Influence of loading or unloading the soil on its long term strength.

USE OF TOTAL STRESS AND EFFECTIVE STRESS ANALYSIS IN PRACTICE

Engineering projects usually involve loading or unloading the soil, as indicated in Figure 3. The loading shown is a building but could also be an earth embankment. When load is applied the soil consolidates with time and its strength increases. The rate at which this increase occurs is generally uncertain. It is thus sound practice to base the estimate of bearing capacity for the design of the foundations on the initial strength of the soil, which is easily determined by measuring its undrained strength S_u.

On the other hand, if the soil is experiencing a decrease in stress, as in an excavation, water will be drawn towards it and the strength will decrease with time. In this situation, an estimate of bearing capacity based on the initial undrained strength would not be a safe procedure. The only realistic means of estimating the long term stability of the excavated slopes is by using an effective stress analysis.

There are other situations needing further consideration. For example, with natural hill slopes, only an effective stress analysis can be expected to give a reasonable answer, primarily because it is the only method that allows seepage pressures to be taken into account. There is a secondary reason which relates to the way the two methods assign the shear strength to the slip plane. Consider points A and B in Figure 1. In terms of effective stress the strength at point A will clearly be much greater than that at point B because of the higher normal stress on the slip plane. This difference in shear strength is unlikely to be reflected in a total stress analysis, partly because sufficiently detailed measurements of undrained strength would not normally be carried out, and partly because even if they were they would not reflect the same difference in strength that the effective stress analysis shows.

There are three further reasons for basing the design of shallow foundations on clay on the undrained shear strength. The first is that use of the normal bearing capacity equations and effective stress parameters c' and ϕ' can lead to unrealistically high values of bearing capacity. Some clays have ϕ' values in the 35° to 40° range in which case the bearing capacity factors become very large and the estimated bearing capacity is much greater than is practically feasible. The second is that the application of the foundation load will induce a change in the pore pressure state; this will not be uniform and cannot be estimated with much reliability. The third is that accurate measurement of the cohesion intercept c' in a soft clay is very difficult. Despite what has been said above, measurements of the undrained strength are more reliable.

A final comment: when short term and long term stability estimates are made they must be made using the same method of analysis in order to give a reliable picture of the change in safety factor with time. We cannot do a short term analysis using total stress and a long term analysis using effective stress and expect the difference to be a true measure of the change in safety factor. For example, when an embankment is built on soft clay, the "end of construction" safety factor would normally be based on the initial undrained shear strength. The long term stability could be estimated by using effective stresses and assuming all pore pressures had fully dissipated. This would not give a reliable estimate of the increase in safety factor due to consolidation of the soil. The only reliable estimate would be obtained by measuring the new undrained shear strength and repeating the total stress analysis. The increase in shear strength can be estimated using empirical relationships, but these are far from reliable and to be sure, new measurements of undrained shear strength are necessary.

Practical Approaches to Seismic Design of Deep Foundations

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THIS IS A **re-print of a paper selected as being of particular interest to NZGS members. It was originally presented as a keynote speech at the NZGS 19th Symposium in 2013. Poulos, H.G. (2013). Practical approaches to seismic design of deep foundations. Proc. 19th NZGS Geotechnical Symposium. Hanging by a Thread. Ed. C Y Chin, Queenstown (2013). ISSN 0111-0532, Vol. 38, Issue 1(GM) 2013.**

ABSTRACT
This paper sets out a simplified approach by which the practical foundation designer can undertake the relevant calculations to satisfy the requirements for deep foundation design in seismic areas. The following matters are dealt with:

a. Design issues that should be addressed;
b. Pile design for axial loading, including the possible effects of liquefaction;
c. Pile design for lateral loading where liquefaction does not occur;
d. Pile design for lateral loading where liquefaction does occur;
e. Measures to mitigate against liquefaction effects.

1. INTRODUCTION

Consideration of the effects of earthquakes and seismic loadings is an increasingly important aspect of modern foundation design, and most contemporary standards have a mandatory requirement for such consideration. For example, the Australian Piling Code, AS 2159-2009, states that "a pile shall be designed for adequate strength, stiffness and ductility under load combinations including earthquake design actions". However, the methods by which such considerations can be undertaken are generally not set out in the standards, and many approaches have been utilized, ranging from very simplistic methods to extremely complex computer analyses. Accordingly, there appears to be scope for an approach that is soundly based but which is neither too simplistic nor too complex.

Poulos (1989) has suggested that there are three categories of analysis and design, as follows:

1. Category 1: empirical methods.
2. Category 2: simplified but soundly-based methods.
3. Category 3: more comprehensive methods that are soundly-based, and site-specific.

The objective of this paper is to set out a systematic but simplified approach which falls into Category 2 above, and by which the foundation designer can undertake the relevant analyses to satisfy the foundation design requirements for seismic regions. Emphasis is placed on methods that do not require "black box" software or which employ complex soil models in which the physical meaning of the parameters is unclear.

2. DESIGN ISSUES

In addition to the conventional design issues of axial load capacity, settlement, structural adequacy and durability under static imposed loadings, the design of deep foundations in seismic areas requires consideration of the following factors:

1. The effects of earthquake excitation on the axial load capacity of the foundation system;
2. The effects of earthquake excitation on the lateral response and the structural integrity of the foundation system.

In both cases, the possible loss of soil support during the earthquake due to liquefaction or partial loss of soil strength must be considered. Both the geotechnical and the structural strength of the foundations can be compromised by the earthquake effects, and so each has to be examined in turn.

For assessment of the response of the foundation during a seismic event, it is also necessary to estimate the stiffness and damping of the foundation system, since the foundation response can influence the natural period of the supported structure.

3. EFFECTS OF EARTHQUAKES ON FOUNDATION SOILS

Soil deposits at a site subjected to an earthquake may experience the following effects:

- Increases in pore pressure within the soils;
- Time-dependent vertical ground movements during and after the earthquake;
- Time-dependent lateral ground movements during the earthquake.

In foundation design, consideration must therefore be given to possible reductions in soil strength arising from the build-up of excess pore pressures during and after the earthquake. In extreme cases, the generation of pore pressures may lead to liquefaction in relatively loose sandy and silty soils.

As a consequence of the earthquake-induced ground movements, piles and other deep foundations will be subjected to two sources of additional lateral loading:

a. **Inertial loadings** – these are forces that are induced in the piles because of the accelerations generated within the structure by the earthquake. Consideration is generally confined to lateral inertial forces and moments, which are assumed to be applied at the pile heads.

b. **Kinematic loadings** – these are forces and bending moments that are induced in the piles because of the ground movements that results from the earthquake. Such movements will interact with the piles and because of the difference in stiffness of the piles and the moving soil, there will be lateral stresses developed between the pile and the soil, resulting in the development of shear forces and bending moments in the piles. These actions will be time-dependent and need to be considered in the structural design of the piles.

Thus, in addition to the usual design considerations for static loading, the above factors of strength reduction, inertial loadings, and kinematic loadings, need to be incorporated into the design process.

When considering both the strength and stiffness of soils, consideration should also be given to the effects of the high rate of loading that occur during a seismic event. Such loading rate effects tend to increase both the strength and stiffness of soils, especially clay soils.

Appropriate assessment of the geotechnical parameters is a critical component of geotechnical design for seismic actions, as it is for other types of imposed loadings. This issue is however outside the scope of the present paper, and reference should be made to references such as Kramer (1996) who discusses such issues as the effects of strain, cyclic loading and loading rate effects.

3.1 Earthquake Characteristics Required for Foundation Design

Seismic design of foundations requires information on a number of characteristics of the anticipated earthquakes at the site, including the following:

- Earthquake size or magnitude;
- Seismicity rate;
- Maximum bedrock acceleration, and it attenuation with distance from the causative fault;
- The duration;
- The predominant period;
- Representative time-acceleration relationships at bedrock level.

More details of these and other earthquake characteristics can be found in Kramer (1996). However, interaction with seismologists is highly desirable in making the above assessments for a specific site or area.

4. PILE DESIGN FOR AXIAL LOADING

4.1 Without Liquefaction

In soils that are assessed to be non-liquefiable, the conventional methods may be adopted to assess the ultimate shaft friction and end bearing capacity of the piles, including the possible effects of group action.

Only limited attention has been paid to the axial response of piles during and after an earthquake. Poulos (1993) has provided an example of a pile in clay subjected to earthquake action. The generation of excess pore pressures, and the consequent loss of strength of the clay, has been incorporated into the analysis. The main conclusions drawn from this study are as follows:

1. Earthquakes with a Richter Magnitude in excess of

about 6 have the potential to generate significant excess pore pressures in the clay, causing subsequent consolidation settlement.

2. The rate of development of these settlements is similar to that obtained from Terzaghi's consolidation theory, using the coefficient of consolidation for the soil in an over-consolidated state.
3. Piles in clay may be subjected to a short-term loss of axial capacity due to the "softening" of the surrounding soil arising from pore pressure build-up.
4. Piles may also experience a long-term increase in settlement and axial force, due to the ground settlements induced by the earthquake.

4.2 Effects of Liquefaction

If liquefaction is assessed to be likely to occur within soils supporting a piled foundation, some of the issues that require consideration are as follows:

a). Seismic excitation will cause settlements as well as lateral ground movements and thus there will be a tendency for the development of negative skin friction on those parts of the pile which tend to settle less than the soil.

b). There will tend to be a substantial reduction in effective stress in the soil due to the generation of excess pore pressures, and this will lead to a reduction in the lateral effective stress between the pile and the soil, and a consequent reduction in the ultimate shaft friction. This will reduce the axial capacity of the pile and the factor of safety against geotechnical pile failure. This reduction of capacity is however temporary and the initial shaft resistance should be largely re-instated after the excess pore pressures have dissipated.

To avoid temporary failure, a check should be made of the ultimate axial capacity of the pile, excluding any resistance from the liquefiable layers. Ideally, this reduced capacity should exceed the design axial load that is likely to be applied to the pile during the earthquake by a suitable factor of safety, for example, 1.25.

An additional mode of failure, buckling of the pile, may be initiated. This matter is discussed further below.

4.3 Pile Buckling

Bhattacharya and Bolton (2004) have identified another mechanism of axial failure of piles in liquefiable soils, namely buckling under axial loading due to loss of lateral support. This mechanism is generally overlooked in design, yet can be an important contributor to foundation failure. They emphasize that buckling of a pile is an unstable and destructive failure mechanism, whereas pile bending is a more stable mechanism. Pile buckling is a possible mechanism if the following conditions exist:

- The pile is end bearing and socketed into rock;
- The pile is carrying a relatively large axial load compared to the Euler buckling load of an equivalent column.

Madabhushi and May (2009) recommend that the pile design process should incorporate the following considerations:

- The ratio of the axial load to the Euler buckling load should be limited to about 1/5 to provide a safety margin on buckling;
- The slenderness ratio of the piles, $SR = L/(I/A)^{0.5}$ (where L=effective pile length within the liquefiable layer, I = minimum moment of inertia, A = pile cross-sectional area), in the buckling zone should be no greater than 50 to avoid buckling instability.

The Euler buckling load P_E can be estimated as follows:

$$P_E = \pi^2 EI/L_e^2 \qquad (1)$$

where EI=flexural rigidity of the pile
L_e = equivalent length of pile.

The equivalent length Le depends on the end conditions of the pile. For the extreme case of a pile head that is unrestrained against both translation and rotation, but with a fixed base, $L_e = 2L_o$, where L_o = length of pile in liquefied soil, while for a pile head that is restrained against rotation, $L_e = L_o$. Alternatively, estimates of the pile buckling load can be made using the results summarised by Poulos and Davis (1980).

5. PILE DESIGN FOR LATERAL LOADING – NO LIQUEFACTION

5.1 Assessment of Inertial Loadings

For geotechnical analysis, the inertial forces imposed on the foundation system by the structure are usually obtained by the structural designers. Broadly, the maximum lateral force on the structure is generally estimated by multiplying the mass of the structure by the peak spectral acceleration, a_{smax}. a_{smax} can be obtained most readily via the use of code-specified values for bedrock acceleration and then applying site factors to obtain surface acceleration. For a piled foundation, the lateral inertial force can be estimated as $P.a_{smax}/g$ where P = axial force on pile, and g = gravitational acceleration.

Rather than simply adopting a code-specified site factor, a site response analysis using representative earthquake records at "bedrock" level (e.g. where $v_s > 750$m/s) may be carried out to assess the ground response. From this analysis, response spectra can be obtained for the acceleration versus time history at an appropriate depth in the layer, e.g. at about 1/2 to 2/3 of the pile length, to reflect the effect of the structure being founded on a pile foundation system.

If an elastic analysis is applied to the pile and a linearly varying Young's modulus with depth is assumed to apply within the soil, then the maximum bending moment due to inertial loading, M_{imax}, can be estimated approximately from the following expression given by Randolph (1981):

a) For a free-head pile:

$$Mimax = 0.1 H_i L_c / \rho c \qquad (2)$$

b) For a fixed head pile: (fixing moment at the pile head):

$$M_{imax} = -0.1875\ H_i\ L_c\ /(\rho_c)^{0.5} \qquad (3)$$

where H_i = inertial force on pile
L_c = effective pile length = d(Ep/Gc) 2/7 (4)
G_c = average shear modulus of soil over a depth equal to the effective length of the pile
ρ_c = ratio of soil modulus at a depth of · of effective length to that at a depth of · of the effective pile length.

5.2 Assessment of Kinematic Loading Effects

To estimate the possible additional bending moments and shears in the piles arising from the kinematic ground movements during and after a seismic event, there are at least two design approaches that may be adopted:

a) A simple approach employing the results of analyses reported by Nikolaou et al (2001), among others
b) A more detailed approach involving the use of a pseudo-static analysis in which the results of a site response analysis are combined with a pile-soil interaction analysis (e.g. Cubrinovski and Ishihara, 2004, Tabesh & Poulos, 2001).

5.2.1 Simplified Analysis Method for Kinematic Moments in Pile

A convenient design approach for estimating the maximum moment induced in a pile by kinematic bending has been provided by Nikolaou et al (2001). They found that the induced moments were a maximum at interfaces between layers of different stiffness and then undertook a series of analyses to compute the bending moment at the interface between two layers (see Figure 1). They recognised that a distinction must be made between the maximum bending moment under steady state harmonic motion and the bending moment that would be developed under transient excitation, such as during an earthquake. The latter would generally be smaller than the steady state value, which would only be developed after a very large number of cycles. This distinction was also emphasized by Sica et al (2011) and Figure 2 shows, diagrammatically, the relationship between the bending moment and the frequency ratio (ratio of the predominant frequency of the earthquake to the natural frequency of the subsoil.

The following approximate relationship was developed for the peak bending moment, Mpk, during the transient phase of seismic excitation:

$$M_{pk} = \eta .\ M_{res} \qquad (5)$$

where M_{res} = bending moment developed under resonant conditions
η= reduction factor to allow for non-resonant conditions.

From the results of a frequency domain analysis, Nikolaou et al developed the following fitted formula for M_{res}:

$$M_{res} = 0.042\ \tau_c\ d^3\ (L/d)^{0.30}\ (E_p/E_1)^{0.65}\ (v_{s2}/v_{s1})^{0.50} \qquad (6)$$

$$\text{with} \qquad \tau_c = a_s\ \rho 1\ h1 \qquad (7)$$

where d = pile diameter
L= pile length
E_p = Young's modulus of pile
E_1 = Young's modulus of upper layer
v_{s1} = average shear wave velocity in upper layer
v_{s2} = average shear wave velocity in lower layer
a_s = peak ground surface acceleration
ρ_1 = mass density of upper layer
h_1 = thickness of upper layer.

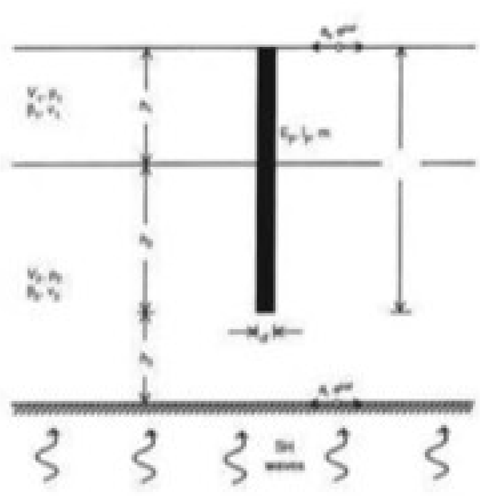

Figure 1: Model adopted by Nikolau et al (2001) for kinematic moments in pile

In the original paper, Nikolaou et al give the following expressions for the reduction factor h:

Case 1: For resonant conditions in which the fundamental period of the deposit lies within the range of predominant periods of the excitation:

$$\eta = 0.04N_c + 0.23 \quad (8)$$

Case 2: For non-resonant conditions where the fundamental period of the deposit lies outside the range of predominant periods of the excitation:

$$\eta = 0.015N_c + 0.17 \approx 0.2 \quad (9)$$

where N_c = effective number of cycles within the earthquake record.

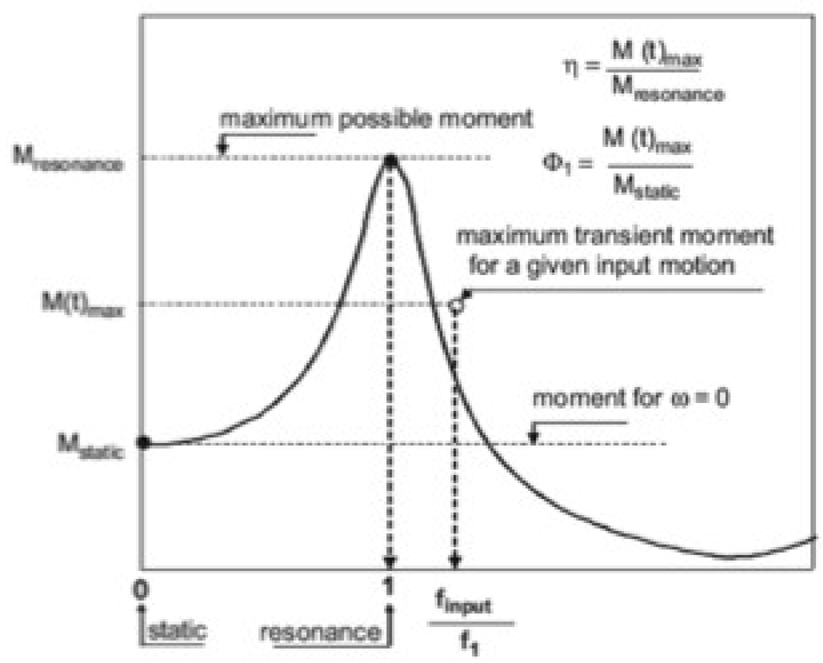

Figure 2: Relationship between pile bending moment and frequency ratio (Sica et al, 2011)

Subsequently, Sica et al (2011) have suggested the following alternative expression for the average value of h based on a series of parametric analyses:

$$\eta = 0.68(f_{input}/f_1)^{-1.5} \quad (10)$$

where f_{input} = predominant frequency of earthquake
f_1 = natural frequency of soil profile.

The above expression holds for $(f_{input}/f_1) \geq 1.5$, but for frequency ratios less than 1.5, $\eta = 0.37$, with a standard deviation of 0.17.

The value of f_{input} can be obtained from the input earthquake record, but can also be estimated (very approximately) as the inverse of the predominant period. The value of the natural period of the soil deposit, f_1, can be obtained from the following expression:

$$f1 = v_{sav} / 4H \quad (11)$$

where H = thickness of soil profile
v_{sav} = average shear wave velocity of soil profile.

5.2.3 Pseudostatic Analysis for Pile Response

Tabesh and Poulos (2001) have proposed a pseudostatic approach for estimating the maximum response of a pile during an earthquake. The approach involves the following steps:

1. A free field site response analysis is carried out to obtain the time history of surface motion and the maximum horizontal displacement of the soil along the length of the pile. In general, a one-dimensional analysis can be employed, using commercially available codes such as SHAKE or DEEPSOIL, or else custom codes such as ERLS (Poulos, 1991).
2. The surface motion obtained in the above step is used in a spectral analysis of a single degree of freedom system whose natural period is equal to that of the supported structure. The spectral acceleration a_{spec} is thus obtained.
3. A static analysis of the pile is carried out in which the pile is subjected simultaneously to the application of the following loadings:
   (i) A lateral force at the pile head equal to a_{spec}.P, where P = vertical load acting on pile head;
   (ii) The maximum ground movements along the pile length, as obtained from Step 1.

The analysis will give the maximum moment and shear force developed in the pile by the simultaneous application of the inertial and kinematic loadings.

5.3 Combined Inertial and Kinematic Effects

The approach employed by Tabesh and Poulos (2001) may be conservative as the analysis implicitly assumes that both the inertial and kinematic loadings are in phase. However, they have found that this approach gives reasonable agreement with the results of a more complete dynamic analysis, although it does tend to be conservative, and also good agreement is found when applied to a case history in Japan.

A modification has been suggested by Tokimatsu et al (2005), using the following approach to deal with combined inertial and kinematic loadings:

- If the natural period of the superstructure is less than that of the ground, the kinematic force tends to be in phase with the inertial force, increasing the stress in the piles. The maximum pile stress occurs when both the inertial force and the ground displacements take the peak values and act in the same direction. In this case, the maximum moment is the sum of the values for inertial and kinematic effects
- If the natural period of the superstructure is greater than that of the ground, the kinematic force tends to be out of phase with the inertial force. This restrains the pile force, rather than increasing it. The maximum pile stress tends to occur when both inertial force and ground displacement do not become maxima at the same time. In this case, the maximum moment is the square root of the sum of the squares of the moments due to each effect

The moments via this approach have been found to be in good agreement with model tests, both with and without the effects of liquefaction.

5.3 Design Charts

Using the pseudostatic approach, Tabesh and Poulos (2007) produced some simple design charts for piles within a uniform soil profile. Figure 3 shows the problem addressed and Figure 4 gives examples of these charts for the case of a 20m long pile with a vertical load corresponding to a factor of safety of 2.5 against geotechnical failure. These charts are meant to provide only a very preliminary estimate of lateral pile response.

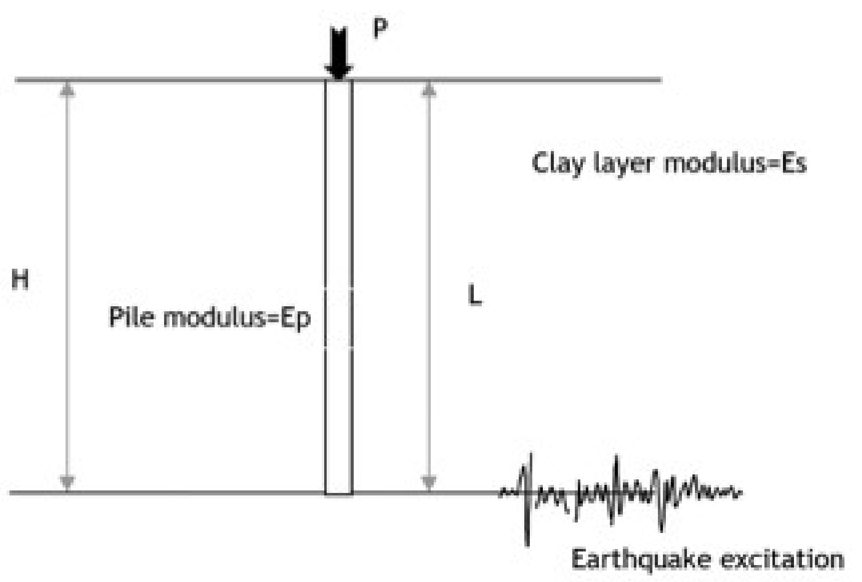

Figure 3: Problem considered for design charts

6. PILE DESIGN FOR LATERAL LOADING – INCLUDING LIQUEFACTION

6.1 Introduction

Soil liquefaction during a seismic event results in almost a complete loss of strength and stiffness in the liquefied soil, and consequent large lateral ground movements. The consequences of liquefaction on pile foundations can be very significant and can result in failure of the piles, as evidenced by many case histories during the 1995 Kobe earthquake (e.g. Ishihara and Cubrinovski, 1998) and the 2010-2011 events in Christchurch (e.g. Cubrinovski, 2013). A rational analysis of the behaviour of a pile in a liquefiable soil during and after an earthquake needs to take account of the effects of liquefaction on the soil properties, the natural period of the ground and the ground movements.

6.2 Simplified Analysis Based on Soil Stiffness Reduction

A procedure that has been employed in Japan to allow for the effects of liquefaction is to reduce the soil parameters by a factor which depends on the following factors:

i. The factor of safety against liquefaction, FL;
ii. The depth below the ground surface;
iii. The original SPT (N) value of the soil.

An example of values of a reduction factor (r_k) for the modulus of subgrade reaction (k) for building foundation design in Japan is shown in Table 1 (JGS, 1998). This approach is very simplified, and considers only the effects of inertial loading. It does not incorporate the effects of kinematic loading.

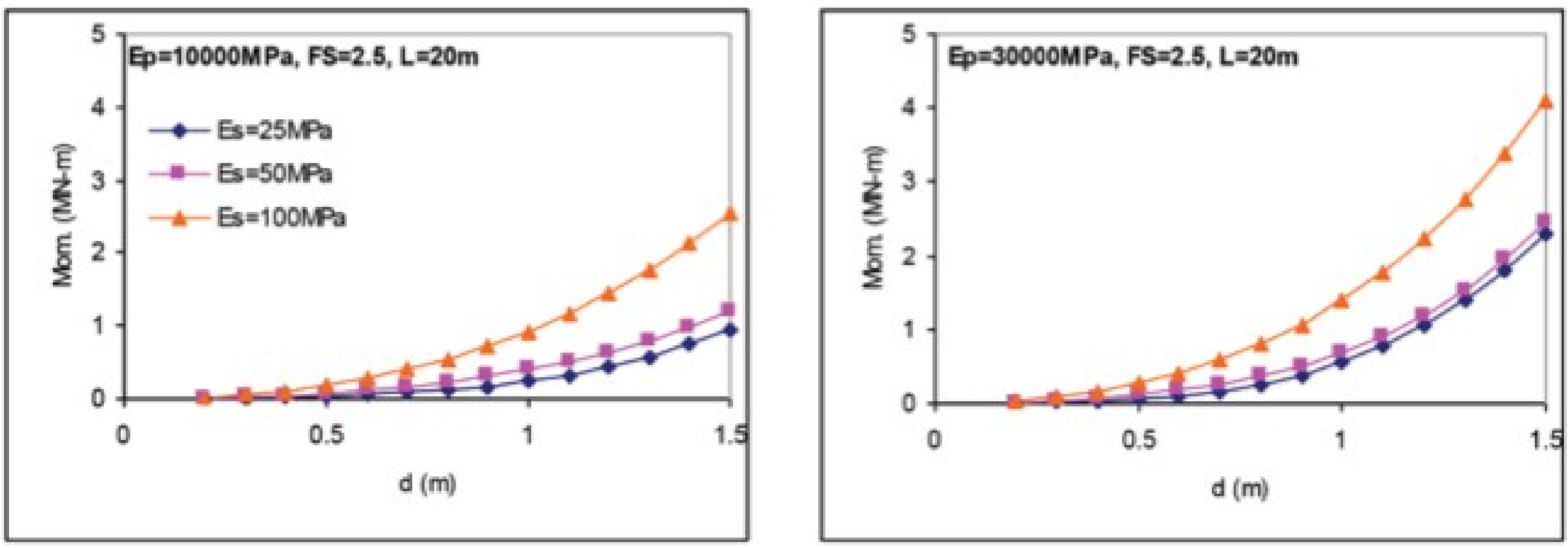

Figure 4: Typical design charts for maximum moment in 20m long pile (Tabesh & Poulos, 2007).

Factor of Safety Against Liquefaction, F_L	Depth z below ground surface (m)	Reduction factor r_k, applied to modulus of subgrade reaction k			
		N≤8	8<N≤14	14<N·20	N>20
≤0.5	0≤z≤10	0	0	0.05	0.1
	10<z≤20	0	0.05	0.1	0.2
0.5<FL≤0.75	0≤z≤10	0	0.05	0.1	0.2
	10<z≤20	0.05	0.1	0.2	0.5
0.75<FL≤1.0	0≤z≤10	0.05	0.1	0.2	0.5
	10<z≤20	0.1	0.2	0.5	1.0

Table 1 Reduction factor for modulus of subgrade reaction – building foundations (JGS, 1998)

6.3 Simplified Approach Based on Parametric Analyses

Valsamis et al (2012) have carried out an extensive set of parametric analyses and developed simplified expressions for lateral pile response in a laterally spreading soil profile. Three cases have been considered, as illustrated in Figure 5, and are as follows:

1. Case a: a 2-layer soil profile where a thick liquefiable layer overlies a non-liquefiable layer, and the pile head is unrestrained.
2. Case b: a 2-layer soil profile, as in Case a, but with the pile head restrained from both translation and rotation.
3. Case c: a 3-layer soil profile, where the liquefiable layer lies between two non-liquefiable layers, an overlying crust, and a lower continuous layer. The pile head is restrained from rotation but can translate freely.

The following expressions were derived for the maximum pile deflection, dpile:

Case a: $\delta_{pile} = \min[1.22\delta_{gr}, 29b^{1.44}.\delta_{gr}^{0.28}.J^{0.72}]$ (12)

Case b: $\delta_{pile} = \min[0.15\delta_{gr}, 0.12b^{0.74}.\delta_{gr}^{0.26}.J^{0.74}]$ (13)

Case c: $\delta_{pile} = 1.22\delta_{gr}$ (14)

where δ_{pile} = ground surface movement (m)
$J = H^6 l_{iq.} d/EI$
E = pile modulus (kPa)
I = pile moment of inertia (m^4)
H_{liq} = depth of liquefied soil (m)
d = pile diameter (m)
b = parameter related to corrected SPT value $(N_1)_{60,cs}$ and derived from Figure 6.

The maximum bending moment, M_{max}, is approximated by the following expressions:

Case a: $M_{max} = 2.2EI.\delta_{pile}/H^2_{liq}$ (15)

Case b: $M_{max} = 18EI.\delta_{pile}/H^2_{liq}$ (16)

Case c: $M_{max} = 17(EI.\delta_{pile}/H^2_{liq})^{0.65}$ (17)

In all cases, it is very important to use the correct units indicated above.

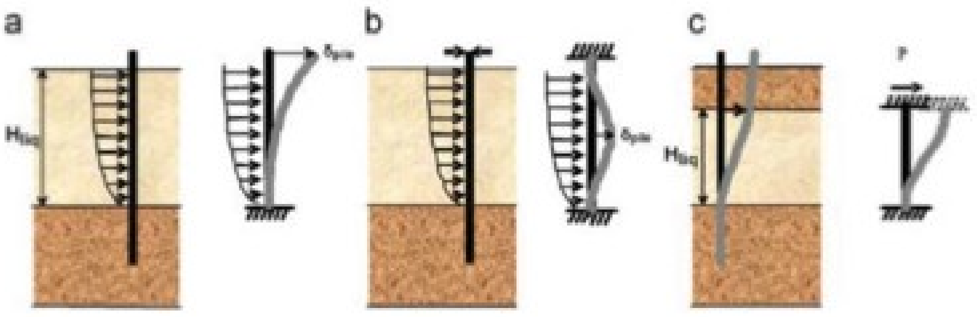

Figure 5: The design cases considered by Valsamis et al (2012)

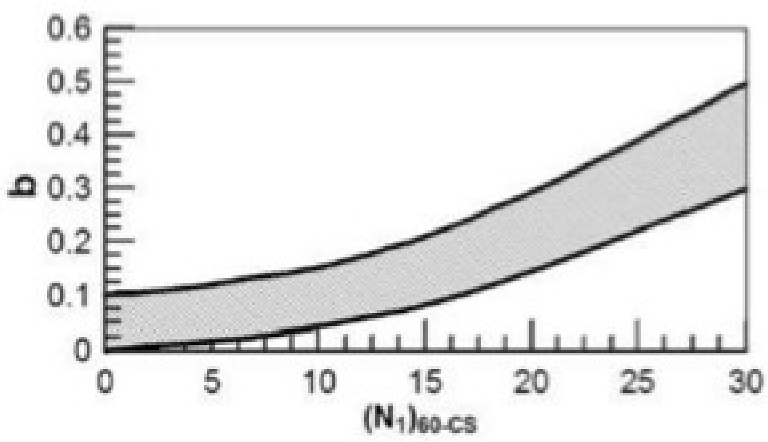

Figure 6: Parameter "b" related to corrected SPT value (Valsamis et al, 2012)

A practical difficulty with the above procedure is that the ground surface movement of a liquefied layer, dgr, is very difficult to estimate, and the computed responses depend greatly on this value.

6.4 Modified Approach Using Nikolau et al (2001)

6.4.1 Procedure

Use can be made of the equations developed by Nikolau et al (2001) to develop a simpler approach in which the amount of computational effort is relatively limited. This approach involves the following steps:

1. Estimate the reduction in shear modulus via the approach suggested by Ishihara and Cubrinovski (1998), in which the post-liquefaction shear modulus, G_{pl}, is given by:

$$G_{pl} = \beta G \qquad (18)$$

where β = reduction factor;
G = shear modulus prior to liquefaction.

2. Estimate the bending moment for resonant conditions via equations 6-11. The shear wave velocity in the (upper) liquefied layer is now $v_{s1\text{-}liq} = v_{s1} \sqrt{\beta}$ where v_{s1} is the original shear wave velocity of the layer.
3. Estimate the modified natural frequency of the liquefied layer from equation 11, using the reduced shear wave velocity $v_{s1\text{-}liq}$.
4. Compute the correction factor η from either equations 8 and 9, or equation 10.
5. Compute the bending moment from equation 5.

6.4.2 Modulus Reduction Factor η

A key parameter choice is the value of the modulus reduction factor β. Cubrinovski (2006) suggests that b values range between 0.02 and 0.10 for cyclic liquefaction, and 0.001 to 0.02 for lateral spreading.

An alternative approach to estimating β is to relate it to the factor of safety against liquefaction F_L, or perhaps more logically, to the liquefaction potential index LPI, which is defined as follows:

$$LPI = \cdot F(z)\, w(z)\, dz \qquad (19)$$

where $F(z) = 1-F_L$ when $F_L<0.95$, and FL = factor of safety against liquefaction
$F(z) = 2x10^{-6} \exp(-18.427F_L)$ when $0.95<F_L<1.2$
$F(z)= 0$ when $F_L \geq 0$
$W(z) = 10-0.5z$
z = depth below surface in metres.

The integration in equation 19 is carried out from the surface to a depth of 20m. Table 2 summarises the general levels of risk based on LPI.

To do this, use can be made of the recommendations shown in Table 1, from which ranges of values of LPI can be obtained from the range of values of F_L given in that table for the upper 10m of the profile and profile from 10 to 20m depth. Figure 7 shows the values of the modulus reduction factor β can be plotted against the derived average values of LPI, and from this plot, an empirical relationship can be drawn as follows:

$$\beta = \beta_{lim} + e^{b.LPI} (1-\beta_{lim}) \qquad (20)$$

where β_{lim} = lower limit value of b
b = index derived from fitting through the points in Figure 7, and found to be approximately -0.20
LPI = liquefaction potential index.

This relationship is also shown in Figure 7 for b = -0.2 and blim = 0.01, and shows reasonable agreement with the points derived from Table 1, despite the considerable scatter.

LPI	Liquefaction Risk
0	Non-liquefiable
0-2	Low
2-5	Moderate
5-15	High
>15	Very high

Table 2: Liquefaction Risk Based on Liquefaction Potential Index, LPI

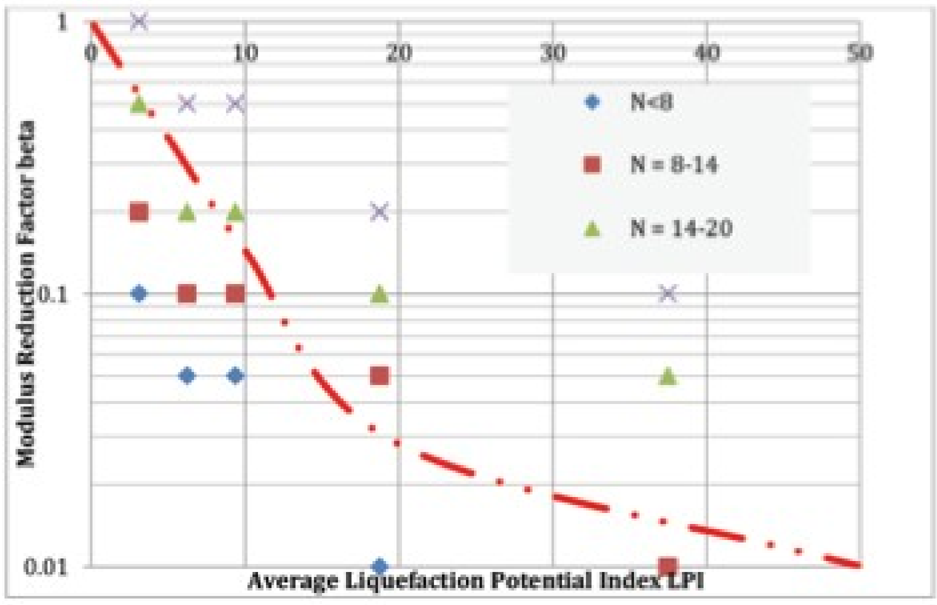

Figure 7: Relationship between modulus reduction factor b and liquefaction potential index LPI

6.4.2 Upper Limit to Kinematic Bending Moment

There will be an upper limit to the kinematic bending moment, which will occur if the layer has completely liquefied and flows past the pile. In this case (assuming that the applied pressure from the liquefied layer acts in the same direction along the whole layer), the limiting kinematic bending moment, Mklim, will be given by the following approximate expression):

$$M_{klim} = n.s_{u\text{-}Liq}.d\ h_1(h_1+dh) \quad (21)$$

where n = undrained shear strength multiplier for limiting lateral pile-soil pressure

$s_{u\text{-}Liq.}$ = shear strength of liquefied soil
d = pile diameter
h_1 = thickness of liquefied layer
dh = additional distance within underlying layer at which the maximum moment occurs (typically expected to be 0.5d-1d).

Data presented by Cubrinovski et al (2009) indicate that the parameter n in equation 21 may be about 9 (i.e. 4.5 times the Rankine passive pressure).

6.4.3 Estimation of Inertial Bending Moment in Pile

In cases where complete liquefaction of the upper layer occurs, with resultant lateral spreading of the liquefied soil, it has been suggested by Klimis et al (2004) that inertial effects can be neglected. However, the analyses and comparisons carried out by Liyanapathirana and Poulos (2005) indicate that inertial effects can be present and should be considered.

Following the recommendations of Liyanapathirana and Poulos (2005), the inertial force H_i can be estimated as follows:

$$H_i = a_s . P \tag{22}$$

where a_s = peak ground acceleration;
P = vertical load acting on pile.

Assuming (albeit boldly) that an elastic analysis can be applied to a pile in a liquefied layer, and that a constant Young's modulus applies to the liquefied layer, the maximum bending moment due to inertial loading, M_{imax}, can be estimated from the following expression derived from Randolph (1981):

a) For a free-head pile:

$$M_{imax} = 0.1H_i (L_c + h_i) \tag{23}$$

b) For a fixed head pile: Fixing moment at the pile head):

$$M_{imax} = -0.1875\ H_i (L_c + h_i) \tag{24}$$

where H_i = inertial force on pile
h_i = depth of liquefied soil
L_c = critical pile length in the non-liquefied soil, and approximated as:
$L_c = d(E_2/G_{red})^{2/7}$
G_{red} = shear modulus of the non-liquefied layer.

6.4.4 Effect of Near-Surface Crust above Liquefiable Layer

In cases where a stiffer (and non-liquefiable) crust may exist above the liquefiable layer, account needs to be taken of the effect of the stiffness of this crust on the behaviour of the pile under inertial loading, and its effect on the limiting bending moment that can be imposed on the pile.

An approximate, but conservative, allowance for the stiffness of the crust may be made by using a weighted average Young's modulus, E_{cl}, of the crust and the liquefiable layer, as follows:

$$E_{cl} = (E_c . h_c + E_l . h_l)/(h_c + h_l) \tag{25}$$

where E_c = Young's modulus of crust
h_l = thickness of crust
E_l = Young's modulus of liquefiable layer
h_l = thickness of liquefiable layer.

The maximum inertial bending moment can then be obtained from equation (23) for a free-head pile, or equation (24) for a fixed head pile.

To obtain the maximum kinematic bending moment at the interface between the liquefiable and non-liquefiable layers, the crust should not influence this value, and the simplified approach may again be used, using as before, the modulus of the liquefiable layer G_{pl} (equation 18).

When estimating the limiting bending moment that can be imposed on the pile, an additional shear force, H_{cr} and moment M_{cr} will be applied to the pile by the crust. These can be estimated as follows:

$$H_{cr} = h_{cr} . p_{ucr.} d \tag{26}$$

$$M_{cr} = H_{cr} (h_1 + 0.5h_{cr} + L_c) \tag{27}$$

where h_{cr} = thickness of non-liquefied crust
p_{ucr} = ultimate lateral crust-pile pressure, and which can be estimated as approximately 4.5 times the Rankine passive pressure exerted by the crust.
d = pile diameter
h_1 = thickness of liquefied layer (below crust)
L_c = critical pile length in the non – liquefied soil layer (see equation 24).

The combination of inertial and kinematic effects can then be considered as per the recommendations of Tokimatsu et al (2005) in Section 5.3.

6.5 Pseudostatic Analyses

6.5.1 Cubrinovski et al (2009).

The model developed by Cubrinovski et al(2009) is shown in Figure 8. The soil profile contains three components:

- A surface layer or crust which does not liquefy;
- A liquefiable layer;
- An underlying base layer that does not liquefy.

Each layer is characterised by a stiffness (expressed in terms of a modulus of subgrade reaction) and a limiting pile-soil pressure. The pile is analysed as a beam-spring model, and non-linear behaviour of the pile, as well as the soil, can be incorporated into the analysis by use of a finite element approach. Account is taken of both inertial and kinematic loadings.

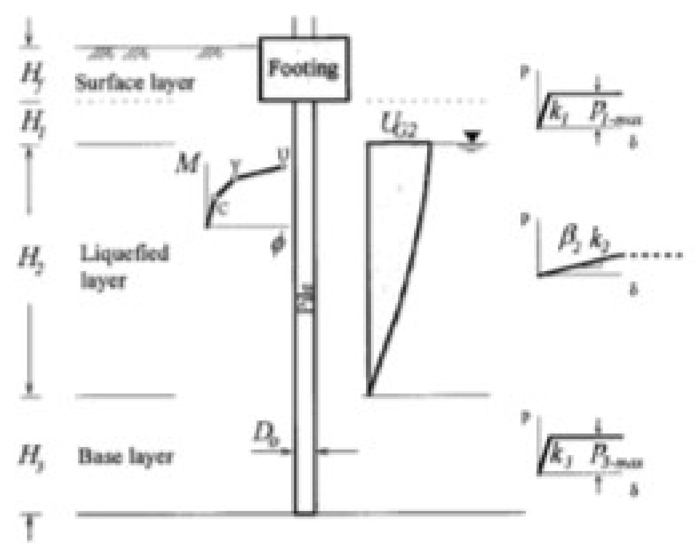

Figure 8: Model for pile in liquefied soil (Cubrinovski et al, 2009)

6.4.2 Liyanapathirana and Poulos (2005)

For piles in soils subject to liquefaction, Liyanapathirana and Poulos (2005) have developed an extension of the approach used by Tabesh and Poulos (2001) for piles in non-liquefiable soils. Account has been taken of the degradation of shear modulus of the soil that occurs with the generation of pore water pressure in the soil. The shear modulus of the soil is assumed to vary with the effective stress level of the soil as shown below:

$$G_s = G_O \left((1 + 2KO)\,\sigma\upsilon\,/pa\right)^{0.5} MPa \qquad (28)$$

where sv' is the effective stress level of the soil, K_O is the coefficient of earth pressure at rest, pa is atmospheric pressure, and G_O is a constant which varies with the relative density, D_r, of the soil.

Although spectral acceleration has been used by Tabesh and Poulos (2001), it has been found that the inertial force at the pile head calculated using the spectral acceleration may overestimate the pile response when the surrounding soil starts to liquefy. Hence, the maximum acceleration at the ground surface, rather than the spectral acceleration, has been used to calculate the inertial force at the pile head.

The calculation steps involved in this approach can be summarised as below:

1. First, a free-field site response analysis is performed by taking into account the pore pressure generation and dissipation in the soil deposit due to the earthquake loading (Liyanapathirana and Poulos, 2002). From this analysis, the maximum ground surface acceleration, maximum ground displacement along the length of the pile, the minimum shear modulus and the effective stress level attained during the seismic activity can be obtained.
2. The superstructure is modelled as a concentrated mass at the pile head. Generally the superstructures supported by pile foundations are multi-degree-of-freedom systems but in the design of pile foundations, the superstructure is reduced to a single mass at the pile head to simplify the analysis.
3. The lateral force to be applied at the pile head is the cap-mass (vertical load divided by g), multiplied by the maximum ground surface acceleration obtained from the ground response analysis.
4. The pile-soil interaction is modelled using the spring coefficients calculated from elastic theory, based on the minimum shear modulus of the soil deposit at each depth, at any time, given by the free-field site response analysis (Step 1).
5. A non-linear static load analysis is carried out to obtain the profile of maximum pile displacement, bending moment and shear force along the length of the pile by applying the lateral forces calculated in Steps 3 and 4, and the soil movement profile calculated in Step 1, simultaneously to the pile.

This approach has been verified by comparison with the results of centrifuge tests (Wilson et al, 1999; Abdoun et al, 1997).

Comparisons have also been made with the field measurements made in the piles at the Pier 211 in Uozakihama Island after the Hyogoken-Nambu earthquake which occurred on 17th January 1995. Piles at bridge Pier 211 have a diameter of 1.5 m, and Figure 9 shows the crack distributions observed in piles after the earthquake. Ishihara and Cubrinovski (1998) have simulated this case, using their approach, and Liyanapathirana and Poulos (2005) also analysed it using their pseudostatic approach. The water table was 2.0 m below the ground surface and the upper 20 m of this site consisted of Masado sand with an initial shear modulus of 57.8 MN/m^2 and density of 2000 kg/m^3. Soil liquefaction was observed in the Masado sand layer below the water table only. Therefore only the top 20 m layer was analysed using the effective stress method incorporating pore pressure generation and

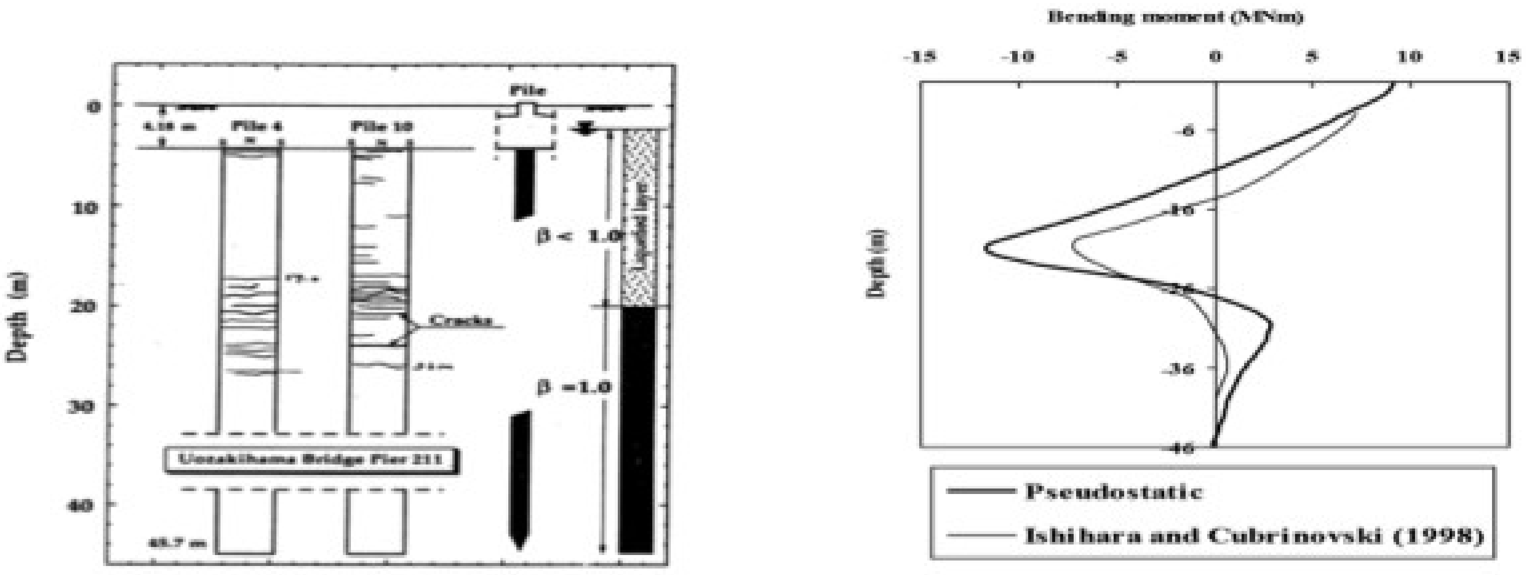

Figure 8. Cracks observed in the piles at bridge pier 211 in Uozakihama Island (Ishihara and Cubrinovski, 1998).

Figure 9: Cracks observed in piles at the bridge pier 211 in Uozakihama Island (Ishihara and Cubrinovski, 1998) and (b) Bending moment of the pile at Bridge Pier 211 in Uozakihama Island calculated from the pseudostatic approach and Ishihara and Cubrinovski (1998) results (β=0.01).

dissipation. After liquefaction, the effective stress level in the soil was reduced to a minimum of 2% of the initial effective overburden pressure. For this analysis the cyclic shear strength curve for the Masado sand given by Ishihara (1997) was used. It was assumed that the base rock had a density of 2200 kg/m^3 and a shear modulus of 75 GN/m^2.

Figure 9 shows the maximum bending moment profile along the pile obtained from the pseudostatic approach of Liyanapathirana and Poulos (2005), and also that calculated by Ishihara and Cubrinovski (1998). The lower end of the RC pile was assumed to be fixed while the pile head was assumed to be fixed to the footing but free to move in the horizontal direction. The predictions made by the pseudostatic approach agree reasonably well with the results given by Ishihara and Cubrinovski (1998). The yield moment for these piles is about 5 MNm. The computed maximum bending moment profile exceeds the yield moment near the pile head and in the vicinity of the boundary between the liquefied and non-liquefied layers. This is consistent with the location of cracks observed after the earthquake shown in Figure 9.

6.6 Example of Simplified Approach

To illustrate the application of the simplified approach, the above case of the bridge pile BP211 that was damaged in the 1995 Kobe earthquake (Ishihara and Cubrinovski, 1998) will be considered. The pile is one of a group of 22 bored piles, 1.5m in diameter, and is 41.5m long below the pile cap. The liquefiable layer is a reclaimed deposit 16m thick (below the pile cap) and has an average SPT-N value of about 10. The underlying layers consist of strata of sandy silt, with gravel, with SPT-N values ranging between about 10 and 50, with an average value of about 25. The average vertical load on a pile is 2.94 MN. A Young's modulus of the pile of 30000MPa will be assumed.

Using the relationship between shear wave velocity and SPT-N suggested by Imai and Tonouchi (1982), the shear wave velocity of the upper and lower layers is 173 and 234 m/s respectively, and the consequent small-strain shear moduli are 53.7 MPa and 103.9 MPa. Minimum and maximum damping ratios of 0.025 and 0.26 have been assumed for the strata at the site.

The earthquake characteristics at the site have not been documented, but based on other available information, it is assumed that the peak bedrock acceleration is 0.45g and that the predominant period of the earthquake is 1s.

Assuming that the piles have a fixed head, the inertial and kinematic maximum bending moments can be estimated for two cases:

- During the cyclic loading stage and prior to liquefaction;
- After liquefaction of the upper layer has developed.

(a) Cyclic Loading Phase

For the first case, the first step is to assess the amplification of ground motion, and based on the above assumptions, the natural period of the site is about 0.79s. Using, as an approximation, the classical expression for amplification of uniform harmonic motion, the ratio of predominant period to natural period is about 1.27, and for a damping ratio of 0.025, the amplification factor is found to be 2.64. Thus, the peak ground surface acceleration is estimated to be 0.45*2.64 = 1.188g. Using the small-strain modulus values, the following maximum moments are computed:

- Inertial: -7.65 MNm (using equation 24)
- Kinematic: 3.10 MNm (using the method of Nikolau et al, 2001, equations 5 to 9).

(b) Post-liquefaction phase

The small-strain modulus is used for the lower non-liquefiable layer, but a degraded modulus is used for the liquefied layer. To obtain this degraded modulus, given that liquefaction did indeed occur, a Liquefaction Potential Index, LPI, of 50 can be used. From equation 20, the reduction factor b is 0.01 (the limiting value assumed). The corresponding value of shear modulus is 0.54 MPa. The site period now becomes 1.10s, while the damping ratio has increased to its assumed maximum value of 0.26. Accordingly, the amplification factor from the classical dynamic theory decreases from 2.64 (for no shear modulus degradation) to 1.64 (for LPI=50), and the peak ground acceleration is then 0.45*1.64 = 0.738g.

From the expressions for inertial and kinematic bending moments (equations 24 and 5 to 9), the following values are obtained:

- Inertial moment (at pile head): -11.96 MNm
- Kinematic moment: 121.1 MNm.

However, a check needs to be made with the limiting kinematic bending moment that can occur when the liquefied layer flows past the pile. Adopting an undrained shear strength of 3 kPa for the liquefied soil, the limiting bending moment is found, from equation 27, to be 12.3 MNm. It is clear that the assumption of elastic behaviour in the Nikolau et al method (giving a huge kinematic moment of 121.1 MNm) is not valid in this case.

The values obtained from this simple analysis compare reasonable well those reported by Liyanapathirana and Poulos (2005) (approximately -9 MNm at the pile head and about 12 MNm at the base of the liquefied layer). Ishihara and Cubrinovski (1998) incorporated pile cracking into their analysis, and their computed maximum moments of about 7MNm reflect the occurrence of pile cracking, which limited the moment that could be induced in the piles. Figure 9 shows details of two piles that were excavated and shows the cracks in the piles at or near these two locations.

Most importantly, the simplified analysis indicates that large moments would have been generated at both the pile head and at the interface between the liquefied and non-liquefied layer.

6.7 Group Effects

Under inertial loading, it is now well-recognised that group effects are detrimental in that they tend to reduce the stiffness of piles within the group and decrease the overall group capacity. Methods of dealing with these effects are given by Poulos and Davis (1980) and Fleming et al (2009).

Under kinematic loading, group effects however tend to be beneficial, due to the "shielding" action of the piles. In particular, inner piles within a group tend to be subjected to smaller forces and moments developed by ground movements than outer piles, and all piles in the group tend to experience less effect than a single isolated pile. As a consequence, the consideration of a single isolated pile will generally be conservative when considering kinematic effects.

Towhata (2008) has presented an approximate approach for estimating the reduction in lateral earth pressure on a pile due to soil movement or flow past a group of piles. This reduction can be expressed in terms of two components:

1). A "shadow factor" which reduces the "downstream" pressure by a factor of 0.8 for each successive row of piles;
2). A spacing factor, α_p, which can be expressed as follows:

(i) For a non-liquefied layer:

$$\alpha_p = (0.877(1-\eta)f) / (0.877\eta + 0.123) + 1.0 \qquad (29)$$

where $f = 1.0$ when $h_l/d \leq 3.8$
$f = 14[1/(h_l/d)^2] + 0.03$ when $h_l/d > 3.8$
h_l = depth of flowing soil
d = pile diameter
η = ratio of pile diameter to pile spacing (d/s).

(ii) For a liquefied layer:

$$\alpha_p = 1 / (0.599\eta + 0.401) \qquad (30)$$

For practical use, it is convenient to derive a group reduction factor for the pressure developed by moving soil. If, from a practical viewpoint, s/d = 20 is considered as being sufficiently widely-spaced to represent a single pile, then the Group Reduction Factor for Pressure, RFp, can be defined as:

$$RFp = \alpha_p / \alpha_p\ (s/d=20) \qquad (31)$$

Figures 10 and 11 plot the resulting values of RFp for the non-liquefied and liquefied soil cases. The beneficial effects of group action can clearly be seen from these figures. For the non-liquefied case, the effects of grouping become increasingly beneficial as the ratio hl/d decreases, i.e. as the thickness of the moving soil decreases relative to the pile diameter.

When both inertial and kinematic effects are present, the group effects may tend to counteract each other.

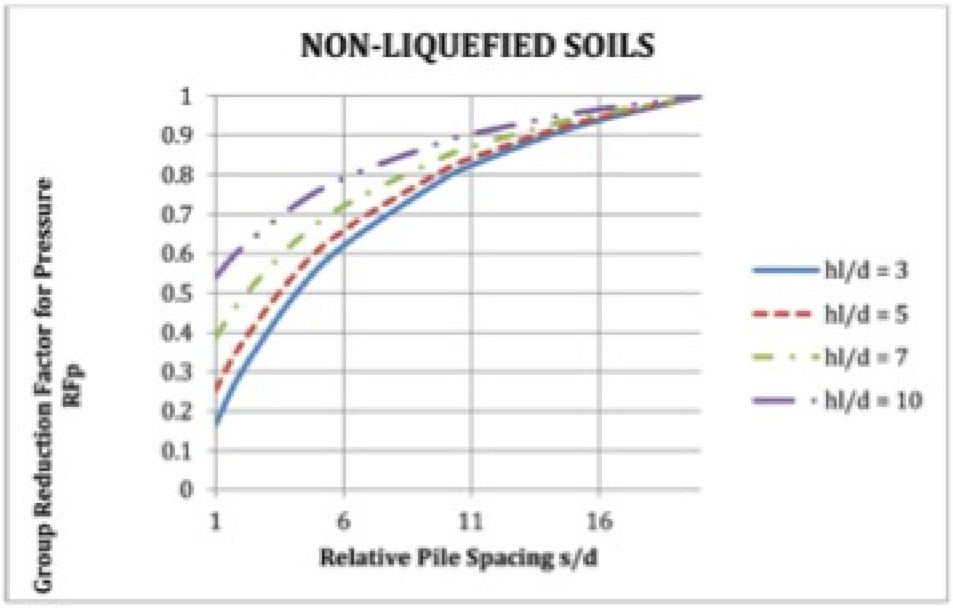

Figure 10: Group reduction factor for lateral pile-soil pressure – non-liquefied soil

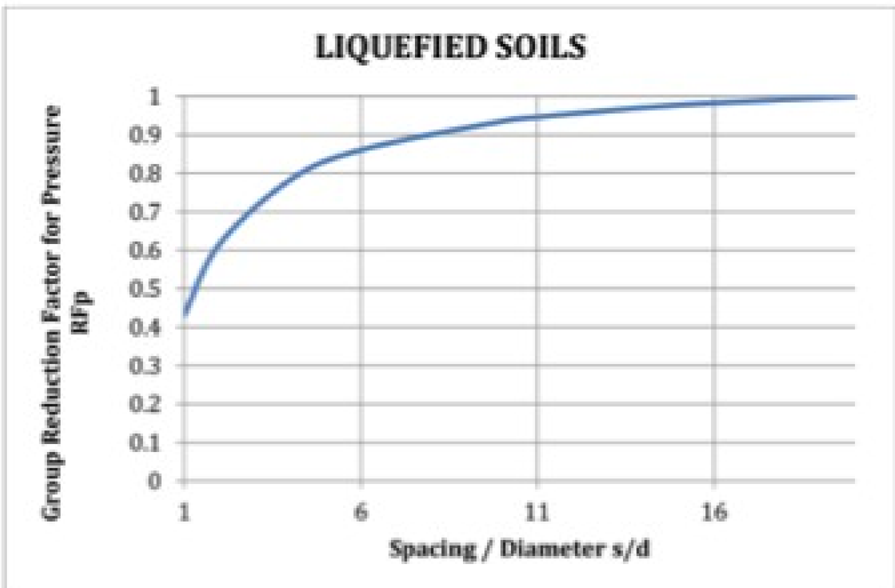

Figure 11: Group reduction factor for lateral pile-soil pressure – liquefied soil

7. FOUNDATION STIFFNESS AND DAMPING

Piles will experience vertical and horizontal movements during an earthquake due to the inertial loadings imposed by the supported structure. These movements will generally be dynamic in nature and therefore it is necessary to consider the pile head stiffnesses under dynamic loading, and the damping that will be generated by the dissipation of energy away from the piles into the surrounding soil (the radiation damping). Knowledge of foundation stiffness and damping is also required to assess the effects of structure-foundation interaction on the natural period and damping of a structure.

Theoretical solutions for the axial and lateral stiffnesses of a single pile have been provided by Novak (1987) among others. Convenient approximate solutions have been given by Gazetas (1991) for various simplified soil profiles.

In contrast to static loading, where the interaction factors between piles decrease with increasing distance between the interacting piles, the dynamic interaction factors can either increase or decrease, depending on the frequency of the loading and the spacing between the piles. It is possible that the interaction factors can be negative under some circumstances, so that group effects may possible lead to an increase in stiffness of the piles within a group, as compared with the case of static loading, where group effects generally reduce the stiffness of piles in the group. For practical design, it may often be more convenient to simplify the group as an equivalent pier, and then to use the corresponding solutions for stiffness and damping as given by Gazetas (1991).

8 MITIGATION OF LIQUEFACTION EFFECTS

8.1 Categories of Mitigation Measures

Conventional remedial measures to mitigate the effects of liquefaction can be divided into three broad categories (JGS, 1998):

- Treatment of the liquefiable soil to strengthen it;
- Treatment of the soil to accelerate the dissipation of seismically-induced excess pore pressures;
- Measures to reduce liquefaction-induced damage to the structure or facility.

8.2 Conventional Measures to Strengthen the Liquefiable Soil

An extensive discussion of measures that can be used to strengthen liquefaction-susceptible soils is given in JGS (1998). Some of the more common measures are as follows:

- Soil densification, by a variety of means, e.g. vibroflotation, blasting;
- Insertion of stiffer columns;
- Provision of drainage via stone columns.

A major limitation of all of these methods is that they cannot be used to remediate sites on which structures or facilities already exist. A number of the methods for soil densification may also not be feasible if the site is within a commercial or residential area, because of the noise and vibrations involved.

Methods involving the insertion of stiffer columns can be effective and generally involve less noise and vibration than conventional methods of densification.

8.3 Conventional Measures to Accelerate Excess Pore Pressure Dissipation

The use of stone columns or gravel drains to accelerate pore pressure drainage was developed by Seed and Booker (1977) and has been used successfully in a number of cases. In the design of stone columns for liquefaction mitigation, it is common to specify a limiting maximum pore pressure ratio (excess pore pressure divided by vertical effective stress), and this is often taken as 50%. The required length of the columns will depend on the depth of the liquefiable layer, while the required spacing of the columns will depend on the following factors:

1. The specified maximum pore pressure ratio;
2. The permeability and compressibility of the liquefiable layer;
3. The earthquake duration;
4. The equivalent number of cycles of loading from the earthquake;
5. The number of cycles to cause liquefaction;
6. The column diameter;
7. The finite permeability of the column and the effects of well resistance.

8.4 Mitigation Measures for Pile Foundations

Sato et al (2004) have suggested various countermeasures for pile foundations subjected to lateral flow of liquefiable soils. Three of these measures are illustrated in Figure 12.

The "drain piles" method aims to prevent the liquefaction of the lower layers while reducing the stiffness of the stiffer crust above the liquefiable layers. The drain piles should extend only about half-way into the upper crust, and the permeability of the drain piles should be greater than that of the liquefiable layers so that the excess pore pressures that are generated are transmitted to the upper layer. In the "earth retaining wall" method, an earth retaining wall is constructed in front of the pile foundation, to block the lateral flow. The "streamlined shield block" method involves the casting of an angled face on the upstream side of the pile cap. This angled face is meant to disperse the flowing soil and reduce its effect on the foundation.

Centrifuge tests were carried out by Sato et al (2004) to examine the effectiveness of the above countermeasures. Figure 13 summarises the test results and indicates that the "streamlined shield block" method is very effective in reducing the residual lateral displacement, and also the bending strain in the piles. It has the advantage of not requiring any ground improvement.

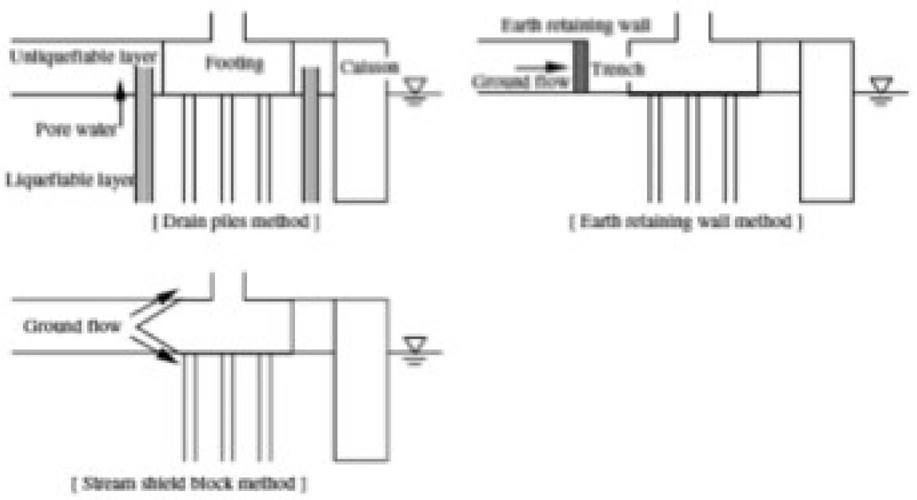

Figure 12: Countermeasures for lateral ground flow (Sato et al, 2004)

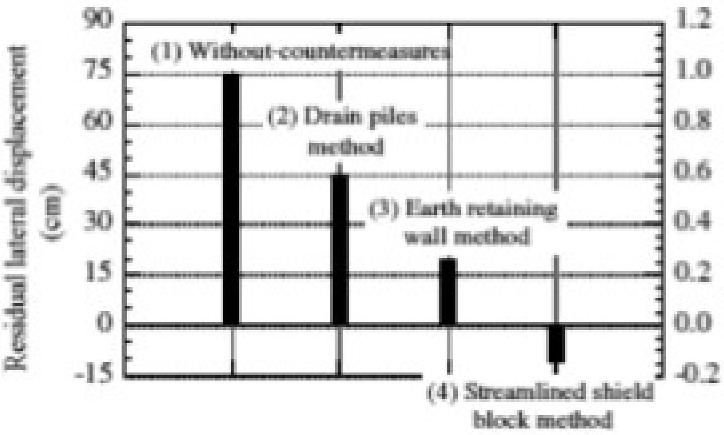

Figure 13: Measured residual lateral movements for various countermeasures (Sato et al, 2004)

8.5 SOME INNOVATIVE METHODS FOR LIQUEFACTION RISK MITIGATION

There are a number of recent innovative methods that have been explored for reducing the risk of liquefaction, several of which have the potential to be used for sites on which structures or facilities exist. A brief review of some of these methods is given below.

8.5.1 Passive Remediation via Infusion of Colloidal Silica

Colloidal silica is a dispersion of silica particles in water. With about 5% weight of silica, a colloidal silica aqueous dispersion has density and viscosity values that are similar to water, and this dispersion becomes a permanent gel abruptly after a period of time, generally a few months at most. Colloidal silica is non-toxic, biologically and chemically inert, and relatively durable. The influence of the gel is to reduce the strains in the treated soil developed by cyclic loading, and to reduce the soil permeability.

Figure 14 shows results of cyclic triaxial tests on Monterey sand, both untreated, and treated with 10% colloidal silica. The increased resistance to liquefaction is very clearly demonstrated in this figure.

Gallagher et al (2007) have described field tests to assess the performance of a dilute colloidal silica stabilizer in reducing the settlement of liquefiable soils. Slow injection methods were used to treat a 2m thick layer of liquefiable sand, using eight injection wells around the perimeter of a 9m diameter test area. The gel times ranged between 10 and 30 days. A subsequent blasting test revealed that the settlement of the treated soil was only about 60% that of an adjacent untreated area. Interestingly, there appeared to be no significant increase in the CPT resistance or the shear wave velocity, and the mechanism of improvement was considered to be due to the development of cohesion within the treated soil due to the formation of interparticle siloxane bonds.

Gallagher and Lin (2009) demonstrated that colloidal silica could be successfully delivered through 0.9m diameter columns packed with loose sand. The main factors influencing the transport of the stabilizer were the viscosity of the colloidal silica stabilizer, the hydraulic gradient, and the hydraulic conductivity of the liquefiable soil.

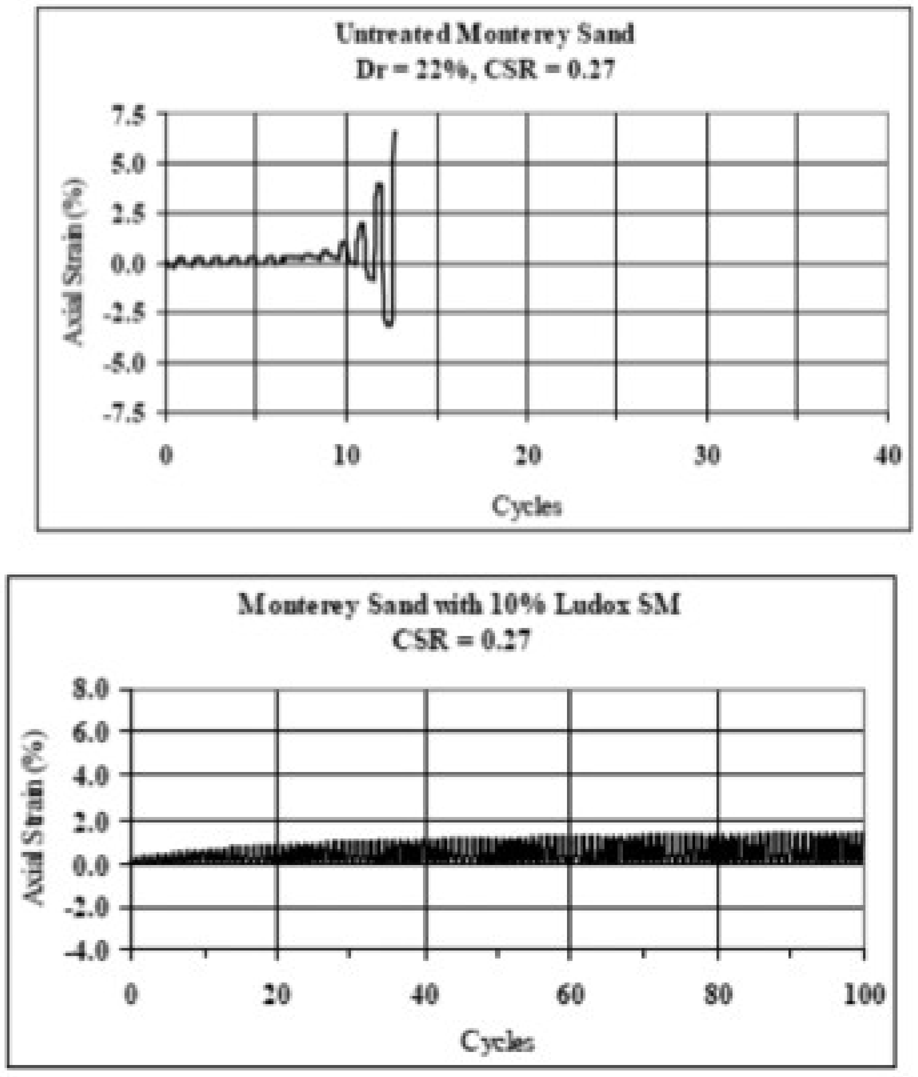

Figure 14: Comparison between cyclic behaviour of untreated sand and sand treated with 10% colloidal silica (Gallagher 2000).

8.4.2 Biogeochemical Remediation

Over the past decade, new biogeochemical techniques have been explored for the improvement of ground properties. DeJong et al (2011) summarise some of these techniques, and at least two of these have the potential to improve the liquefaction resistance of soils. The first of these involves the use of microbes to induce calcite precipitation between soil particles. The second involves the use of microbes to generate small gas bubbles within the soil and thus increase the resistance to liquefaction.

Microbially Induced Calcite Precipitation (MICP) occurs through a variety of microbiological processes, such as urea hydrolysis and denitrification. The process of calcite precipitation appears to have been discussed initially by van Meurs et al (2006), using the terms "smart soils" and "biogrout". In Biogrout, specific bacteria create a reaction which results in the precipitation of calcium carbonate in the form of crystalline calcite on the surface of the sandy particles (see Figure 15). When this reaction occurs under the right conditions, crystals are formed between two adjacent particles, producing a connection between them. This connection increases the strength and stiffness of the soil. The longer the nutrients, bacteria and reactants are present, the thicker the layer of mineralization. An important characteristic of this process of cementation is that the permeability of the porous material only reduces slightly. The biomineralization process progresses slowly in natural circumstances, but the rate of mineralization can be enhanced by stimulating the conditions.

Test data presented by Meurs et al indicated that the strength of a treated sand could be increased by a factor of almost 5, while the stiffness was increased by a factor of 3.

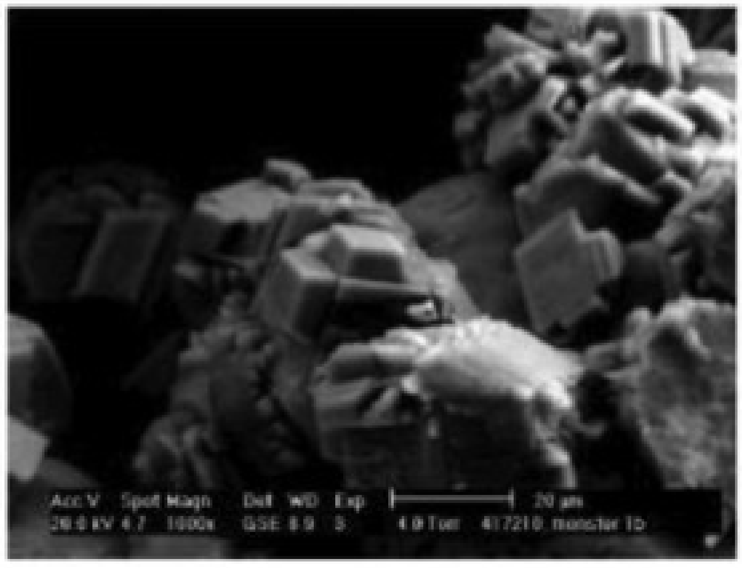

Figure 15: Scanning electron micrograph showing the mineralization of calcite onto sand grains.

DeJong et al (2010) provide a more detailed description of the process of calcite precipitation and its consequences on the geotechnical properties of the treated soil. Significant increases can be obtained in both strength and stiffness, and the volumetric behaviour can be altered from contractive to dilative, thus improving the resistance to liquefaction.

DeJong et al (2011) point out that there are a number of major challenges in implementing MICP technology. One is related to upscaling to a field scale, with issues related to cost, the stimulation of native biota, the uniformity of treatment, and the management of potentially harmful by-products. Another issue relates to the alternatives of employing bio-stimulation of native bacteria species, or the augmentation of a specific species of bacteria for a treatment zone. They suggest that bio-stimulation may be the preferred approach. A further issue may be the longevity of the treated soils, which is related to the pH of the groundwater. Tests suggest that MICP products will remain stable provided that the pH remains above 6.3.

With respect to the second technique involving microbial gas generation, Chu et al (2011) discuss some types of microorganisms that may contribute to biogas effects. This latter approach, sometimes referred to as the "Induced Partial Saturation" technique, involves the injection of non-hazardous chemicals into the ground to create gas bubbles and hence to reduce saturation. This in turn reduces the potential for liquefaction. This again has the potential to be used to improve liquefaction resistance below existing structures and buildings.

Updated information on bio- and chemo-mechanical processes in soils is provided in Geotechnique (2013).

9. CONCLUSIONS

This paper has provided a review of practical methods of assessing the effects of seismic events on the behaviour of piles, and thereby, a means of designing the piles to resist seismically-induced actions. Consideration has been given to both axial and lateral effects on piles, and for cases involving no liquefaction and also where liquefaction is assessed to occur.

In cases where liquefaction may occur, consideration needs to be given to the following issues:

1. Temporary loss of axial load capacity;
2. The possibility of axial buckling of the pile;
3. Bending moments and shears developed in the pile due to both inertial and kinematic loadings;
4. The possible beneficial effects of group action in reducing kinematically-induced bending moments.

It has been demonstrated that bending moments are likely to be a maximum at or near soil layer interfaces where there is a significant difference in the stiffness of the layers. An important case is at the junction between liquefied and non-liquefied layers. A further consequence of this behaviour is that large bending moments may occur well below the pile head, where it has been customary in the past to reduce or eliminate reinforcement in concrete piles because the moments due to inertial loads have become small. Unfortunately, in cases of earthquake loading, the kinematically-induced moments may be substantial at depth if the soil is layered and the stiffness of adjacent layers differs markedly.

There are a number of methods available for mitigating the effects of liquefaction. While some of the more traditional methods may be suitable for green-field sites, they may not be feasible for existing sites. In such cases, some of the more recent innovative methods involving bio-geo-chemical processes hold considerable promise, provided that they can be demonstrated to be economically feasible at field scale.

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Compaction versus consolidation in reinforced soil structures

THE DIFFERENCE BETWEEN compaction and consolidation is usually taught in first year studies and would be second nature to most engineers, so you might think further discussion is not required. Think again. Recent experiences in the market have shown me that the practical application of these two principles to reinforced soil structures is not as clear as one might think.

Compaction can be described as packing together soil particles at the optimum moisture content, as reported by the lab, to reduce air voids and arrange the particles in the densest arrangement possible. Consolidation being the gradual densification over time of a soil mass as a result of the expulsion by pressure of the pore water. Therefore effective compaction will reduce or remove consolidation.

Post construction consolidation in a reinforced soil slope or wall affects the structure in a number of ways, some more obvious than others. Firstly, consolidation can reduce the final grade level at the top of the wall over time. For applications where a structure is supported such as a bridge or building, this is critical and the designer will insist on granular material and close construction monitoring. Likewise for a wall with a rigid facing such as modular blocks or concrete panels, settlement of the fill can place unacceptable loads on the reinforcement as the reinforced mass moves in relation to the facing. However in some applications the possibility of some settlement may be acceptable to the client. Clearly a marginal fill consisting of some cohesive material will be more sensitive to moisture, and more challenging to compact properly on site, especially if the weather is not cooperating. If allowing the use of these materials we may have to practically assume that some settlement may occur, and manage compaction and client expectations accordingly. Therefore rigid face systems will require a granular fill, but for soft face systems I have worked on projects where the effect of settlement was considered acceptable given the substantial savings achieved from using readily available site fill, and the design engineer must manage this for the best needs of their client.

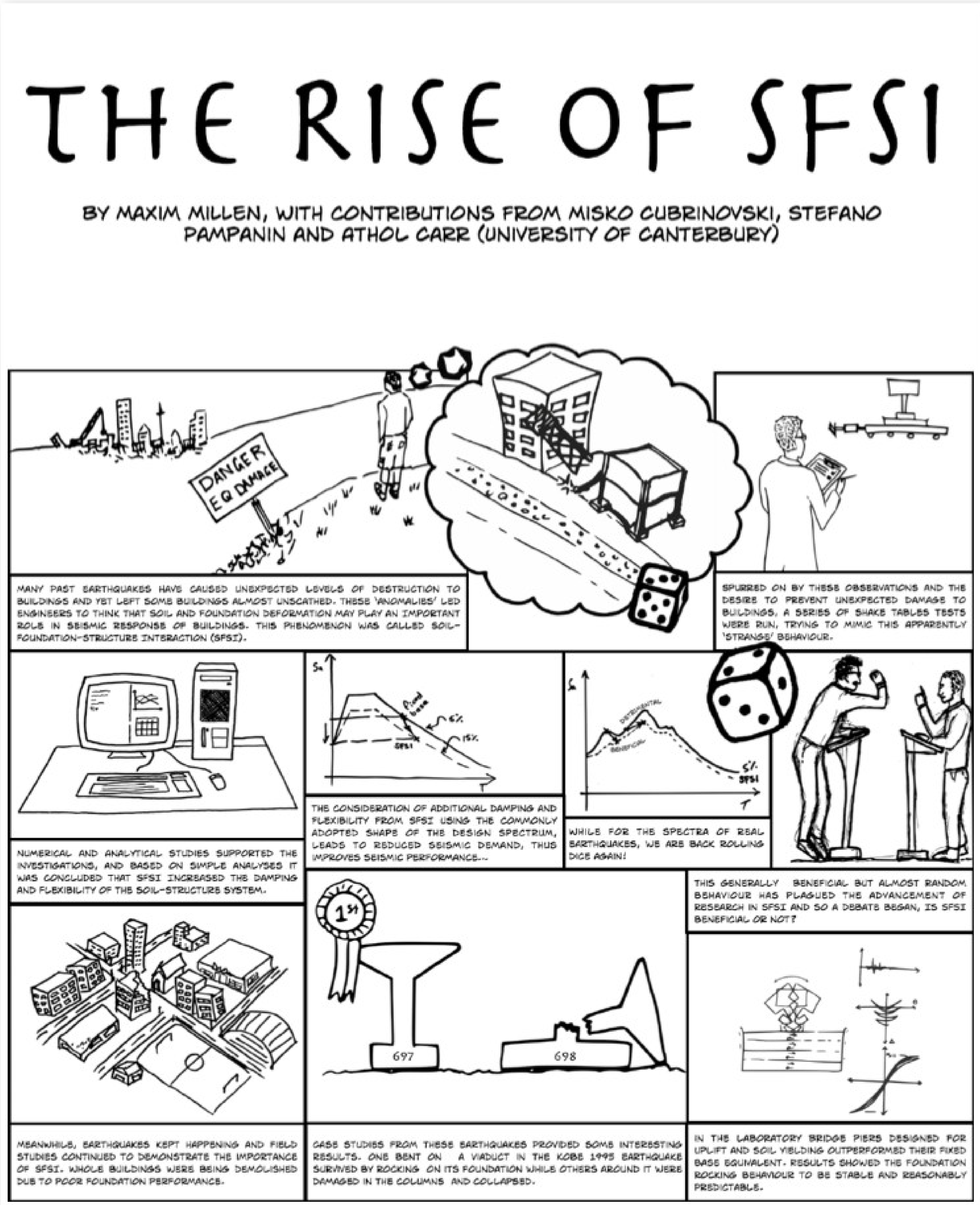
Where I have seen a lot of confusion is around deformation of the face in steel mesh faced systems. These systems which are very popular for good reason use the steel mesh as a facing to form the required angle of the wall during construction. The mesh must therefore be strong enough to withstand the forces of compaction, and any reputable supplier will give construction guidelines detailing the need to keep heavy equipment away from the face area to manage this process. There is certainly more than one suitable system available and I have been privileged to work on many of these projects and can confirm their effectiveness. However if the fill is not compacted correctly and settles over time, the load applied to the lower layers from the soil surcharge is immense, and can cause even a very rigid panel to deform. This is where the challenge arises, what has caused the face to bulge, and how could it be avoided. Regrettably there has been a tendency to promulgate photos of post construction settlement and infer the cause as construction damage, which is not only naive but may bring the industry into disrepute and potentially denigrate what is an excellent construction method. A simple indication for this is to check if there is void behind the deformed panel, indicating post construction settlement, or if the fill is compacted into the deformed section, indicating construction movement.

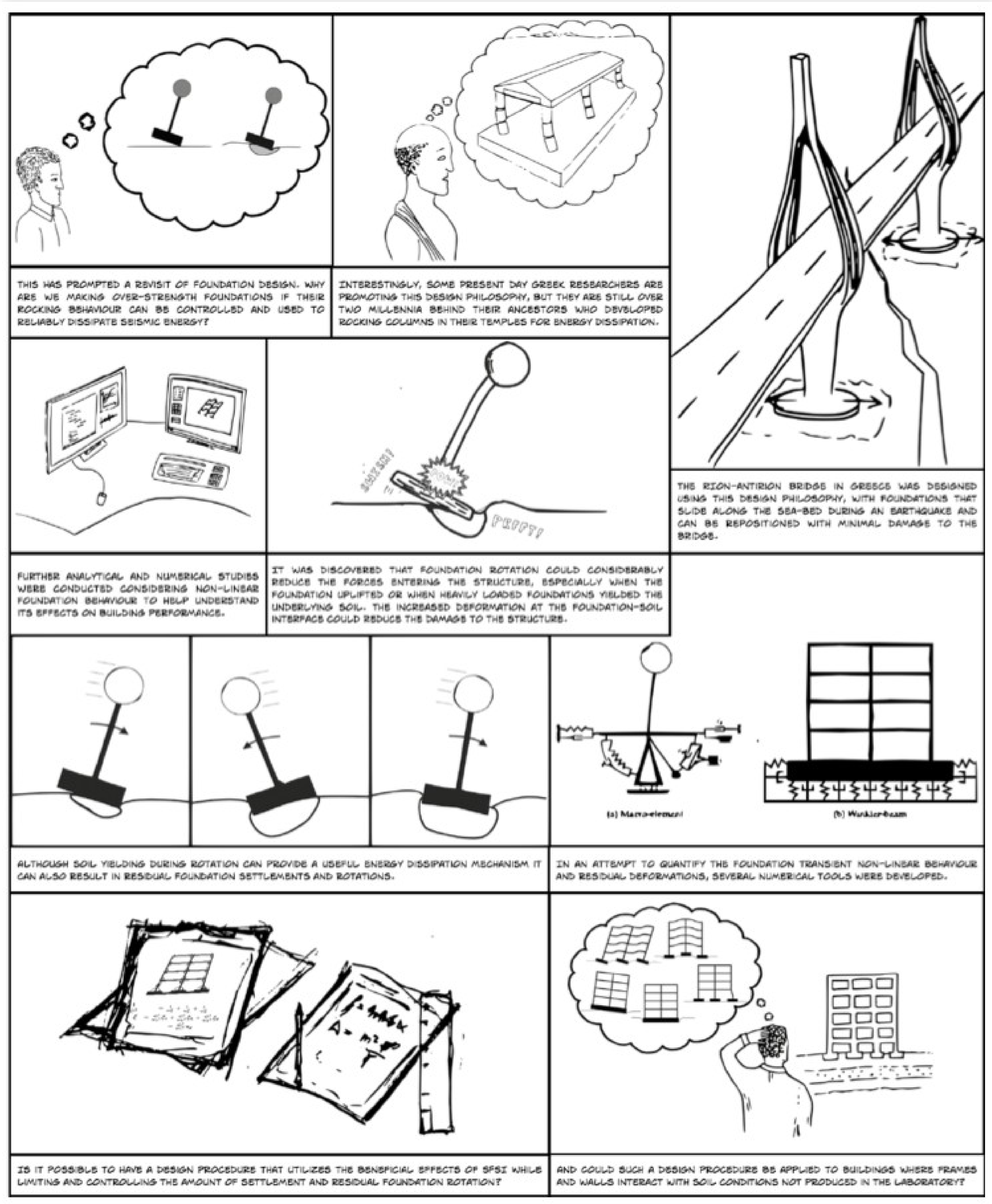
In summary, MSE Walls including mesh face systems are long established globally as a viable and efficient engineering structure. If best practice is followed and a suitable steel mesh system with preformed brackets is used there is no reason for face bulging to occur as a result of construction. If you want information on a system, ask the supplier for irrefutable evidence and case study, rather than relying on vague innuendo supplied by someone with an ulterior motive.

Michael K Sorensen

Michael is the Technical Support team leader at Cirtex and specialises in Geosynthetic applications, with a specific interest in polymeric reinforcement and earth retaining structures. Michael is passionate about increasing the knowledge of Geosynthetics among the engineering community. He works with engineers and contractors throughout New Zealand and the South Pacific to assist them with applying Geosynthetics to their projects in the most efficient and cost effective way.

Soil Foundation Structure Interaction – Maxim Millen

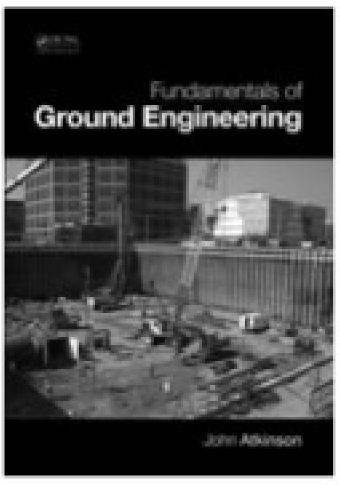



THIS HAS PROMPTED A REVISIT OF FOUNDATION DESIGN. WHY ARE WE MAKING OVER-STRENGTH FOUNDATIONS IF THEIR ROCKING BEHAVIOUR CAN BE CONTROLLED AND USED TO RELIABLY DISSIPATE SEISMIC ENERGY?
INTERESTINGLY, SOME PRESENT DAY GREEK RESEARCHERS ARE PROMOTING THIS DESIGN PHILOSOPHY, BUT THEY ARE STILL OVER TWO MILLENNIA BEHIND THEIR ANCESTORS WHO DEVELOPED ROCKING COLUMNS IN THEIR TEMPLES FOR ENERGY DISSIPATION.
THE RION-ANTIRION BRIDGE IN GREECE WAS DESIGNED USING THIS DESIGN PHILOSOPHY, WITH FOUNDATIONS THAT SLIDE ALONG THE SEA-BED DURING AN EARTHQUAKE AND CAN BE REPOSITIONED WITH MINIMAL DAMAGE TO THE BRIDGE.
FURTHER ANALYTICAL AND NUMERICAL STUDIES WERE CONDUCTED CONSIDERING NON-LINEAR FOUNDATION BEHAVIOUR TO HELP UNDERSTAND ITS EFFECTS ON BUILDING PERFORMANCE.
IT WAS DISCOVERED THAT FOUNDATION ROTATION COULD CONSIDERABLY REDUCE THE FORCES ENTERING THE STRUCTURE, ESPECIALLY WHEN THE FOUNDATION UPLIFTED OR WHEN HEAVILY LOADED FOUNDATIONS YIELDED THE UNDERLYING SOIL. THE INCREASED DEFORMATION AT THE FOUNDATION-SOIL INTERFACE COULD REDUCE THE DAMAGE TO THE STRUCTURE.
(a) Macro-element
(b) Winkler-beam
ALTHOUGH SOIL YIELDING DURING ROTATION CAN PROVIDE A USEFUL ENERGY DISSIPATION MECHANISM IT CAN ALSO RESULT IN RESIDUAL FOUNDATION SETTLEMENTS AND ROTATIONS.
IN AN ATTEMPT TO QUANTIFY THE FOUNDATION TRANSIENT NON-LINEAR BEHAVIOUR AND RESIDUAL DEFORMATIONS, SEVERAL NUMERICAL TOOLS WERE DEVELOPED.
IS IT POSSIBLE TO HAVE A DESIGN PROCEDURE THAT UTILIZES THE BENEFICIAL EFFECTS OF SFSI WHILE LIMITING AND CONTROLLING THE AMOUNT OF SETTLEMENT AND RESIDUAL FOUNDATION ROTATION?
AND COULD SUCH A DESIGN PROCEDURE BE APPLIED TO BUILDINGS WHERE FRAMES AND WALLS INTERACT WITH SOIL CONDITIONS NOT PRODUCED IN THE LABORATORY?

Fundamentals of Ground Engineering

– John Atkinson

Review by:
Rolando Orense
Rolando is an Associate Professor in Civil & Environmental Engineering at the University of Auckland, specialising in earthquake geotechnical engineering and assessment of liquefaction potential of soils. Rolando has spent 18 years researching, teaching and consulting in Japan on topics relating to soil liquefaction, ground response analyses and soil-structure interaction during earthquakes.



WHEN I WAS approached to review two recently published books dealing on core concepts of soil mechanics, I accepted the challenge. I thought that reading them would provide me with different insights on how to teach Geomechanics 1 paper to 2nd year university students.

The first book, the Fundamentals of Ground Engineering by John Atkinson, is not the conventional off-the-shelf geotechnical book that you will find in the library. With its bite-sized note-type explanation (one or two pages per topic), it is a concise and easily readable summary of important concepts in soil mechanics and ground engineering. It starts off with well-explained simplified presentation of ground engineering as a whole, and the complementary roles geology and engineering play in the whole exercise.

Presented as a guide rather than a textbook, the book contains compact summaries of the basic ground engineering principles, such as geological and ground investigation, and physics and mechanics, and then applies these principles to the fundamentals of soil mechanics and geotechnical engineering. It covers all key concepts, including soil classifications, material behaviour, seepage, soil strength and deformation, slope stability and foundations. It also covers state parameters, Cam clay and numerical modelling as well as unsaturated and problematic soils. One need not start from the beginning to use this book. At the end of each chapter there are suggestions for things to do to illustrate the principles discussed in the chapter – suggestions which can be routinely done without the use of special equipment. There are also further readings suggested, but unlike ordinary textbook with list of references, this book has none. Indeed, who needs a reference list these days when one can easily google the topic! Here, the reader is advised to go to a book (or books) that deals with the topic in more detail.

The book is structured along its well-illustrated diagrams and concentrated data to assist in explaining the key concepts. There are many photographs used to illustrate the general features of geotechnical engineering practice, although I would have appreciated it more if they are in colour, since the black and white presentation does not give justice to the exciting world of ground engineering. The last chapter sets it apart from other conventional textbooks because it adequately addressed and explained the principles of geotechnical design.

The book is not a collection of lecture notes nor a design guide. It is a comprehensive summary of the fundamental theories and analysis that underpin all geotechnical engineering. It is always a delight to read a book written by someone who knows what he is writing about. The presentation is simple that even first year undergraduate students can understand, and yet so complete that postgraduate students will find it helpful. It is a handy resource book for practicing engineers and geologists, one that I would surely like to have in my own bookshelf.

Title	Fundamentals of Ground Engineering
Author	John Atkinson
Publisher	CRC Press
Year Published	2014
Hardback	242 pp
ISBN	9781482206173
Web shopping	http://www.crcpress.com/product/isbn/9781482206173
Price	GBP 15-20 (paperback)

Core Principles of Soil Mechanics – Sanjay Kumar Shukla

IN COMPARISON, THE book Core Principles of Soil Mechanics by Sanjay Sukla is a conventional textbook on soil mechanics. It covers the key concepts of soil mechanics, such as physical properties of soils and their behaviour when subjected to loads, seepage, consolidation, shear strength, lateral earth pressure, slope stability and bearing capacity. The concepts are presented in a simple manner, with many worked examples. One of the things I like about the presentation is that each chapter starts with key learning aims, and within the chapter major revision points are highlighted. Finally, each chapter ends with summaries of the key points and further reading suggestions. It has good drawings and diagrams to illustrate theory as well as worked examples and exercises for the students to practise.

The book is primarily designed and developed to cover core concepts in soil mechanics for a one-term course in undergraduate programme; indeed, it is a collection of lecture notes used by the author. Therefore, it is evident that the presentation is brief and to the point. While the book is probably pitched about right for the intended target audience, it suffers slightly in places in terms of the order in which information is presented, for example, the section on phase relationships coming ahead of that on soil formation; this makes me wonder if undergraduate students would grasp the three-phase medium concept of soil without them knowing how soils are formed. There are instances where some key terms are mentioned, but their definitions are provided either later in the section or not at all. For example, the SPT N-values are mentioned in relation to soil consistency and drained friction angle, but there is no explanation of their physical meaning (although there is a reference to the ASTM standard).

As in any set of lecture notes (my own included), it is difficult to expand on certain topics without deviating from being simple and concise. In the book, there are some confusing statements (e.g. dewatering and saturating soils are both listed as densification methods) which need further explanations to fully understand. The section on time rate of consolidation does not provide the usual tables and charts for estimating degree of consolidation within a clay profile; is the intention to have the students use the long equations (in summation form) when analysing consolidation problems? I also winced when I read the explanation of liquefaction phenomenon which is discussed with no mention of undrained condition. At least, a specialist book is cited for the complete description of liquefaction.

The readability could also be improved if properly copy-edited, as there are some typographical errors and wrong notations in a couple of equations. In some instances, the author adopts a first-person stance, as if doing a lecture in front of students.

The book is only 200 pages long, and it is indeed a challenge to squeeze all key soil mechanics concepts within such a limited space. Thus, even in teaching a one-term Geomechanics course, I would still prefer the more conventional and yet more complete soil mechanics textbooks available. I can then skip some sections, but the relevant information would still be there for the students to peruse.

Title	Core Principles of Soil Mechanics
Author	Sanjay Kumar Shukla
Publisher	ICE Publishing
Year Published	2014
Hardback	288 pp
ISBN	9780727758477
Web shopping	http://www.icevirtuallibrary.com/content/book/102887
Price	GBP 30 (paperback)

10th ANZ Young Geotechnical Professionals Conference

THE 10TH ANZ Young Geotechnical Professionals Conference (10YGPC) was held in Noosa, Queensland from 9 – 12 September 2014. The YGP conference has been held over the past 20 years for geotechnical professionals from Australia and New Zealand who are 35 years and younger with a maximum of 10 years' experience. The aims of the conference are to:

- Promote the professional development of delegates through sharing experience and ideas, and by presenting a paper to senior professionals and peers
- Expand and strengthen the lines of communications between young professionals within the field of geomechanics
- Promote an enhanced perspective of the varied roles, responsibilities and opportunities encompassed by the geotechnical profession

The three day conference provided a great range of opportunities for delegates to network and discuss their professional careers. The atmosphere was informal but the calibre and quality of the presentations was outstanding. The feedback from the delegates was that it was a very interesting and engaging conference and a brilliant way to meet fellow young geotechnical engineers and engineering geologists. The conference consisted of two days of presentations, a conference dinner (where we are happy to inform you that the NZ contingent took out first and second places in the NZ Vs Australia quiz) and concluded with a field trip taking in a large roading project north of Noosa and finishing in the Glasshouse Mountains. Thanks to conference organiser David Lacey for organising such a successful event, conference sponsors and to all those especially the paper reviewers who contributed behind the scenes.

The senior mentors at this year's conference were Tony Fairclough (NZGS Committee Member – Tonkin and Taylor) along with Allan McConnell (Insitu Geotech Services) and Ben Collingwood of Foundations Specialists Group from Australia. All three presentations were interesting and shared similar themes. Some of the key ideas shared by the mentors were:

- Never say no until you know it's the right answer. Best learnings come from undesirable projects
- Be thirsty for knowledge, every project is a learning opportunity
- Be honest, diplomatic and always act ethically and with integrity
- Don't be conservative without good reason. Investigate and understand the problem, get timely input and help from the right people
- Make it a priority to have quality mentors throughout your career.
- Take time to understand what you know, what you think you know, and what you don't know – this is very important for risk management
- Get out of the office and onto site – see as much as possible! One day is a long time. And there is enormous value in getting to know the guys onsite
- Get involved with NZGS or AGS! Ben was the chair of the 3YGPC in 1997 and Tony attended the 4YGPC in Perth

EQC and NZGS have consistently supported our YGP members with financial sponsorship to attend this conference. A total sum of $10,000 was distributed amongst delegates who were awarded the YGP Conference Award based on their submitted abstracts and circumstances. The support of EQC and NZGS was greatly appreciated by all recipients of the award and the chance to attend an international conference such is this was invaluable. Below are comments sent through from a

Frances Neeson

Frances is an Engineering Geologist with Opus in Christchurch. She holds a Bachelor of Science (Geology) and Post Graduate Diploma in Engineering Geology. Over the last five years Frances has enjoyed working on small and large projects in the North and South Islands including numerous infrastructure projects, earthquake remediation and the Ferrymead Bridge Replacement. Frances is excited to be able to represent the growing numbers of YGP members of NZGS and promote YGP orientated activities!

Frances.Neeson@opus.co.nz

few of the award winners about the conference as well as a photo of all of the New Zealand delegates that attended:

The conference was a fantastic experience. We really enjoyed finding out what other young geotechnical professionals are involved with and what problems they are thinking about. There was a lot of food for thought, in particular the NZGS winning presentation that demonstrated the need for a good understanding of the model and assumptions used in geotechnical analysis. The relaxed, informal environment was a great introduction to professional conferences and now that we have seen the value in participation and the sharing of ideas we will both be looking to attend more geotechnical conferences in the future. Thanks NZGS! **Emilia Belczyk and Holly Le Heux**

I thoroughly enjoyed the YGP conference with a great venue just up from the beach, a good bunch of young friendly Geo's to spend a few days with, the chance to see what other people have been doing and learning and a few days out of the normal work routine. The chance to attend the conference was well worth the effort of preparing and presenting a paper and I would recommend future YGP conferences to anyone who has the chance to attend. **David Buxton**

PHOTOS COURTESY OF EMILIA BELCZYK

Above: The New Zealand 10YGPC contingent at the Glasshouse Mountains

The sand and surf of Australia's Sunshine Coast provided the backdrop for a fantastic YGP conference. For me, the highlight of 10YGPC in Noosa was the opportunity to connect with fellow New Zealand practitioners and our Australian counterparts – both from a technical and (more importantly!) social point of view. The conference opened each of us up to learning about fellow delegates' experience base, involvement in projects and challenges that have been overcome. This was complemented by the mentor presentations that provided us with some choice 'take home' messages. I extend my gratitude to Beca and to the NZ Geotechnical Society for their financial assistance which allowed me to attend. And finally, all the delegates I'm sure will join me in thanking David Lacey for single-handedly organising and delivering this excellent conference. **Martin Barrientos**

THE 11TH ANZ YGPC WILL BE HELD IN NEW ZEALAND IN 2016 – SO KEEP WATCHING THIS SPACE!

NZGS YGP FELLOWSHIP

The YGP Fellowship is awarded to the author of the best paper by a New Zealand representative at each YGP Conference. The award is judged at the conference by the senior geotechnical professionals in attendance (mentors) and announced at the closing ceremony.

This year we are pleased to announce that the winner of the NZGS YGP Fellowship was David Buxton from Tonkin and Taylor. His paper was entitled "Hambly's Paradox and the practising Geotechnical Engineer" and is included in the digital issue of Geomechanics News. The fellowship provides funding of up to $4000 for David to go to the next International YGP Conference, which will be held in Seoul, Korea in 2017. Congratulations also go to Elby Tang as the runner-up with her paper entitled Design Configurations of Stone Columns as a mitigation measure against Liquefaction-Induced Lateral Spreading". A big congratulation goes out to both David and Elby.

Left: NZGS Mentor Tony Fairclough, YGP Fellowship winner David Buxton and Runner Up Elby Tang. Glasshouse Mountains

WINNER

Proceedings of the 10th ANZ Young Geotechnical Professional's Conference – 10YGPC
Noosa, Queensland, Australia
10 – 12 September 2014

HAMBLY'S PARADOX AND THE PRACTISING GEOTECHNICAL ENGINEER

David BUXTON
Geotechnical Engineer, Tonkin & Taylor Ltd, Auckland, New Zealand

ABSTRACT – This paper presents four case studies of simple lateral soil loading problems to assess how intuitive the results might be to a young geotechnical professional starting their career. In particular the paper examines if the level of robustness implied by a load factor, partial factor or factor of safety may be misunderstood. It is demonstrated that for the results to be intuitively understood the designer needs to understand the methods and assumptions behind the analysis and the limitations this brings to the results.

The examples are inspired by Hambly's Paradox which comprises a simple example comparing three- and four-legged chairs and eloquently demonstrates the concepts of ductility, robustness and redundancy. The examples demonstrate of how studies of simplified examples can be used to build understanding of an analysis, assess sensitivity of the results and develop the intuition of a young geotechnical professional.

1. Introduction

Lateral soil loads are a fundamental element of geotechnical engineering with the basis for their assessment having been understood for over two centuries. Assessment of active and passive earth pressures using both the Coulomb theory (Coulomb, 1776) and the Rankine theory (Rankine, 1857) remain a basic and key part of the current education and practice of the young geotechnical engineer.

Given the long established earth pressure theory one might expect a graduate geotechnical engineer to be intuitively able to apply active earth pressures. However, John Burland in the "Structural and geotechnical modelling" chapter of the ICE Manual of Geotechnical Engineering (Burland et. al., 2012) presents a simple structural example, Hambly's Paradox (Hambly, 1985). Whilst Hambly's Paradox (or the three- and four-legged stools) is a very simple example, the results are only intuitive if the subtlety of the problem is understood and applied correctly.

Burland's presentation of Hambly's Paradox (summarised below) prompted the author to investigate four case studies of lateral soil loading problems to assess how intuitive the results might be to a young geotechnical professional starting their career. In particular the paper examines if the level of robustness implied by a load factor, partial factor or factor of safety may be misunderstood.

1.1. Hambly's Paradox

Hambly's Paradox (Hambly, 1985) considers how much load is carried by each leg of three-legged and four-legged stools depicted under a central 60kg load, as shown in Figure 1. The three-legged stool is straightforward with 20kg load in each leg. With the four-legged stool 15kg is wrong, one leg does not quite touch the ground and to maintain equilibrium the total load will be carried by two legs with 30kg each.

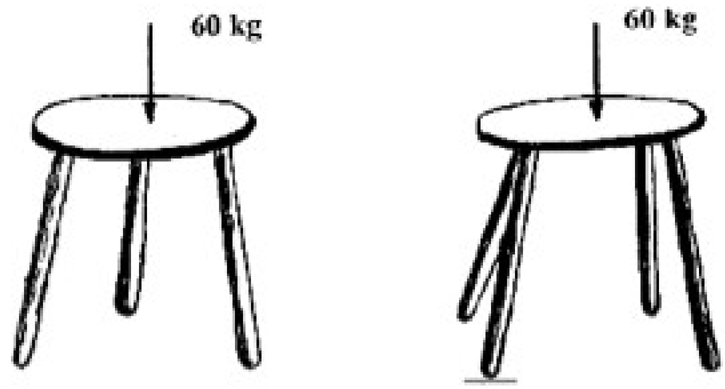


Figure 1. Three- and four-legged stools – Hambly's paradox

The paradox – that the addition of a leg can increase rather than decrease the load carried, can be used to illustrate a number of concepts. In particular the paradox emphasises the importance of ductility, robustness and redundancy. The reader should refer to the "Structural and geotechnical modelling" chapter of the ICE Manual of Geotechnical Engineering for a discussion of these concepts. The aspect of the paradox that prompted this paper was that the answer is not intuitive unless the subtlety of the problem is understood and applied correctly.

1.2. Limitations

In this paper the author does not seek to fully evaluate the field of lateral soil loading but to examine a series of specific situations.

It is noted that the results resented are specific to the geometry and parameters adopted in the case study. The author has not attempted to generalise the results in a way that they can be applied to a wider range of situations. The examples do not include wall friction.

2. Case Study A – Determining Active Earth Pressures using Limit Equilibrium Analysis

Case Study A considers if assessing and applying active earth pressures using limit equilibrium analysis is intuitive. Using limit equilibrium analysis programmes to evaluate earth pressures can be convenient to incorporate the effect of irregular back slopes, surcharges and soil layering or purely as an easy check of another analysis type. The example is extended to consider if a Factor of Safety (FoS) in the limit equilibrium analysis is equivalent to a load factor applied to the active force.

The example comprises a series of analyses in which a vertical face of a single dry soil unit has a horizontal point load equal to the theoretical active earth pressure, with or without a load factor, applied at one-third of the face height. The analyses use the Morgenstern-Price limit equilibrium method in Slope/W with a series of trial surfaces determined by the entry and exit option. The location of the critical slip surface was automatically optimised.

The following range of parameters was considered:

Face Height = 5m*
ϕ' = 30° or 35°
γ = 18 kN/m^3
c' = 0 kPa
Load Factor = 1.0 & 1.5 (for ϕ' = 30°)
= 1.0, 1.3, 1.5, 2.0 and 3.0 (for ϕ' = 35°)

[*The principal evaluated here does not appear to be sensitive to face height]

The analysis is illustrated in Figure 2.

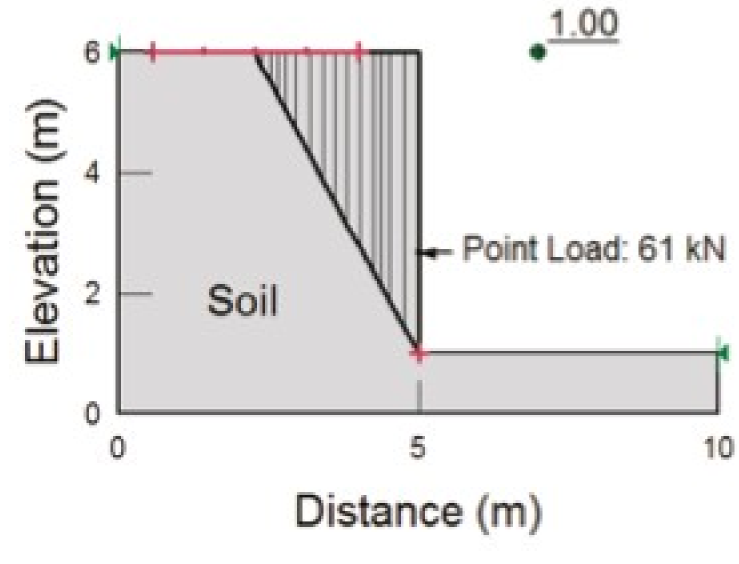


Figure 2. Example of a Case Study A Analysis

2.1. Case Study A - Results

The results of Case Study A are summarised in Table 1.

Table 1. Case Study A Results

ϕ'	Load Factor	Factor of Safety
30°	1.0	1.01
	1.5	1.65
35°	1.0	1.00
	1.3	1.30
	1.5	1.51
	2.0	2.26
	3.0	6.78

2.2. Case Study A - Discussion

Case Study A confirms that active earth pressures (as a force) can be determined using limit equilibrium methods by assessing the force that results in a FoS of 1.0. At a FoS of 1.0 the analysis is in a true equilibrium state equivalent to the active limit state. In this case the answer should be intuitive however several situations can be envisaged where a designer could misunderstand the analysis:

1. The designer may default to a target FoS greater than 1.0 that is typical for demonstrating adequate stability, resulting in an excessively conservative earth pressure.
2. The analysis is determining the active pressure, the designer still needs to separately consider if there will be sufficient displacement to mobilise the active case.

The second part of the example considers if a load factor and FoS are equivalent for this situation and the results are contradictory and therefore not intuitive to someone without understanding of the analysis method. For example in the 35° friction angle case the load factor and FoS are equivalent up to a value of 1.5. At a load factor of 2.0 and 3.0 respectively the FoS increases to 2.26 and 6.78 respectively. For the 30° friction angle case only a load factor of 1.5 is considered however this results in FoS of 1.65.

These results can be understood with a simple conceptual understanding of how the applied force is being incorporated into the limit equilibrium equations, as shown in Equation 1 below (note the point load force will be negative due to its direction):

$$FoS = \frac{F_{Resisting}}{F_{Driving} + F_{Point\ Load}} \quad (1)$$

As the point load approaches the value of the driving force the bottom line of Equation 1 approaches zero and the resulting FoS will tend to infinity. The bottom line will be zero when the point load is equal to the at-rest condition. For a 30° friction angle the at-rest earth pressure is 1.5 times the active earth pressure, hence the FoS of the result with a load factor of 1.5 is infinity for the theoretical active wedge. The analysis still returns a

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result by finding a slip surface encompassing a larger mass with accordingly larger driving and resisting forces in which the modelled force does not result in a zero or negative bottom line in Equation 1. This results in a FoS greater than the load factor.

The ratio of the active to at-rest earth pressure coefficients ranges from 1.34 to 1.64 for soil friction angles of 20° to 40° respectively with no backslope.

3. Case Study B – Retaining Walls with an Inclined Face

Case Study B considers a retaining wall with the face inclined at an angle less than vertical, as shown in Figure 3. It is intuitive that the active force will be less than the case of a vertical wall. It is less intuitive if the level of robustness imparted by a load factor and a partial factor, applied separately to the same results, is equivalent despite the factor being numerically equivalent.

Case Study B comprises limit equilibrium analyses of a 5m high retaining wall with a variety of face inclinations. The following analyses have been undertaken for each face inclination:

Analysis 1. The active force has been determined (ie. the force that results in a FoS = 1.0). From this an active earth pressure coefficient was calculated.

Analysis 2. The FoS of the slope has been assessed with a load factor of 1.5 applied to the active force.

Analysis 3. A factored active force has been determined by both applying a partial factor of 1.5 to the tangent of the soil friction angle in the analysis and applying a load factor of 1.5 directly to the active force results.

The analyses were undertaken using a single dry soil unit and a single horizontal point load applied at one third of the face height. The analyses use the Morgenstern-Price method with a series of trial surfaces determined by the entry and exit option. The location of the critical slip surface was optimised. An example of the analyses is shown in Figure 3.

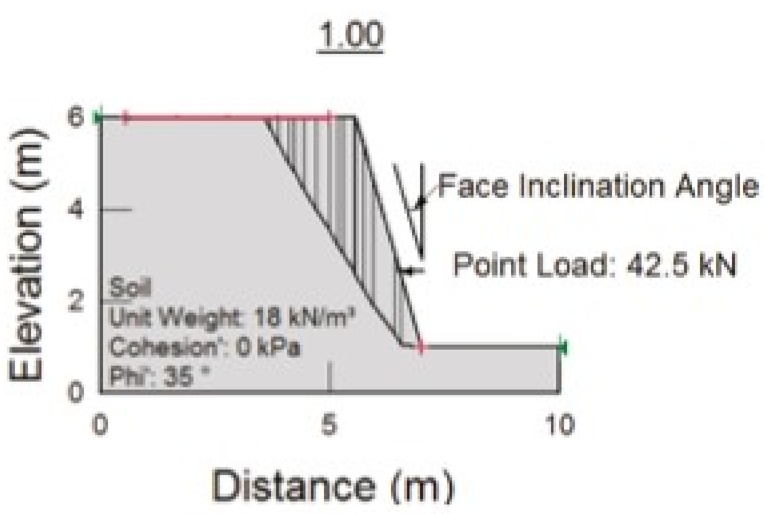

Figure 3. Example of Case Study B Analysis

3.1. Case Study B - Results

The results of Case Study B are summarised in Figures 4 to 6.

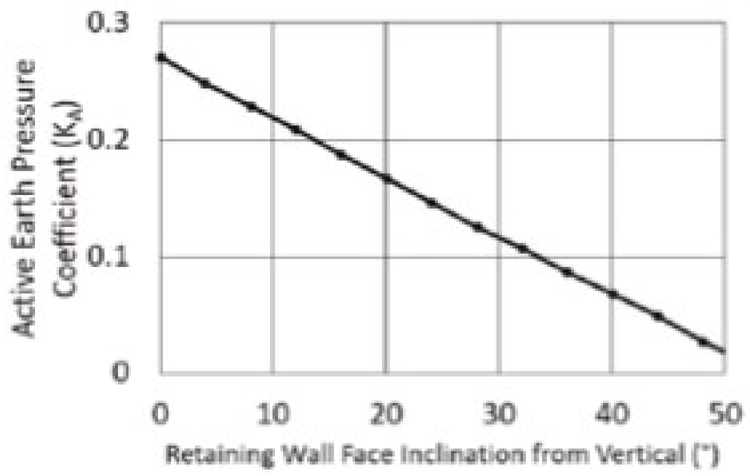

Figure 4. Plot of K_A vs face inclination (Analysis 1 results)

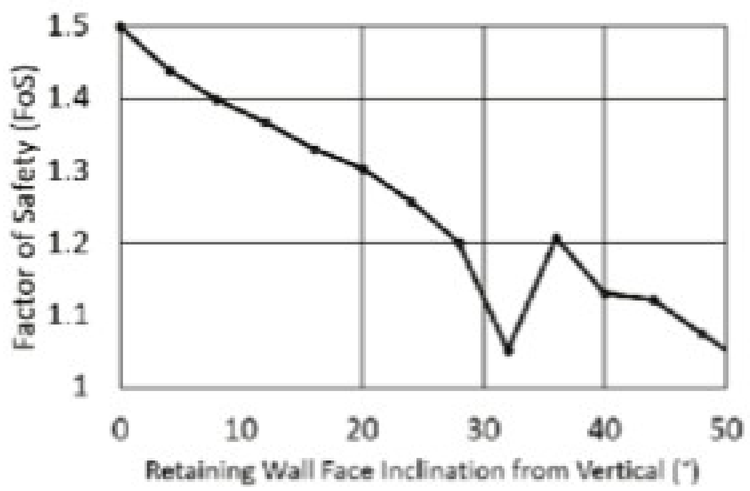

Figure 5. Plot of FoS vs face inclination for a load factor of 1.5 (Analysis 2 results)

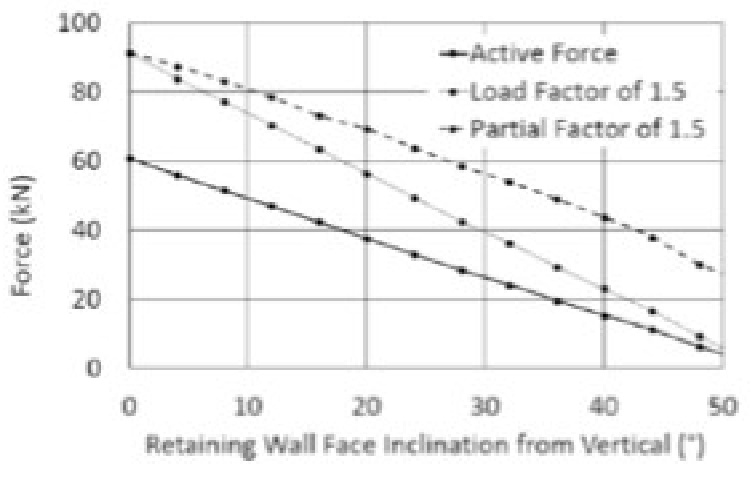

Figure 6. Plot of force vs face inclination for factored and unfactored cases

3.2. Case Study B - Discussion

The results of Analysis 1 show K_A reducing with increasing face inclination, as is intuitive. The trend is linear and although not shown, K_A is nil when the slope angle (90° - face inclination) equals the friction angle.

The results of Analysis 2 (refer Figure 5 above) shows a decreasing FoS with increasing retaining wall face inclination angle, despite a constant load factor of 1.5 being applied. In this example the level of robustness provided by the load factor varies and may be less than the designer had intuitively expected. To understand this re-consider a modified form Equation 1 from Case Study A that includes the load factor (LF), as given in Equation 2 below, and remembering the point load is negative:

$$FoS = \frac{F_{Resisting}}{F_{Driving} + LF \times F_{Point\,Load}} \qquad (2)$$

As the retaining wall face inclination increases the portion of the total resistance provided by the soil strength (top line of Equation 2) increases and that provided by the active force (on the bottom line but negative) decreases. As only the active force is factored and this is providing a reducing portion of the resisting forces the FoS decreases with face inclination.

Typically most retaining walls have a face inclination of less than about 14° (1H:4V), corresponding to a FoS of 1.35 for a load factor of 1.5 (for a soil friction angle of 35°). In practice a FoS of 1.35 is still likely to be sufficient for most retaining walls.

Analysis 3 considers the comparative level of robustness provided by a load factor applied directly to the active force and a partial factor applied to the soil strength respectively. The results show that the partial factor approach is not sensitive to the relative portion of the resisting force provided by soil strength and the applied force and may therefore be the preferred approach for this situation.

4. Case Study C – The Effect of Drained Cohesion

Case Study C shows the effect that drained cohesion has on retaining wall design actions. The effect of drained cohesion on earth pressures can be very significant, however ignoring it altogether can be excessively conservative. This can result in a situation where the robustness of the design is dependent on the judgement and approach (i.e. intuition) of the designer rather than load/partial factors applied (after all, two times not much is still not much).

An example could be designing a retaining wall on a site where existing un-retained vertical cut faces of greater height exist. In this situation it may be possible to justify parameters that resulted in no soil pressures on the wall, but the resulting design may lack robustness unless it is only intended as a facing element.

Case Study C comprises three figures based on theoretical earth pressure distributions. Of particular note is Equation 3 below, which gives the depth to which the theoretical active earth pressures are negative or alternatively the height to which a vertical cut slope will be self-supporting.

$$z_0 = \frac{2c'}{\gamma\sqrt{K_a}} \qquad (3)$$

Where Z_0 is the depth of negative active pressures (m), c' is the drained cohesion (kPa), γ is the soil unit weight (kN/m^3) and K_a is the active earth pressure coefficient.

In this case study active earth pressures have been calculated for a dry soil with no wall friction and a soil unit weight (γ) of 18 kN/m^3. Where theoretical active earth pressures are negative they have been taken as zero.

4.1. Case Study C - Results

Figure 7 plots the "self-supporting height" for a range of soil friction angles and drained cohesion. The self-supporting height is the depth to which a cut slope could stand unsupported and over which the earth pressures are theoretically negative but normally taken as zero in design analysis.

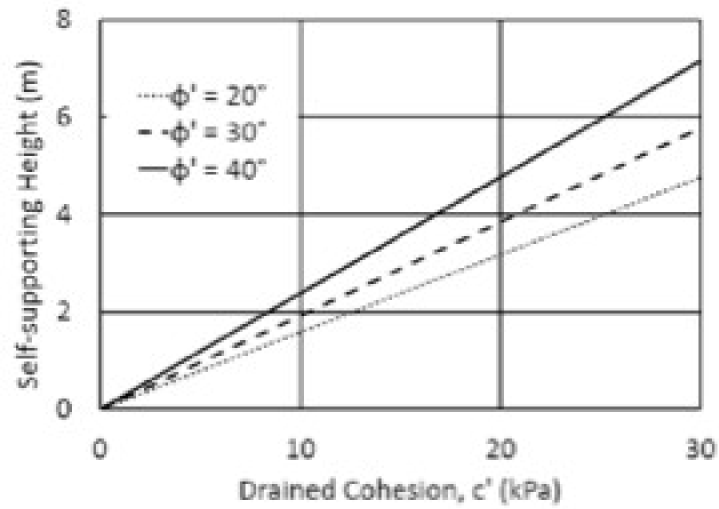

Figure 7. Plot of self-supporting height vs drained cohesion

Figure 8 plots the active earth pressure force vs drained cohesion. The results are normalised by the force with c' = 0kPa.

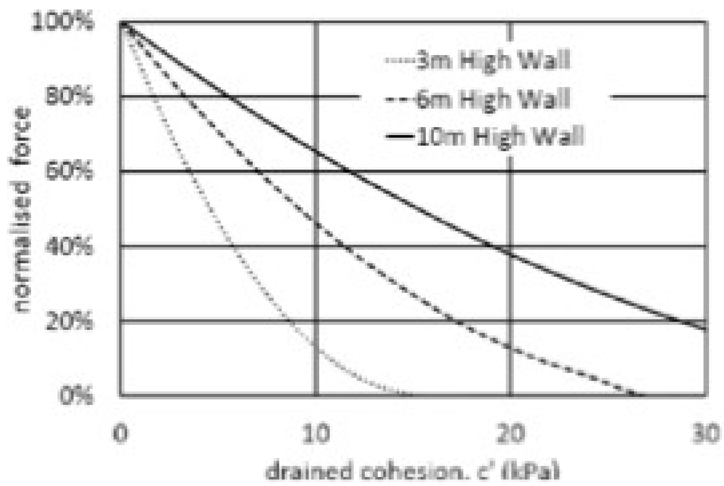

Figure 8. Normalised active force vs drained cohesion (ϕ'=30°)

Figure 9 plots the moment that results from the active earth pressure force at the passive ground

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level. The results are normalised by the moment with c' = 0kPa. Note that the maximum moment for a cantilevered retaining wall will actually be larger than the moment at the passive ground level, occurring below the passive ground level. The moment at the passive ground level is however convenient to calculate.

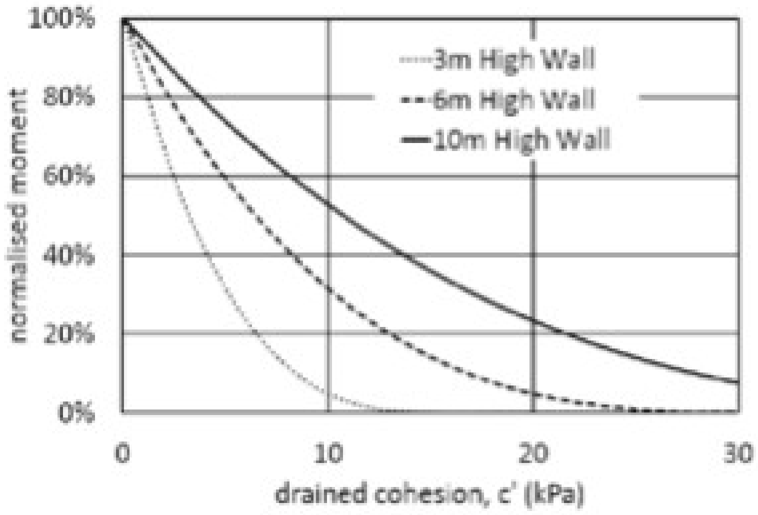

Figure 9. Normalised moment of active force at passive ground level vs drained cohesion (ϕ'=30°)

4.2. Case Study C - Discussion

Figures 7 to 9 show that drained cohesion can significantly reduce retaining wall design actions. Even a seemingly modest 5kPa can result in zero active earth pressure over the upper approximately 1m of a retaining wall, reducing the design active force by approximately 50% for a 3m high retaining wall. For the same 3m high retaining wall the normalised moment of the active force at the passive ground level is only 32%. As the magnitude of this effect can be larger than the typical magnitude of load/partial factors they alone cannot be relied upon to impart robustness into a design with drained cohesion. It is therefore key that the designer understands and treats the situation logically, through intuition from past experience or through an assessment of the model sensitivity.

A commonly adopted recommendation to ensure the design has sufficient robustness is originally found in CP2 (1951) which recommends the use of a minimum equivalent fluid pressure, with the 'equivalent fluid' having a density of 5 kN/m^3.

5. Case Study D – Determining a Lateral Load to Stabilise a Landslip

Case Study D examines a limit equilibrium analysis undertaken to assess the lateral load required to stabilise a landslip. In this example a landslip is located in a paddock adjacent to a rural dwelling. The notional design brief is to stabilise only the upper portion of the landslip, removing risk to the dwelling and reinstating the land at the crest of the slope. The limited design brief is intuitively expected to reduce the size of the retaining structure required.

The case study comprises a series of limit equilibrium analyses as follows:

Analysis 1. A back analysis of the slope is undertaken to determine material strength parameters and to achieve a FoS = 1.0 with design groundwater levels.

Analysis 2. A pile reinforcement is modelled to assess the required force to achieve a FoS > 1.3. The pile location on the slope is varied and is located near the slope crest, mid-slope and near the toe of the slope in analysis 2a, 2b and 2c respectively.

Analysis 3. Analysis 2 is repeated, however the material located downslope of the pile and above the landslip surface is removed from the model.

The analyses use the Morgenstern-Price method with fully specified slip surfaces that are optimised. The groundwater is modelled by a piezometric line and is constant in all analyses. A shear zone is modelled at 4m depth. Sample outputs of analyses are shown in Figures 10 and 11.

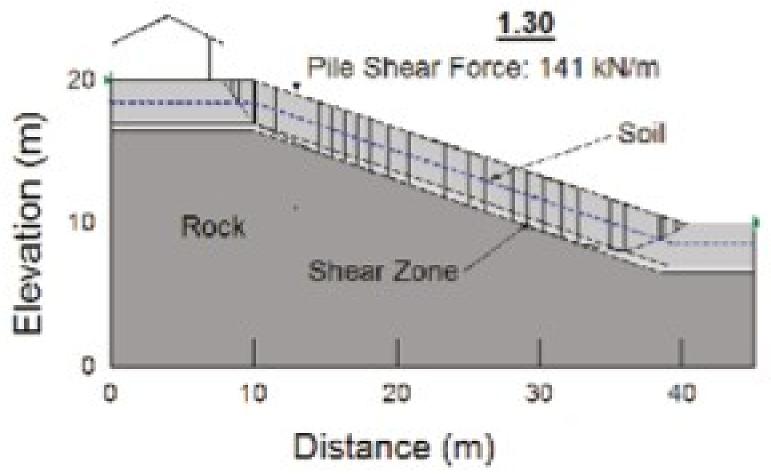

Figure 10. Analysis 2a output.

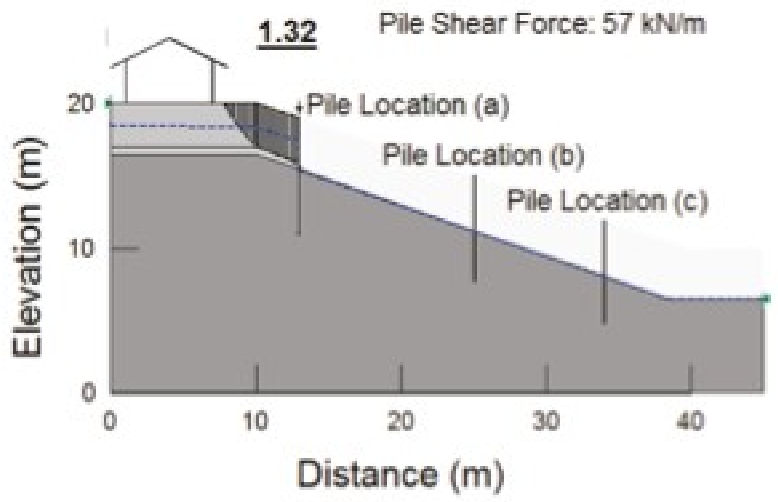

Figure 11. Analysis 3a output, with all pile locations shown.

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RUNNER-UP

DESIGN CONFIGURATIONS OF STONE COLUMNS AS A MITIGATION MEASURE AGAINST LIQUEFACTION-INDUCED LATERAL SPREADING

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ABSTRACT - Liquefaction-induced lateral spreading is a common phenomenon after strong seismic events. Typically lateral spreading occurs in sloping ground close to waterways in regions with liquefiable underlying soils and may result in significant damage and lead to significant replacement expense of existing buildings and structures. The installation of stone columns may be used to reduce liquefaction potential for foundations in level ground but there is little literature on stone columns being used to mitigate liquefaction-induced lateral spreading. This paper presents findings of a study to evaluate the effectiveness of stone columns to mitigate liquefaction-induced lateral spreading. A case study from the recent 22 February 2011 Christchurch Earthquake was used as a basis of the study and the study was carried out using effective stress analysis with the finite element software package FLAC v7.0. Current state-of-the-art design procedures for stone columns to prevent liquefaction have been used to assess its applicability to mitigate lateral spreading. The effects of lateral extent of the improvement zone and the area replacement ratio of stone columns have been investigated. Generally, it was found that stone column remediation was found to be effective in reducing the lateral displacement that was caused by liquefaction due to the seismic event in the numerical analyses. However, complementary ground improvement measures may be required to eliminate lateral displacement adjacent to the waterway.

1. Introduction

Liquefaction has been responsible for many failures of man-made and natural structures. Liquefaction-induced lateral spreading has been commonly documented after strong seismic events such as the sequence of Christchurch earthquakes that began on 4 September 2010. In this paper, two dimensional numerical analyses were used to assess the effectiveness of stone columns in mitigating liquefaction-induced lateral spreading. The state-of-the-art design procedures for stone columns to prevent liquefaction were used in the numerical modelling as the basis of assessing their applicability to mitigate lateral spreading. A case study from the Christchurch 22 February 2011 Earthquake was used to calibrate the numerical model for the study. Focus is placed on the reduction in accumulated surface lateral deformation and excess pore water pressure within the improved ground. The effects of lateral extent of the improved zone and the area replacement ratio of stone columns have been investigated.

2. Stone Columns

The installation of stone columns mitigates the potential for liquefaction by increasing the density of surrounding soil and allowing drainage to control pore water pressure generated. The introduction of stiffer elements, which can potentially carry higher stress levels and reduce the stress levels in the surrounding soils (Priebe 1991), provides resistance to deformation. These effects may reduce the build-up of excess pore water pressure which in turn, reduces the liquefaction potential, and the associated ground deformations.

Recently, installation of stone columns to mitigate liquefaction-induced lateral spreading has been investigated (e.g. Elgamal et al 2009) and it was concluded that stone columns were effective in reducing lateral deformation in sandy stratum. However, the current stone column design methods for liquefaction mitigation available in literature (Priebe 1998; Baez and Martin 1995) are largely focused on its implementation on level ground and foundation design, and not for lateral spreading.

3. Methodology

3.1 Model Definition

3.1.1 Representative site location

The selected site for the analysis is adjacent to the Avon River in the suburb of Dallington. It is located near the artificially straightened reach of the Avon River known as Kerrs Reach, about 2km to the east of the Christchurch CBD. Figure 1 below shows the site location map of the representative section.

Following the 4 September 2010 and 22 February 2011 earthquakes, Robinson et al. (2011) took field measurements of lateral spreading at various sites in Christchurch. The results from the field measurements show that the cumulative displacement along the closest transect, which was approximately 200m to the south, was approximately 0.8m. The lateral displacement data from this study was used as the basis to calibrate the FLAC numerical model.

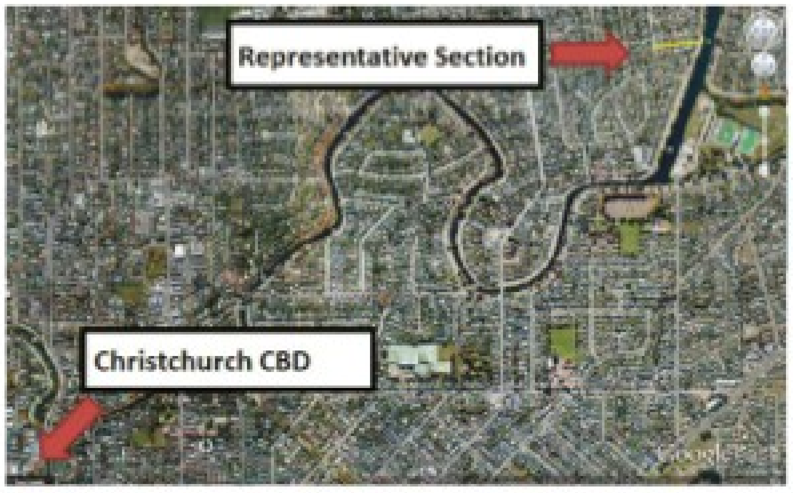

Figure 1. Site Location map of the representative section

3.1.2 Geological Section

To avoid complex geometries in the numerical modelling, the subsurface geological profile has been simplified to represent a generalised profile in the vicinity of the site and subsoil strata have been assumed to be horizontal. The slope geometry has been approximated from LiDar surveying results. Based on available CPT data (CGD 2012), the site is assumed to consist of three geological units: a sandy gravel crust 1.7m thick, overlying 8.2m of loose to medium dense sand which is underlain by dense sand with interbedded silt layers. Figure 2 shows a FLAC screenshot of the geological section used in numerical analyses. The groundwater levels which underlie the subject area are assumed to range from 0.9m to 3.2m below the existing ground surface. The model defines the centre of the river as x=0m, and the crest of the river is at approximately x=-31m.

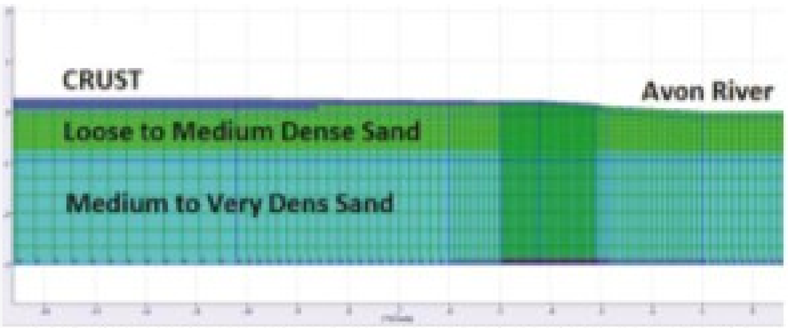

Figure 2. FLAC geological section for modelling

3.2 Static and dynamic material parameters

Subsoil material parameters have been interpreted from the borehole and CPT logs results. The materials subsurface shear wave velocities and dynamic characteristics were derived from the MASW geophysical testing results in the area. The dynamic characteristics of all soils in the model were assumed to be governed by the Seed and Idriss (1971) modulus reduction and damping ratio curves.

3.3 Strong motion record used for analyses

Seismic records from Riccarton High School (RHSC) station were considered to be suitable because no liquefaction was observed here. As the ground motion recorded at the RHSC station was recorded at the ground surface, where the input motion in the FLAC numerical model is at the base of the model, a Deconvolution analysis is required in order to model the representative motion at the base of the mode. The strong motion record was corrected into the direction perpendicular to the cross section and deconvoluted to the depth of engineering rock using a 1-D wave propagation programme Strata (University of Texas, 2009) prior to input to the numerical model. Figure 3 shows the input motion for the numerical analyses.

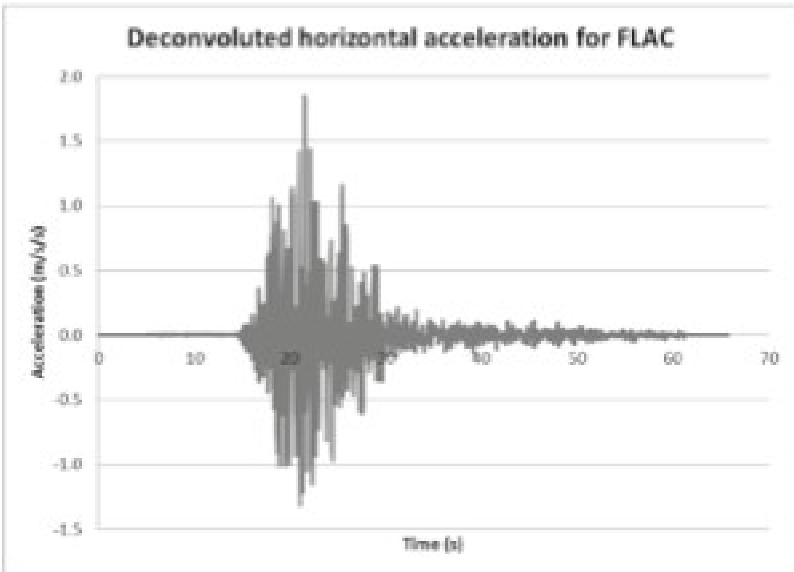

Figure 3. Deconvoluted horizontal acceleration for input into FLAC

3.4 FLAC numerical modelling

The finite difference numerical analysis program FLAC v. 7.0 (Fast Langrangian Analysis of Continua) in 2D was employed in this study. A coupled effective stress analysis was performed using a simple material model to simulate the behaviour of the soils, including liquefaction. The soil behaviour is based on the Mohr-Coulomb plasticity model with material damping added to account for cyclic dissipation during the elastic part of the response and during wave propagation through the site. Liquefaction was simulated using the Finn-Byrne model, which incorporates the Byrne (1991) relation between irrecoverable volume change and cyclic shear-strain amplitude into the Mohr-Coulomb model. Further details of the modelling are presented in Tang (2012).

3.5 Stone column modelling

3.5.1 Stone column parameters

The basic design parameters of stone columns include the stone column diameter, D, pattern, spacing and the backfill material to be used. The diameter of the stone columns in this study has

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been chosen to be 750mm. For the purpose of this study, a square pattern has been modelled with an effective diameter (De) equal to 1.13S where S is the spacing of stone columns. The resulting equivalent cylinder of material having a diameter De enclosing the tributary area of soil and one stone column is known as the unit cell. For this site, the Area Replacement Ratio, ARR (ratio of area of the stone column after compaction (Ac) to the total area within the unit cell) was determined to be 15%. The stone columns were modelled to be installed immediately adjacent to the crest of the waterway.

3.5.2 Modelling a 3D problem in 2D

A series of parallel trenches was used to represent the 3D stone column grid in 2D. The stiffness as well as the permeability of both soft soil and coarse grained inclusion needs to be adapted in order to model the deformation behaviour and drainage conditions correctly. Hird et al. (1992) and Indraratna & Redana (2000) recommended methods to perform a conversion of permeability. These transformations are also applicable to model smear effects. The ideal drainage performance of stone columns can be reduced by a smear zone. During the installation of stone columns, the structure of the soil immediately adjacent to the stone column may be disturbed, leading to a reduction of horizontal permeability. This zone of disturbed soil is called the smear zone.

3.5.3 Stone column mechanisms

3.5.3.1 Densification effect

The effect of granular pile installation on the modifications induced in loose to medium dense granular deposits was studied by Murali Krishna and Madhav (2009). Their findings were presented in the form of design charts that can be used design the required degree of treatment for the expected improvement or to estimate the improved SPT N_1 values of treated ground for respective area replacement ratios and SPT N_1 values of insitu ground. The improved SPT N_1 value for the studied site has been determined to be 26 from these charts,. Moreover, studies have shown that densification of the in-situ soil surrounding the stone columns decreases with distance away from the stone column (Obhayashi et al. 1999 and Weber et al. 2010). They determined that the extent of the disturbed zone is approximately 2.5 times the radius of the stone column and this was adopted in this study.

3.5.3.2 Reinforcement effect

As the current study is a 3D problem being modelled in a 2D model, the stiffness of the stone columns needs to be modified in order to model the deformation behaviour correctly. In a 2D plane strain model, the stone columns will be represented as an infinite trench with a width equal to the diameter of the stone column rather than a single column. The vertical stiffness of the column material used in the 2D model was calculated as an equivalent stiffness of individual columns in a 3D space.

3.5.3.3 Drainage effect

The drainage condition needs to be revised from a 3D problem into a 2D simulation. The excess pore pressure dissipation should be similar in both systems – the radial drainage system must equal the plane drainage system. The Idraratna & Redana (2000) equations to estimate plane strain permeabilities were used in the study. Weber et al. (2010) studied the smear zone and densification zone around stone columns and the smear zone was described as a strongly sheared and remoulded zone, which leads to a reduction of horizontal permeability in this zone. In this study, the smear zone was assumed to be 1/3 of the column radius, i.e. 0.125m. The horizontal permeability of the smear zone was determined by formulae defined by Indraratna & Redana (2000).

4. Numerical Results

4.1 Control model – no stone columns

Figure 4 illustrates the calculated surface displacements by the FLAC numerical model and the measured cumulative lateral displacements at transect CH_DAL_15 by Robinson et al. (2011). The figure shows that there is a similar 'exponential decay' distribution where the lateral spreading displacements rapidly decrease with the distance from the waterway. This is consistent with the conventional liquefaction induced lateral spread mechanism where the lateral spreading displacements reduce exponentially away from the riverbank. The calculated surface lateral displacements by FLAC were compared to the field measurements by Robinson (2011). The magnitude of lateral spread between the field measurements and the FLAC calculations at the crest off the bank differed by about 20%. This could be due to the stream bank in FLAC being modelled on the loose silty GRAVEL, where in real life the bank may be supported by vegetation or small man-made structures. The difference in lateral displacements between the calculated and measured values reduces to zero at approximately 100m away from the centre line of the river. Such difference is considered acceptable and comparable considering the displacement trend is reasonably similar.

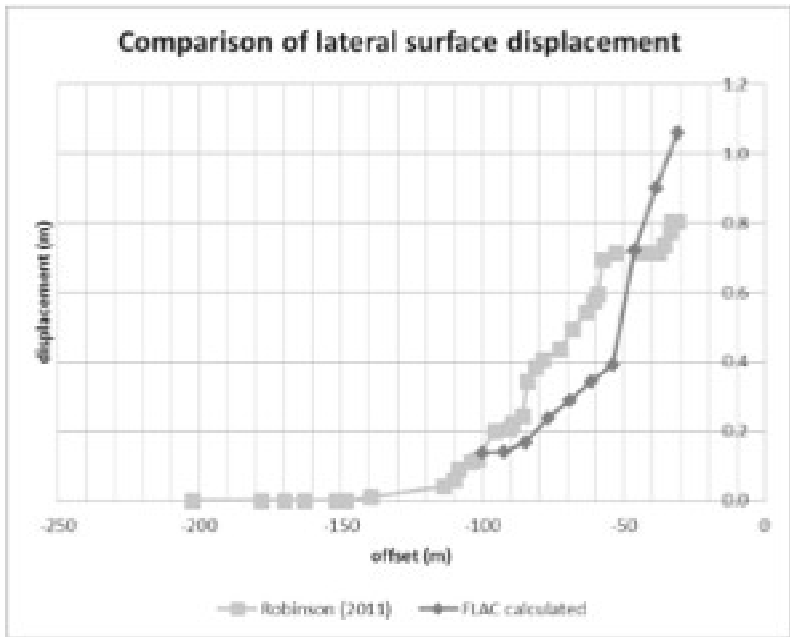

Figure 4. Comparison of lateral surface displacement between Robinson (2011) field measurements and FLAC calculated displacements with distance away from river centre line.

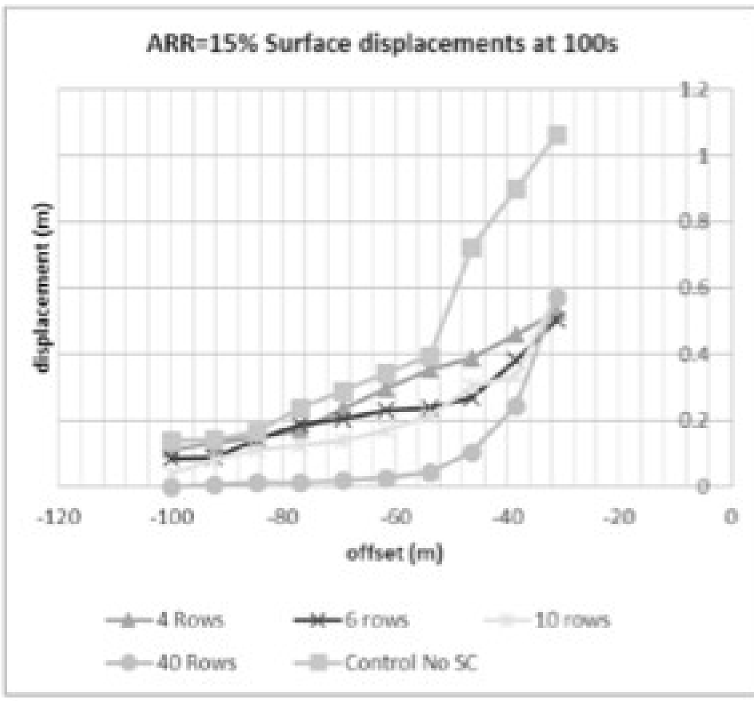

Figure 5. Comparison of lateral surface displacement for stone columns with ARR=15% with varying improvement zones with distance away from river centre line.

4.2 Stone column models

4.2.1 Varying lateral extent of improvement zone

Figure 5 illustrates the comparison of lateral surface displacements between the different numbers of stone columns modelled. There is an obvious trend that as the improvement zone is increased, the lateral displacement along the surface on the cross section is reduced. It can be seen that as the number of stone columns increases, the curvature of the lateral displacement distribution increases. The lateral surface displacement reduces more rapidly with respect to distance away from the crest of the waterway as more stone columns modelled. It should be noted that there is little difference in the lateral displacement at the crest of the riverbank for all cases, varying by approximately 0.1m. This could be explained that the sloping riverbank is still vulnerable to gravitational inertia forces close to the crest of the river driving the soil mass down the slope. Furthermore, the first stone column was modelled at 1.0m away from the crest of the slope, so the 1.0m column of soil directly adjacent to the crest of the riverbank may have benefit from the densification effect, but does not have an increased stiffness. The stone columns could be installed right on the edge of the riverbank for improved results.

The figure shows that four rows of stone columns are not adequate to mitigate liquefaction and the associated lateral spreading towards the riverbank in this subject site. Even though structures may not be built directly adjacent to the Avon River, the displacements are still relatively high at distances further away from the crest. In contrast, in the model where 10 rows of stone columns were modelled, there is a steep gradient over the first 8m away from the crest of the riverbank, which reduces lateral displacement from 0.59m to 0.34m, reducing further to 0.2m to approximately 23m away from the crest of the riverbank. Typically, in the area of the subject site in the Dallington suburb, the closest structure adjacent to the Avon River is the road, which is at least 20m away from the crest of the riverbank, and the closest residential dwelling is approximately 40m away from the crest of the riverbank. At 20m away from the crest of the river (x=-51m), the models with 6 rows and 10 rows of stone columns recorded a lateral displacement of approximately 0.25m whereas the model with four columns recorded a lateral displacement of 0.38m. The model with 40 rows of stone columns recorded a lateral displacement of 0.06m, which may be within the tolerance of these man-made structures.

The lateral surface displacement close to the crest of the riverbank is between 0.5m to 0.6m where stone columns are modelled at ARR=15% even when the improvement zone is extended to 40 rows of stone columns. This implies that the ground improvement near the crest of the riverbank is not adequate to withstand the gravitational inertia forces caused by the sloping geometry of the cross section.

Another way to improve the system of stone columns is to increase the ARR, i.e., installing the stone columns at closer spacing to achieve a stiffer and more densified improvement zone with shorter

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drainage paths. This should reduce liquefaction potential in the improved zone. Stone columns with an ARR of 20% were analysed.

4.2.2 Stone columns with ARR = 20%

Figure 6 illustrates the surface lateral displacements along the cross section plotted for the different stone column configurations. For four rows of stone columns with an ARR of 20%, the maximum lateral displacement at the crest of the riverbank is approximately 0.51m. The surface lateral displacements then drop rapidly to 0.36m at approximately 8m away from the crest, which is just outside the improvement zone. It should be noted that the closest man-made structure to the Avon River is the road along the Avon River which is approximately 20m away from the crest of the riverbank. At 20m away from the crest of the riverbank, the ARR=20% with four rows of stone columns recorded a lateral displacement of 0.3m, compared to the ARR=15% case which recorded a lateral displacement of 0.38m, which is a 21% improvement.

The distributions of lateral surface displacement along the cross section showed a similar trend to the ARR=15% case. An increase in the number of stone columns, results in a greater rate of decrease in lateral displacements with respect to distance away from the crest. At 20m from the crest, the model with 6 rows and 10 rows of columns showed lateral displacements of 0.26m and 0.20m respectively.

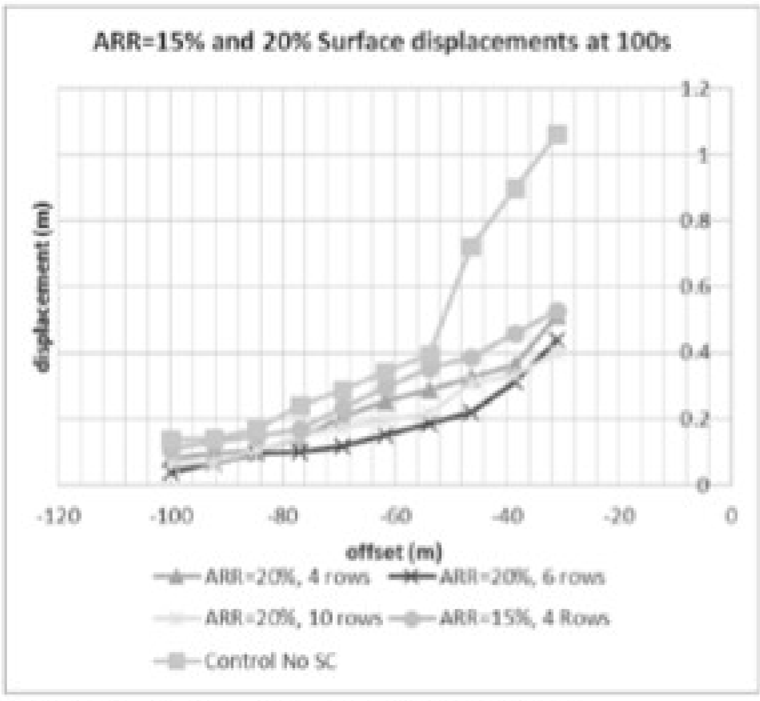

Figure 6. Comparison of lateral surface displacement along the cross section with an ARR=15 and 20% and varying the improvement zones with distance away from river centre line.

5. Discussion

It was found through the analyses that the recommended area replacement ratio estimated using the Priebe and Grundbau (1998) method did not totally eliminate lateral spreading towards the Avon River with a system of four rows of stone columns, resulting in lateral displacements in excess of 0.5m close to the crest of the riverbank. Even this 0.5m of lateral displacement could be very damaging to any underground pipelines, cables or structures that may be constructed close to river. However, it was noted that large displacements at the crest of a waterway may not be critical as the closest structures may be several metres away. The closest structure next to the river for the case study is the road that runs along the Avon River, which is approximately 20m away from the crest of the riverbank. A lateral displacement of 0.35m was calculated at a distance of 20m away from the crest of the riverbank. This magnitude of lateral displacement is considered to be detrimental for any underground services, which are commonly laid beneath roads.

More rows of stone columns were also modeled with the same ARR. It was found that as more rows of stone columns were added, lateral surface displacement decreases at a higher rate with distance from the crest of the riverbank. Where the ground improvement zone is extended to 40 rows behind the crest of the riverbank, the lateral surface displacement at the crest was still in excess of 0.5m, but at 20m away from the crest was approximately 0.1m, which could be acceptable.

6. Limitations and Recommendations

Many assumptions have been made in this study. These assumptions have largely been made on the basis of current literature, but it is understood that there are various limitations to the study. These have been summarised below.

- Only one input ground motion has been used
- Only one cross section has been analysed
- Results were compared to lateral displacement data that was recorded at a site 200m away
- Numerical model in 2D for a problem that is essentially 3D

Further studies would be recommended to produce design charts for stone columns used for mitigating liquefaction induced lateral spreading. A much more comprehensive parametric study analysing different slope geometries, subsoil parameters, groundwater levels and varying input motions would be required to create design charts that would determine the area replacement ratio and the extent of improvement zone required. It would be recommended that these studies be performed using three dimensional numerical modelling to mimic the physical problem as closely as possible. The current study could be further developed into a three dimensional analyses using FLAC 3D, and results from the 2D model and 3D model compared to investigate any differences and the validity of the current 2D model.

7. Conclusions

A study was carried out to assess the effectiveness of stone columns against liquefaction induced lateral spreading using the finite difference programme FLAC. A site in Christchurch which was affected by the February 2011 earthquake was used for this purpose.

Overall, the investigations have shown that the current design procedures are adequate for design of stone columns against earthquake induced liquefaction on a level site, but complementary improvement around the crest of the slope may be required to reduce lateral displacements to tolerable levels close to the crest of a slope or a waterway.

The installation of more rows of stone columns was shown to greatly decrease lateral displacement. However, the installation of some 40 rows of stone columns along a 500m stretch of the Avon River would not be feasible considering there are residential buildings 40m away from the river, and the economic costs of constructing this system of stone columns. It is therefore a problem of optimisation between displacement tolerance, available space and economic costs to decide the extent of the improvement zone width behind the river.

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A CASE STUDY OF AN GROUND INVESTIGATION INTO A BUILDING COMPLEX DAMAGED DURING THE CANTERBURY EARTHQUAKE SEQUENCE, NZ".

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ABSTRACT - Following the Ms7.1 rupture of the Greendale Fault on 4th September 2010, the Christchurch area has experienced and continues to experience a sequence of aftershocks. The most destructive to date was Ms6.3 22nd February 2011 centred within 8km of Christchurch's CBD in which PGAs of up to 0.8g were experienced. This event caused structural damage and ground deformation of a building complex, located on the banks of the Avon River.

This paper describes the initial geotechnical inspection, investigations, resulting engineering geological model and the implementation of the model for remedial design and prevention of further lateral spreading induced structural damage at the building complex during subsequent seismic events.

A geotechnical assessment was undertaken to assess both general site stability and building stability issues. Ground deformation features that were observed included liquefaction, lateral spreading, tension cracking, voids and ground subsidence. Lateral spreading was most evident within the basement where the ramp structure showed 130mm of horizontal and 50mm of vertical displacement towards the river relative to the main basement structure. Up to 700mm of subsidence was visible between the building and the river, inducing tilting of the building. Liquefaction was most evident to the south and north of the complex, with up to 400mm thickness of ejected sand and silt observed. Site investigations included boreholes, piezometers, CPTs, ground penetrating radar and topographic survey.

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PROPOSED KOTUKU FLOOD DETENTION DAM – GEOLOGY AND GEOTECHNICAL DESIGN FEATURES

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ABSTRACT – In the North Island of New Zealand, volcanic soil and rock pose seepage issues for water retaining structures, and very careful investigation and design is necessary. The proposed Kotuku Flood Detention Dam has been designed to reduce flooding within Whangarei City, and is expected to commence construction in the 2014/2015 earthworks season. The proposed dam is classified a high potential impact classification (PIC) dam, and hence the geotechnical issues have been rigorously analysed. The proposed dam and catchment are located within a complex geological environment, where a basalt flow and flow margin, firm alluvial soils, and massive sandstone rock are present in the proposed dam foundation. This paper describes the geotechnical issues posed by site geology in preparing a detailed design of the 18m earth embankment dam. Investigation and testing has been undertaken to assess the site geology, including founding conditions, seepage potential and materials for dam construction. Permeability testing, including packer, falling and constant head tests, provided input to forming a geological model of the subsurface conditions. Transient groundwater analysis on the geological model was undertaken for a 36 hour, 1% annual exceedance probability (AEP) flood event. This analysis led to the development of a detailed design for the proposed dam, which comprises a clay liner, undercuts of alluvial and selected residual and flow margin materials, toe drains and internal drainage.

1. Introduction and Project Background

The Northland region of New Zealand, in which Whangarei is situated, has been subject to significant flood events over the past 10 years. In many cases, these events have highlighted the vulnerability of certain land areas to flooding and a requirement to improve/upgrade flood protection.

The proposed Kotuku flood detention dam is one such flood protection measure identified by Northland Regional Council (NRC). The concept for a dam at the proposed site, however, is not new, with studies dating as far back as 1966. Riley Consultants Ltd has been providing geotechnical and water resources engineering services for the proposed dam since undertaking feasibility studies in 2011.

The proposed embankment-type dam is located on the Nihotetea Stream to the south-west of the Whangarei Central Business District (CBD), as shown on Figure 1. The dam has a high potential impact classification (PIC) by NZSOLD Guidelines (2000).

The dam will be approximately 18m high and contain approximately 1.3 million m^3 of water during the design 1% Annual Exceedence Probability (AEP, which includes climate change) flood event. The 9.1km^2 catchment area shown in Figure 2 is predominantly covered in pasture, with some areas of scrub and bush. A predicted 1% AEP flood event currently results in a flood depth of 0.75m in the CBD, affecting approximately 620 buildings.

Three sites were initially considered, and the proposed dam site was selected after a feasibility study, cost comparison, and a reservoir stability investigation.

Figure 1. Map of proposed dam site

At the time of preparing this paper, August 2014, resource consent for construction of the dam has been obtained, and a contractor has been selected to undertake the construction works. Earthworks will be undertaken in the 2014/2015 earthworks season starting October 2014, and preliminary works to relocate services at the dam site were completed in July 2014.

2. Geology

The Institute of Geological Nuclear Sciences Ltd (GNS) 1:25,000 geological map of the Whangarei Urban Area (GNS geological map 26) indicates the site is underlain by several different geological units. The basement rock in the area is Waipapa

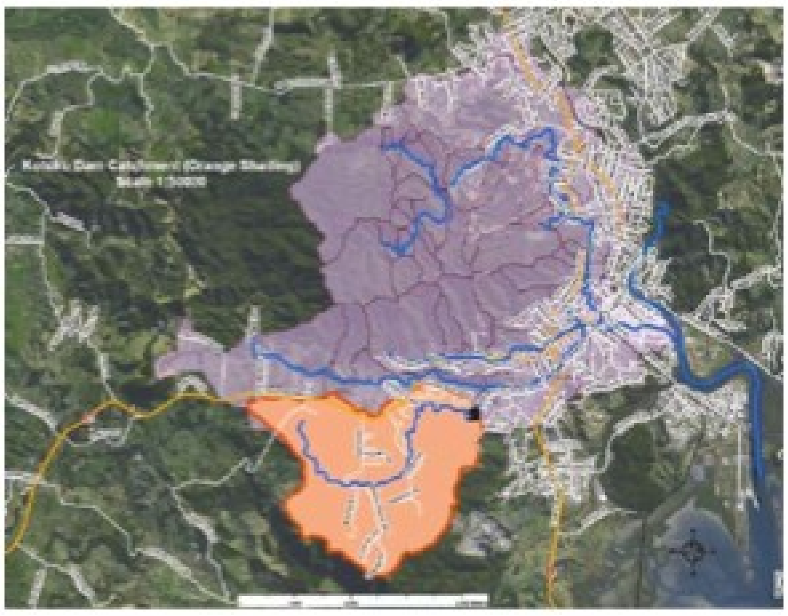

Figure 2. Kotuku Dam catchment

Terrane greywacke, which is overlain by Ruatangata sandstone, and is located near the ground surface on the southern and eastern slopes.

The northern and western slopes of the valley are underlain by basaltic lava of the Kerikeri Volcanic Group. The basaltic lava sourced from the Maunu Mountain volcano and associated scoria cone to the west of the site has flowed down and infilled a paleochannel of the Raumanga Stream and dammed associated tributaries. The basalt lava outcrops as cobbles and boulders at the ground surface and in the stream bed, and is exposed in a waterfall downstream of the dam site. The rocks are generally fine-grained and vesicular.

The lower-lying central portions near the watercourse are underlain by alluvium. The alluvium on the site is fluvial and lacustrine deposits consisting of clays, silts and gravels with generally minor organics. These are likely to have accumulated with the partial damming of streams by lava flows.

Site inspection, mapping, machine boreholes, test pits, and hand auger boreholes were undertaken at the dam site, in the borrow area, and reservoir to investigate the geology at the site. Three phases of investigation work were undertaken: feasibility study, preliminary design and detailed design.

2.1. Dam Site Geology

2.1.1. General

A plan layout of the dam site is shown in Figure 3. The proposed dam footprint extends over variable soil and rock types on both the left and right abutments and upstream to downstream. The right abutment is underlain by competent sandstone with a relatively shallow depth of overlying residual soil. The left abutment consists of a lava flow with mixed flow margin materials surrounding the basalt lava. The lava flow, which sits on the alluvium that overlies sandstone at depth, has filled a paleovalley. The firm alluvium varies in thickness from 2m to 5m in the valley floor at the dam centerline, increasing in thickness upstream into the old basin. Figure 4 presents a geological long section along the centre line of the embankment.

Investigations at the dam site were undertaken to develop and refine a geological model of the subsurface conditions. Testing was challenging due to access constraints from private properties, dense bush, steep slopes and large boulders at the ground surface. During drilling of the machine boreholes, falling head, constant head, and packer testing was undertaken in the Kerikeri Volcanic Group and Ruatangata Sandstone formation materials to assess soil and rock mass permeability. Angled machine boreholes were undertaken at the proposed dam abutments to maximise the likelihood of intercepting steeply dipping and vertical rock joints to provide a better understanding of defects and the rock mass permeability.

2.1.2. Subsurface Conditions

The central stream area has been infilled with alluvial material transported from the catchment upstream. The alluvial material is generally firm to stiff silty material with fractions of organics and a maximum thickness of 5m at the upstream end of the dam footprint.

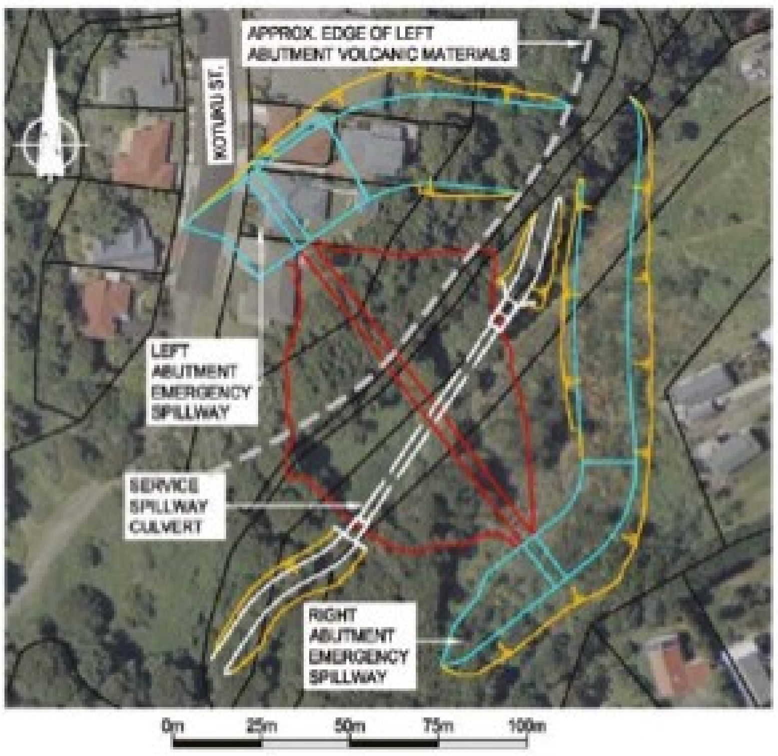

Figure 3. Plan layout of dam site

The right abutment comprises a steep slope of typically fine-grained and massive Ruatangata Sandstone of the Te Kuiti Group. The sandstone is generally completely weathered near the surface grading to unweathered at approximately 7.5m depth. Localised areas of colluvium and alluvium were encountered at the ground surface. Artesian pressures were encountered near the base of the slope, inferred to be from groundwater originating in the area upslope of the proposed dam site.

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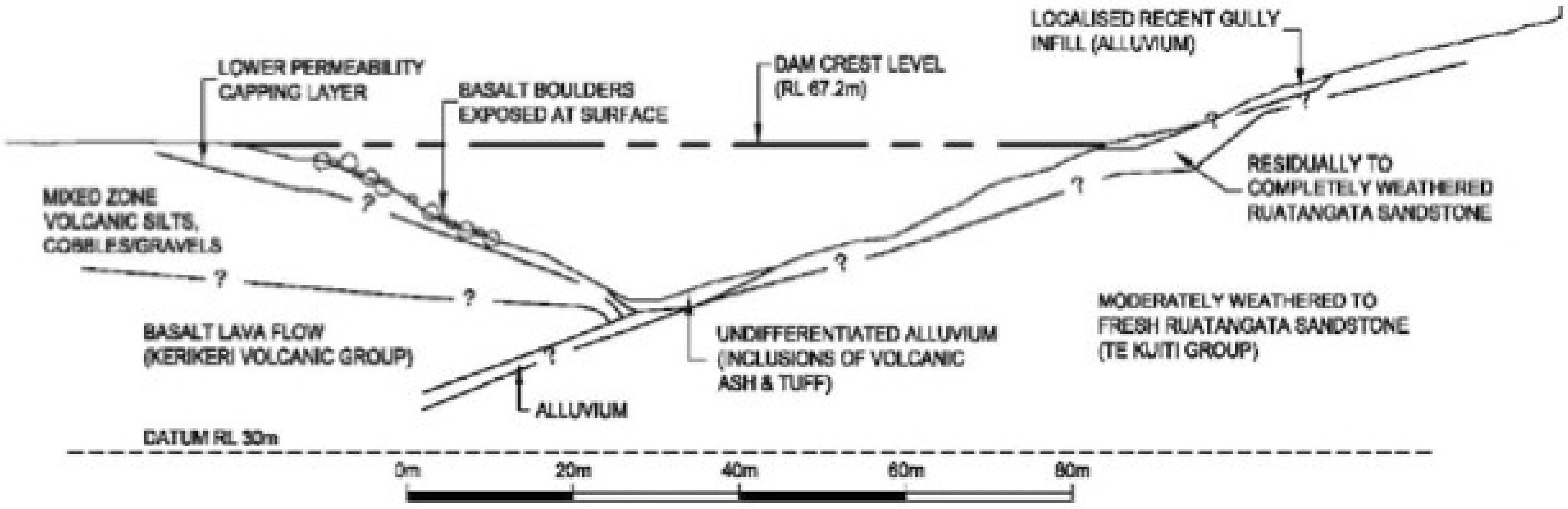

Figure 4. Geological cross section

The left abutment is located on the southern lateral margin of a large basalt lava flow of Kerikeri Volcanic Group material from the Maunu Mountain volcano and associated scoria cone to the west of the dam site. The lava flow margin generally comprises a mixed zone of materials including volcanic silts, gravels, and cobbles overlying the strong basalt lava flow at between 9m and 15m depth in the mid-slope to dam crest area. The basalt lava flow comprises strong fractured basalt rock. Defects in the basalt have a very narrow aperture with clay infill, and are vertical with moderately widely spaced sub-horizontal defects. A capping layer of lower permeability material overlies the mixed zone materials. The lava flow and flow margin are underlain by slightly weathered Ruatangata sandstone and, near the toe of the slope, with a firm to hard alluvial layer.

2.1.3. Permeability Testing

Packer testing was undertaken in the rock encountered at the dam site, with falling and constant head tests in the soils and weathered rock on the left abutment. In packer testing, pressurised water is pumped into an area of open machine borehole shut off by a packer, and the flow of water escaping through defects in the rock mass below the packer is recorded. The flow is recorded over five or more two minute intervals of water pressure being incrementally stepped-up and then down, and the pattern of the flowrate over the entire test is noted. Patterns can involve washing out of material (increasing flow during the test), filling of voids (decreasing flow during the test), higher flow when fractures are open at higher pressures (dilation), turbulent flow in fractures at higher pressures, and laminar flow where all the Lugeon values are the same for all pressures. One Lugeon unit corresponds to 1 litre/minute per metre length of test section at an effective pressure of 1 MPa. Following analysis, the most representative flowrate over the test is correlated to a Lugeon value for the rock mass.

Packer testing in the Ruatangata sandstone on the right abutment generally recorded Lugeon values of 0 to 6 (very low to moderate permeability) consistent with a massive rock with very tight to narrow defects. Packer testing in the basalt lava flow on the right abutment recorded Lugeon values of 2 to 100 (moderate to high permeability) indicating some variability between tight and many open joints.

Falling and constant head testing in the mixed zone of flow margin materials recorded permeability results of 8.3×10^{-5} m/s and 3.5×10^{-4} m/s, which are equivalent to the permeability of a silty sand to sand. A capping layer of lower permeability material was encountered between 3.7m and 6m at the left abutment, overlying the more permeable mixed zone materials.

2.2. Borrow Area Geology

The materials for constructing the dam are to be borrowed from the right abutment emergency spillway excavation and a borrow area located approximately 400m west of the dam site. The borrow area is a small hill underlain by residually weathered to unweathered weak sandstones and mudstones of Ruatangata Sandstone formation. Test pits were undertaken at the borrow area to determine the excavatability of the material and bulk samples were collected for laboratory testing.

2.3. Laboratory Testing

Laboratory testing was undertaken on samples from the left and right abutments and the borrow area. Key objectives of the laboratory testing were to assess the nature and strength of materials from the borrow area and right abutment spillway cut, and the grading of the flow margin soils on the left dam abutment to correlate to permeability and provide input for filter design. Compaction, grading and dispersivity tests were undertaken on the borrow materials to determine their suitability as dam construction materials, and strength tests were undertaken to assess the excavatability of the sandstone rock.

The borrow area and right abutment spillway materials are generally non-dispersive (ND1 and ND2), having over 65% fines (smaller than 0.063mm particle diameter, i.e. silt and clay sized particles), and achieving maximum dry density of around 1.35t/m^3 (standard compaction) and over

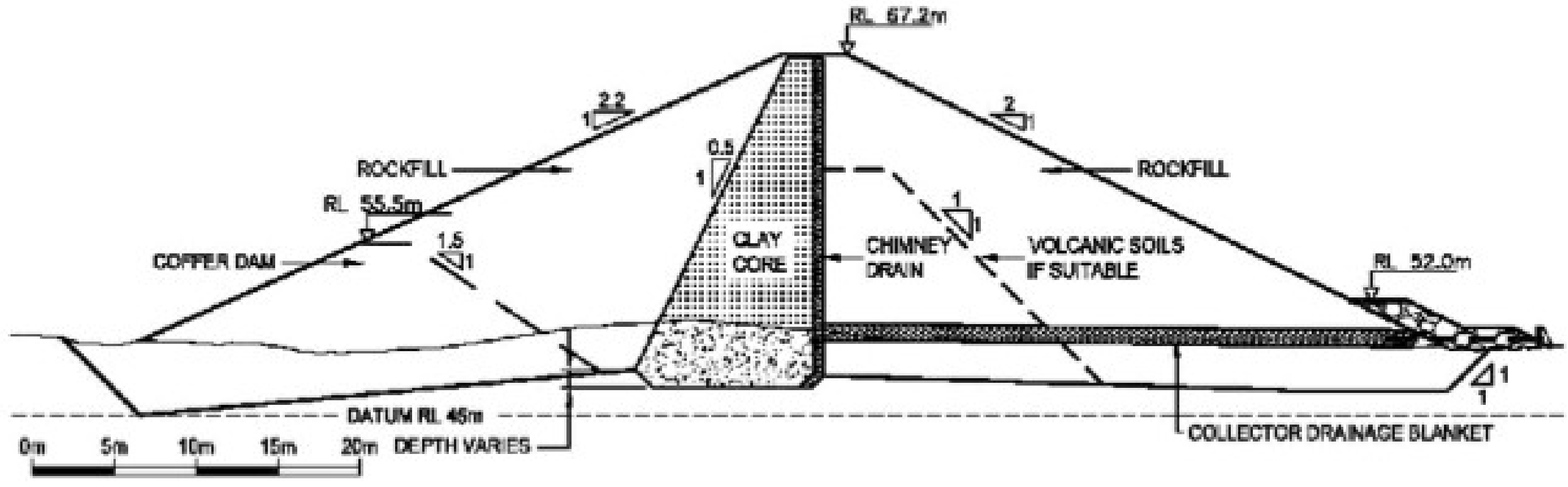

Figure 5. Typical embankment cross section

1.55t/m^3 (heavy compaction). The tests in sandstone material from the borrow area recorded uniaxial compressive strength in the range of 5 to 25MPa, and will likely require ripping to excavate. The flow margin material on the left abutment has gradings ranging from silty gravel to clayey silt.

2.4. Reservoir Stability

In the preliminary stages of investigation and design, when evaluating multiple dam sites, consideration was given to geological features that could lead to instability when the reservoir is temporarily at peak levels. This stage of assessment comprised a site walkover, field testing, and analysis of results from around the proposed reservoir.

The likelihood of large-scale instability within the basin slopes due to temporarily higher reservoir levels is considered low. In an extreme rainfall event, the slopes surrounding the reservoir will likely be saturated and localised surficial slumping may occur within the lower-lying areas of the basin, which is not considered to pose an unacceptable risk to the dam.

However, as part of prudent risk management, monitoring of reservoir stability will be incorporated into a long-term dam safety assurance programme.

3. Geotechnical Design Features

3.1. Foundation Treatment

As shown in Figure 4, the geological model for the dam foundation comprises volcanic materials on the left abutment, and weathered sandstone on the right abutment, with alluvium in the central valley. The volcanic materials are variable in composition, whereas sandstone on the right abutment is relatively uniform.

In the valley floor, the alluvium will be undercut to the stronger sandstone to minimise settlement and stability issues. At the upstream toe of the dam, the undercut may be 4m to 5m or greater, which is considered achievable. Such an earth embankment dam is typically expected to settle less than 0.7% of its height (i.e. less than 140mm), and undercutting the alluvial and completely weathered sandstone materials and compacting the dam construction materials well will ensure that the settlement is maintained in that range.

On the right abutment the moderately weathered sandstone is typically shallow (less than 2m) and is an acceptable founding material. Overlying soil, including completely weathered rock and any localised areas of alluvium and colluvium, will be stripped. There is no evidence of significant defects within the massive sandstone rock.

On the lower left abutment, where the dam height is greatest, any weaker near-surface materials and the mixed zone materials will be removed to sandstone or the basalt lava flow. Treatment of basalt defects will possibly be required when the lava is exposed in the dam foundation. As the dam section rises up the relatively steep abutment, the depth to the basalt rock increases significantly and it is not practical to sub-excavate to that depth. Consequently, rubbly flow margin materials comprising variable cohesive soils mixed with gravels and volcanic materials will be left in place in the foundation. There is no evidence of continuous highly permeable or erodible materials in the flow margin, but there are gap-graded materials that may be susceptible to piping if significant sustained seepage flow were to occur. Although peak reservoir levels are only sustained for a matter of hours, seepage control features are considered essential in the left abutment foundation, which are discuss in section 3.3 and include an upstream clay liner.

3.2. Embankment Design

The proposed dam zoning consists of a central clay core, a zone of volcanic soils immediately downstream of the core, and upstream and downstream rockfill shoulders. Figure 5 presents a typical cross section through the embankment.

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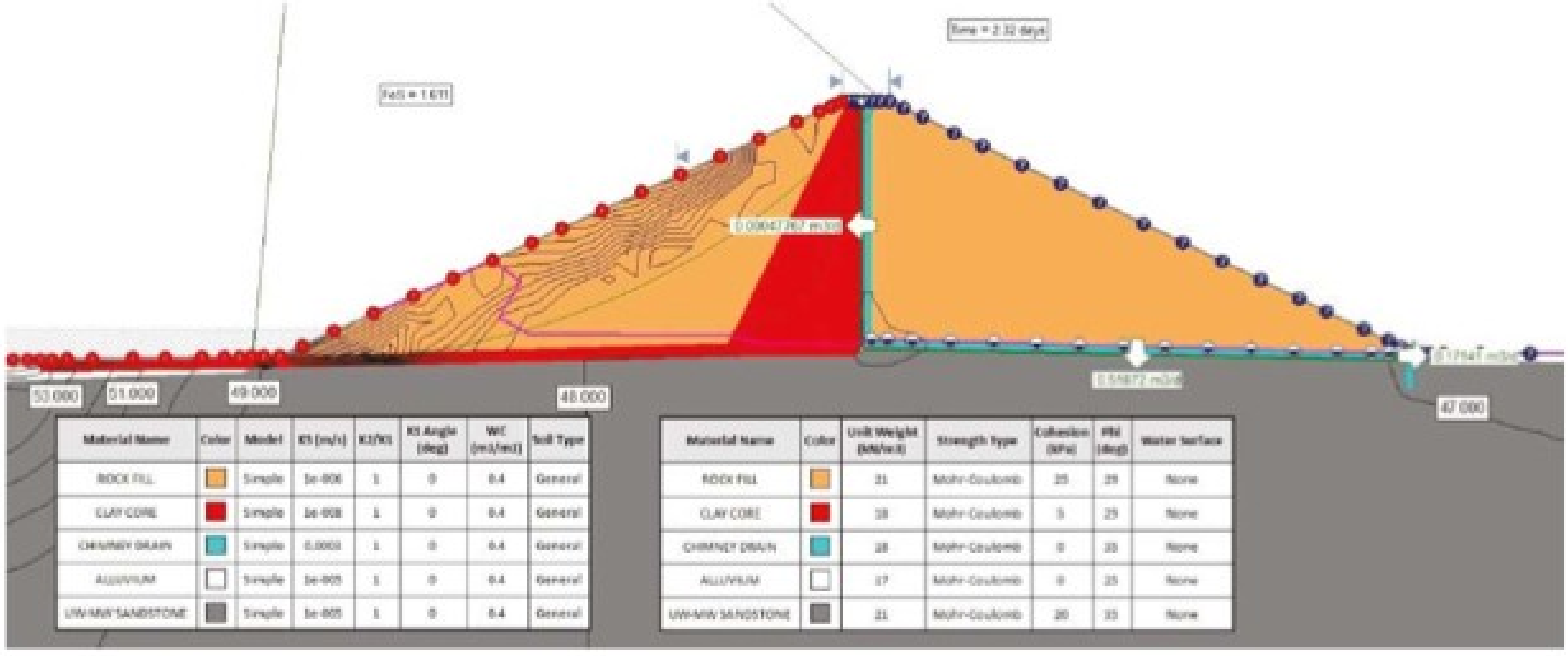

Figure 6. Rapid drawdown slope stability analysis

The core materials will be obtained from the surficial soils or completely weathered rock. Completely weathered rock (especially siltstone) is likely to be an acceptable core material, as it breaks down/compacts to a fine-grained material. Sheepsfoot rollers are nominated in the construction specification so that gravel-sized particles are broken down by roller action.

The rockfill and clay materials will be obtained from the nominated borrow area and the right abutment emergency spillway excavation. As the rockfill is sourced from a weak rock, the compacted material will have a lower percentage of gravel-sized particles, and permeability is expected to be low. Selected soils from the left emergency spillway excavation are proposed to be used in the downstream shoulder. A high standard of compaction is necessary for the clay core and clay liner. It is recommended that 200mm loose layers of material with greater than 65% fines content are compacted to achieve a shear vane strength of greater than 120kPa and air voids of less than 8% (5% average). The rockfill on the upstream and downstream shoulders will be compacted to 95% of the dry density standard compaction with air voids less than 10% (8% average).

3.2.1. Slope Stability

Slope instability is most likely to occur during rapid drawdown conditions when the dam freeboard is much greater than at the peak flood level. The likelihood of a slope failure causing a large wave and potentially overtopping the dam is therefore much less with a detention dam compared with a permanent reservoir.

Slope stability analysis was undertaken using the Slide software package on three sections through the dam (two on the left abutment) for a variety of cases, including effective stress, total stress, seismic loading from an operating basis earthquake (OBE: 1:150 year event, 0.1g) and maximum design earthquake (MDE: 1:10,000 year event, 0.32g), and a rapid drawdown analysis on the upstream slope. Input for geotechnical material parameters came from triaxial testing on typical rockfill material from the borrow area (c' = 25kPa, φ' = 29°) and assumed parameters for other materials in the dam embankment and foundation based on in-situ strength testing. Sensitivity analyses were undertaken on all the geotechnical material parameters used. A typical Slide printout for the rapid drawdown analysis is shown in Figure 6. The required Factors of Safety (FoS) adopted from NZSOLD Guidelines (2000) are 1.5 under steady state seepage, 1.2 under OBE, a FoS of greater than 1 under MDE, and 1.2 to 1.3 during rapid drawdown conditions on the upstream slope.

A deformation analysis was also undertaken for the OBE and MDE events, which showed minor to moderate damage including 40mm crest settlement and 30mm crack width in an MDE event.

3.3. Seepage – Internal Drainage and Clay Liner

The internal drainage consists of a fully intercepting 600mm wide chimney drain that extends to peak maximum flood level and connects to a base horizontal drainage/filter blanket, as shown in Figure 5. The horizontal drain is locally thickened in the vicinity of the service spillway culvert (beneath the dam and in the current stream bed location), where there is a greater risk of cracking and concentrated leaks. A 250mm thick continuous filter layer is provided on the left abutment foundation surface with a series of 1m thick high capacity strip drains, consisting of a filter layer surrounding a coarser drainage material. A toe drain is provided on each abutment.

The filters have been designed to allow for compatibility with the embankment and foundation soils at site, with the critical surfaces being the clay core to vertical drain interface and the foundation to horizontal/toe drain interface. The filter design was undertaken according to guidelines set out in Fell et

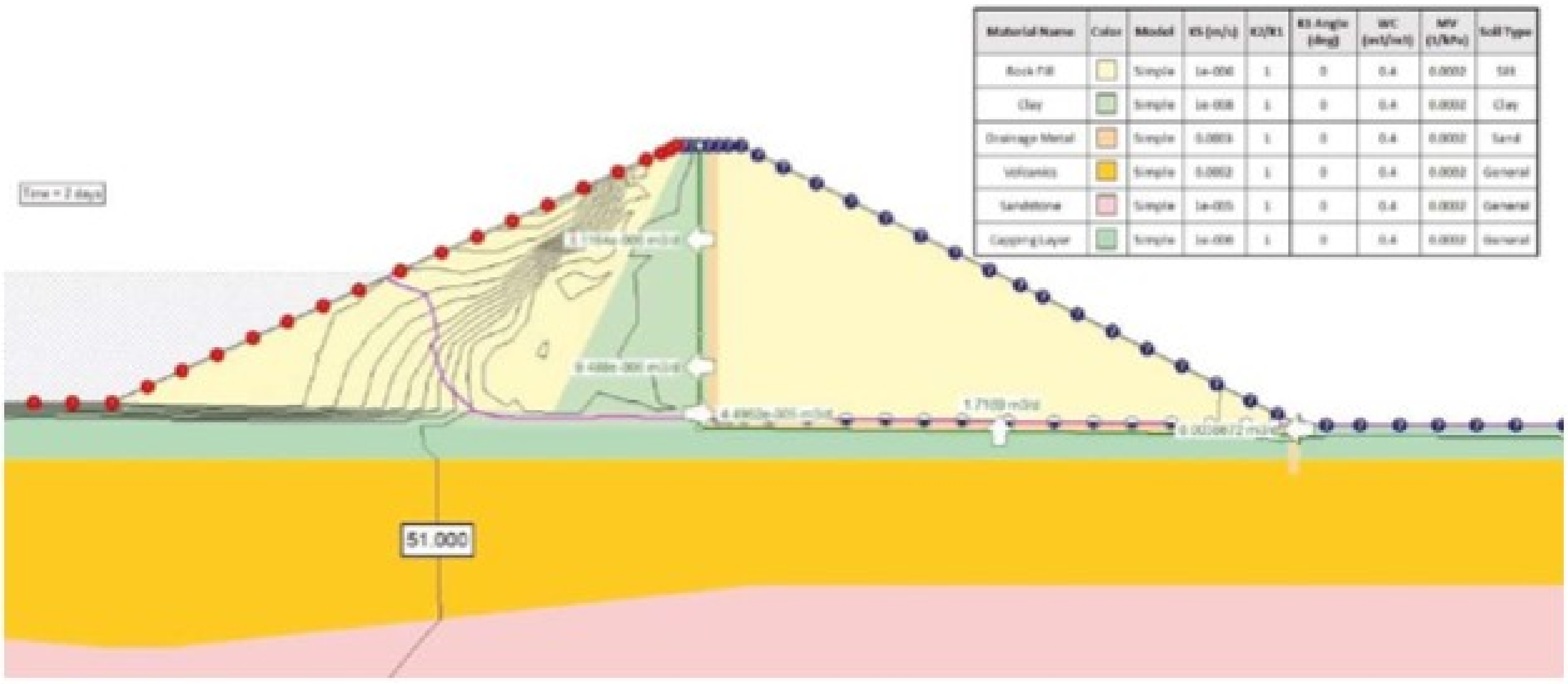

Figure 7. Transient groundwater analysis

al (2005), which has specific criteria for determining the filter design gradations for the critical filters. Two filter gradings are recommended, with the coarser (all gravel-sized, >2mm particle diameter) drainage material only to be used in the high capacity strip drains. The second filter material consists of well-graded sandy gravel with less than 2% fines. A horizontal drain thickness of 250mm, which is approximately 20 times the maximum particle diameter, has been adopted to prevent segregation of particles in the filter blanket.

An upstream clay liner on the left abutment is provided as a means to reduce seepage potential and pressures in the left abutment foundation. The clay liner is 1m thick and extends 20m from the upstream toe of the dam. This liner was modeled in the seepage analysis.

A transient groundwater analysis of flow through the dam embankment and foundation was prepared using the Slide software package groundwater function. An input hydrograph of the three day 1% AEP storm event on the upstream side of the dam was provided, which has a maximum water level after 1.32 days and no water in the basin after 2.32 days. The permeabilities for the embankment and foundation materials were inferred from the packer test and falling and constant head test results and from assumptions based on the geological model for the dam embankment and foundation materials. A key assumption in the Slide model is a pressure head of zero in the horizontal drain on the downstream shoulder, which assumes the drain is flowing in a large detention event. A printout from the Slide groundwater model is shown in Figure 7. The analysis is sensitive to input parameters and assumptions.

The highest flowrate into the horizontal drains was obtained on a mid-slope section on the left abutment, with the most likely flowrate into the drains ranging between 0.3 and 2 m^3/d/m. Assuming the most conservative parameters in the analysis, a value of 34 m^3/d/m is obtained. Lower flowrates were calculated for the lower left abutment and central dam sections. A design flowrate of 1 m^3/d/m has been used in design of the horizontal drains on the left abutment. The proposed strip drains on the left abutment achieved a FoS of 11 for this design flowrate, which is greater than a FoS of 10 that is considered acceptable for the drains.

4. Conclusions

The proposed 18m high Kotuku flood detention dam is a high PIC embankment dam to be constructed on the Nihotetea Stream to reduce flooding in Whangarei City. The dam is located within a complex geological environment with three different geological units including a moderately permeable basalt lava flow and flow margin within the dam foundation. Detailed site investigation has been undertaken to assess the subsurface conditions and prepare a geological model. Foundation treatment, embankment, seepage, and internal drainage designs have been prepared for the proposed dam, which include a clay liner, undercuts of alluvial and selected residual and flow margin materials, toe drains, and internal drainage. The dam is scheduled for construction in the 2014/2015 earthworks season.

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FERRYMEAD BRIDGE REPLACEMENT – DESIGN AND CONSTRUCTION CHALLENGES

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ABSTRACT – The Ferrymead Bridge, which is a key lifeline to the Sumner and Redcliffs area in Christchurch, was identified as being vulnerable to damage in a major earthquake event. Seismic strengthening and widening of the bridge was started prior to the 2010 Darfield earthquake. The February 2011 earthquake caused extensive damage to the superstructure and partially constructed widening works. As a result it was decided to demolish the existing bridge and replace it.

This paper outlines the damage at the bridge site due to the 2010-2011 Christchurch earthquake sequence, and the extent of the site investigations. The resulting ground model used in the design of the replacement bridge structure is described. Key geotechnical issues associated with design, including the liquefiable nature of the soil, large lateral spread loads, the influence of the existing piles and implications of the highly variable nature of the founding volcanic layers on the bridge are discussed. The new bridge is supported by piles socketed into bedrock, and designed to resist the lateral spread loads on the bridge. The permanent pile casing was grouted into bedrock. Challenges and key findings from the piling operations for the replacement bridge are also presented.

1. Introduction

Ferrymead Bridge is located in Ferrymead, Christchurch. It crosses the mouth of the Heathcote River where the river opens into the Avon-Heathcote River Estuary, providing a link between the city and the south eastern suburbs including Sumner and Redcliffs (refer Figure 1). The bridge is a key lifeline to these areas as it carries all major services and, with the alternative road route expected to be closed due to rockfall following a major earthquake event, provides the only connection to the rest of Christchurch City. The liquefaction potential of the soils contributed to the vulnerability of the bridge in a seismic event. Construction works to widen and seismically strengthen the bridge began one week prior to the September 2010 Darfield earthquake. No damage occurred due to this earthquake event and construction works continued. The epicenter of the February 2011 Christchurch earthquake was within 5 km of the site. This earthquake caused liquefaction resulting in lateral spreading at both abutments and extensive damage to the superstructure and the partially constructed widening works. Due to the extent of the damage and health and safety risks associated with continuing the strengthening and widening works with continuing aftershocks it was decided to demolish the existing bridge and replace it.

2. Christchurch Earthquake Sequence 2010-2011

On the 4th September 2010 a M_W 7.1 predominantly strike-slip earthquake occurred. The epicenter was some 43 km west of the Ferrymead Bridge site and despite predictions, no significant damage occurred at the site as a result of this earthquake. The

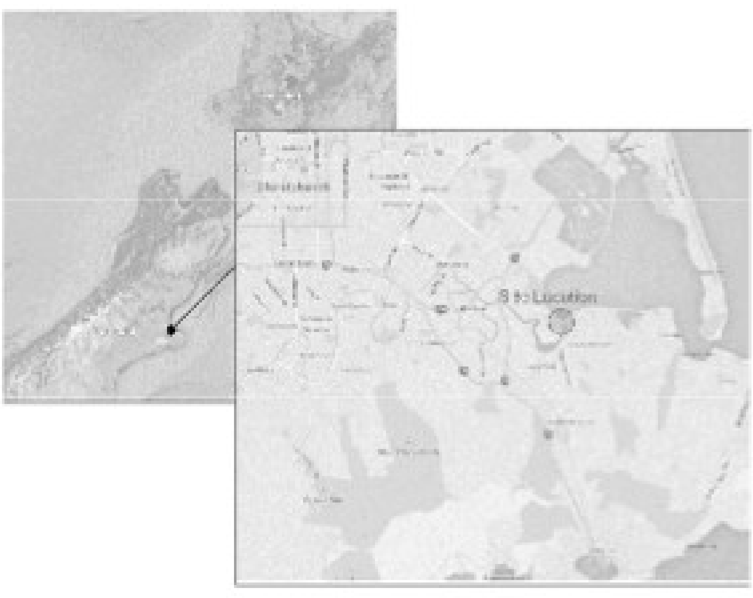

Figure 1. Site location plan

subsequent aftershock sequence also resulted in no significant damage at the site until the 22 February 2011 earthquake (the Christchurch Earthquake) occurred. The epicenter of this M_W 6.2 earthquake was to the southwest of the site and within 5 km, associated peak ground accelerations at the bridge were estimated to be 0.68 g (Canterbury Geotechnical Database, 2013).

At the time of the Christchurch Earthquake all four of the 1.1 m diameter reinforced concrete seismic strengthening and widening abutment piles had been constructed, three with surveyed as-built locations. One of the abutment beams had also been constructed. Construction of the raked pier piles was well under way at one pier.

The Christchurch Earthquake caused extensive liquefaction at the bridge site resulting in lateral spreading of the approach embankments. Loads imposed on the seismic strengthening and widening abutment piles resulted in the piles rotating towards the river with measurements indicating movement at the top of the pile of 0.65 m to 0.74 m. In addition, rotation occurred at both abutments and displacements were obvious at the piers.

The bridge was able to be opened, with speed restrictions, to light vehicles just two days after the damaging earthquake. However, it took more than a year for the bridge to be opened to heavy vehicles such as buses and trucks. Temporary strengthening was carried out to allow this to happen.

3. Site Information

3.1. Geological Setting

Christchurch City is built on low-lying land that was previously swamp, sand dunes, estuaries and lagoons. Banks Peninsula borders the city to the south and is located on the eastern side of Ferrymead Bridge, towards Mount Pleasant. The bridge crosses the river mouth of the Heathcote River near the Heathcote-Avon Estuary on the margin between the low lying land and hilly Mount Pleasant area (Christchurch City Council, 2011).

Geological mapping of the bridge area shows the site is underlain by Holocene aged estuarine sand and silt, which makes up part of the Christchurch Formation. This is underlain by Late Miocene aged Mt Pleasant Formation comprising of basalt interbedded with breccia and tuff. The Mt Pleasant Formation forms part of the Lyttelton Volcanic Group.

3.2. Site Investigations

Due to the difference in design between the seismic strengthening and widening works and the replacement bridge, additional site investigations were required to provide information for design. With the form of the replacement bridge known, site investigations were able to target the proposed pile footprints. As the piles were to be fixed into the rock, the criteria for the termination of the boreholes was based not only on the requirement of finding an appropriate founding layer, but also a minimum depth into rock to provide lateral restraint.

There was to be at least one borehole at each of the 10 pile locations, however due to the large diameter, three boreholes were initially proposed at the pier piles. Restrictions associated with the existing bridge beams resulted in some of the boreholes being shifted so they were no longer within the proposed pile footprint. As a result all of the 10 proposed piles had at least one borehole in close proximity to the proposed pile location. In addition to the boreholes, seven cone penetration tests (CPTs) were carried out, two at each abutment and three at pier pile locations.

Figure 2. Site investigation location plan

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By the time the investigations had been completed, a total of 12 boreholes had been drilled near proposed pile locations, a further five boreholes had been drilled in the approach embankments, and seven CPTs had been undertaken across the site (Figure 2). The additional site investigations were undertaken while the damaged bridge remained open to traffic.

Laboratory testing of the samples recovered was also undertaken, and included particle size distribution tests and unconfined compressive strength testing of the rock with stress stain measurements.

3.3. Ground Model

Development of the ground model relied heavily on the boreholes drilled specifically for the replacement bridge but also used information obtained during piling operations of the seismic strengthening and widening works.

Globally at the site, the ground model consisted of 2 m to 3 m of fill at the abutments overlying estuarine sediments varying in thickness from 6.8 m at the eastern abutment to 20.6 m at the western abutment. Underlying these sediments were the interbedded basalt, breccia and tuff (Figure 3). The breccia was further classified as Agglomerate or Volcanic Breccia based on the composition of the samples recovered and the expected in-situ behavior.

From the borehole results it became apparent there was significant variability within the rock and as such ground models were developed at each specific pile location.

4. Proposed Replacement Bridge Form

For various reasons including the thickness of liquefiable deposits, at the western abutment in particular, the presence of a large number of services and the large costs associated with ground improvement, the solution for the replacement bridge did not incorporate ground improvement. The decision to not improve the ground at the abutments resulted in a need to carefully consider the effects of liquefaction and resulting lateral spread loads and design the foundation system accordingly. In doing so fewer abutment piles and of a smaller diameter were utilised to minimise lateral spreading loads being imposed on the piles and bridge. The bridge has been designed such that lateral loads on the abutment piles are to be resisted by the larger diameter pier piles and the opposite abutment.

The final bridge form is a two span bridge with four large diameter (2.4 m) pier piles and three smaller diameter (1.1 m) piles at each abutment (Figure 4). A landspan at each abutment, which bridges a void behind the abutment piles, has been used to further reduce the lateral spread loads on the abutment piles. Piles have been socketed into the rock to prevent displacement should liquefaction occur.

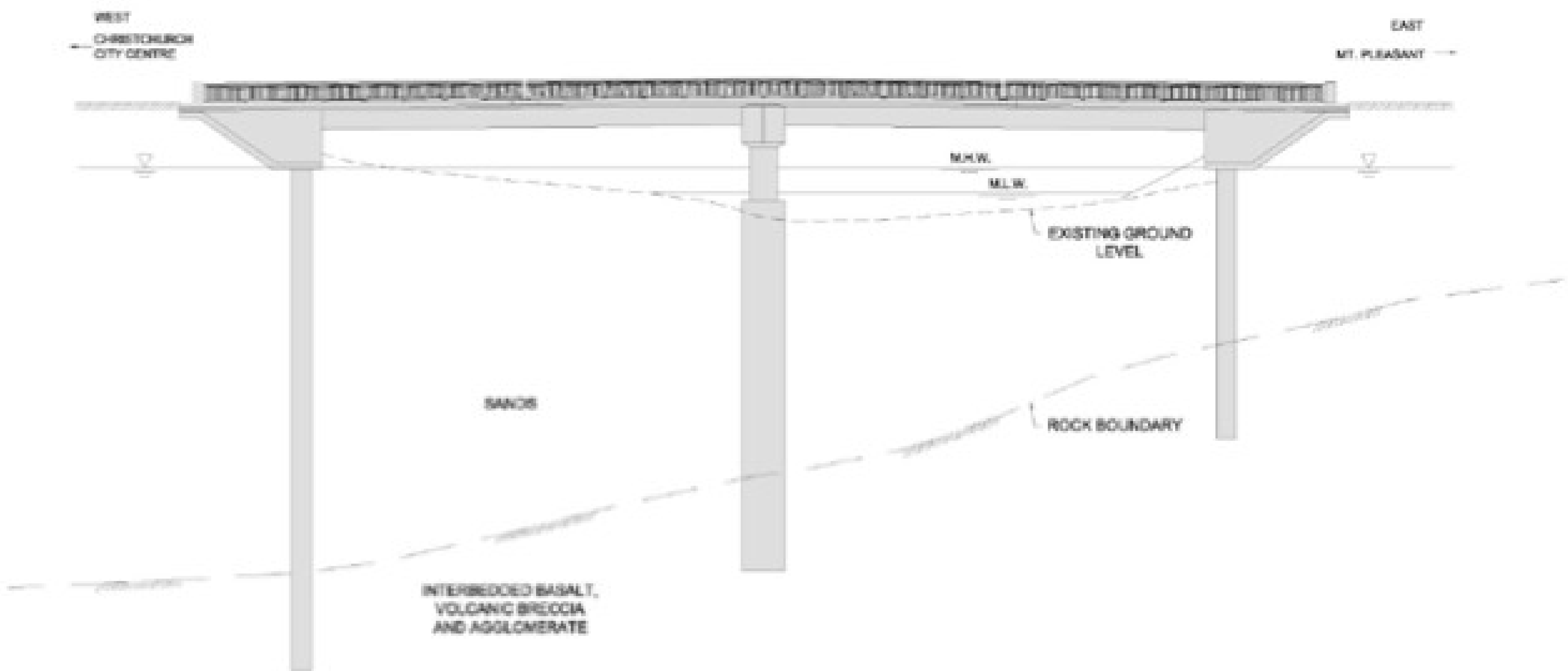

Figure 3. Simplified ground model

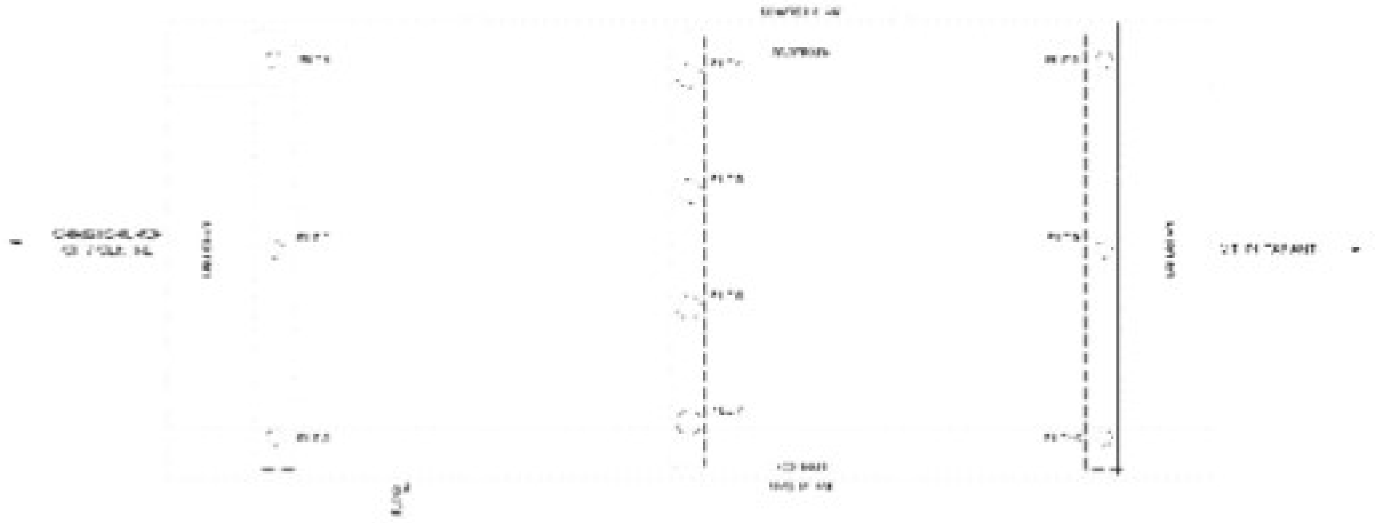

Figure 4. Proposed replacement bridge

5. Geotechnical Issues

As identified in early assessments pertaining to the seismic strengthening and widening of the existing bridge the soils at the site are susceptible to liquefaction. In addition the non-liquefiable crust at the abutments will impose large lateral spread loads at the abutments. The effects of lateral spread loads have been minimised through the use of a landspan in addition to specific construction detailing of the approach embankments (Figure 5). The approach embankments are to be constructed such that they are susceptible to liquefaction below a certain depth, this will reduce the thickness of the non-liquefiable crust therefore reducing the lateral spread loads. As the approaches can easily be reinstated following liquefaction in an earthquake event, the thickness of material susceptible to liquefaction at the approach embankments is unlikely to have long term detrimental effects on the post-disaster functionality of the bridge.

With the replacement bridge philosophy resulting in fewer abutment piles of a smaller diameter, soil is assumed to flow between the piles under cyclic loading. However, the foundations of the previous bridge, which consisted of two rows of closely spaced 0.4 m rectangular raked piles at each abutment, are likely to cause soil to arch between the piles. Arching of the soil will essentially cause the piles to act as a wall and ultimately result in the new abutment piles for the replacement bridge being subjected to much larger loads. In order to reduce the impact of soil arching a number of existing piles were removed from each abutment during demolition of the existing bridge.

As previously alluded to, ground conditions at the site are highly variable. Of particular importance is the variability in the layering and quality of the rock encountered. Strength testing of the rock material indicated a range from 2 MPa to over 100 MPa for the varying rock types encountered. The large variability in rock quality and uncertainty regarding the in-situ behaviour of the volcanic breccia and agglomerate required the piles to be founded in basalt layers of sufficient thickness. In addition to this, it was difficult to distinguish between boulders within the agglomerate material and thin basalt layers. This uncertainty made identifying founding layers, particularly at the abutments, difficult. The risk associated with this uncertainty and the

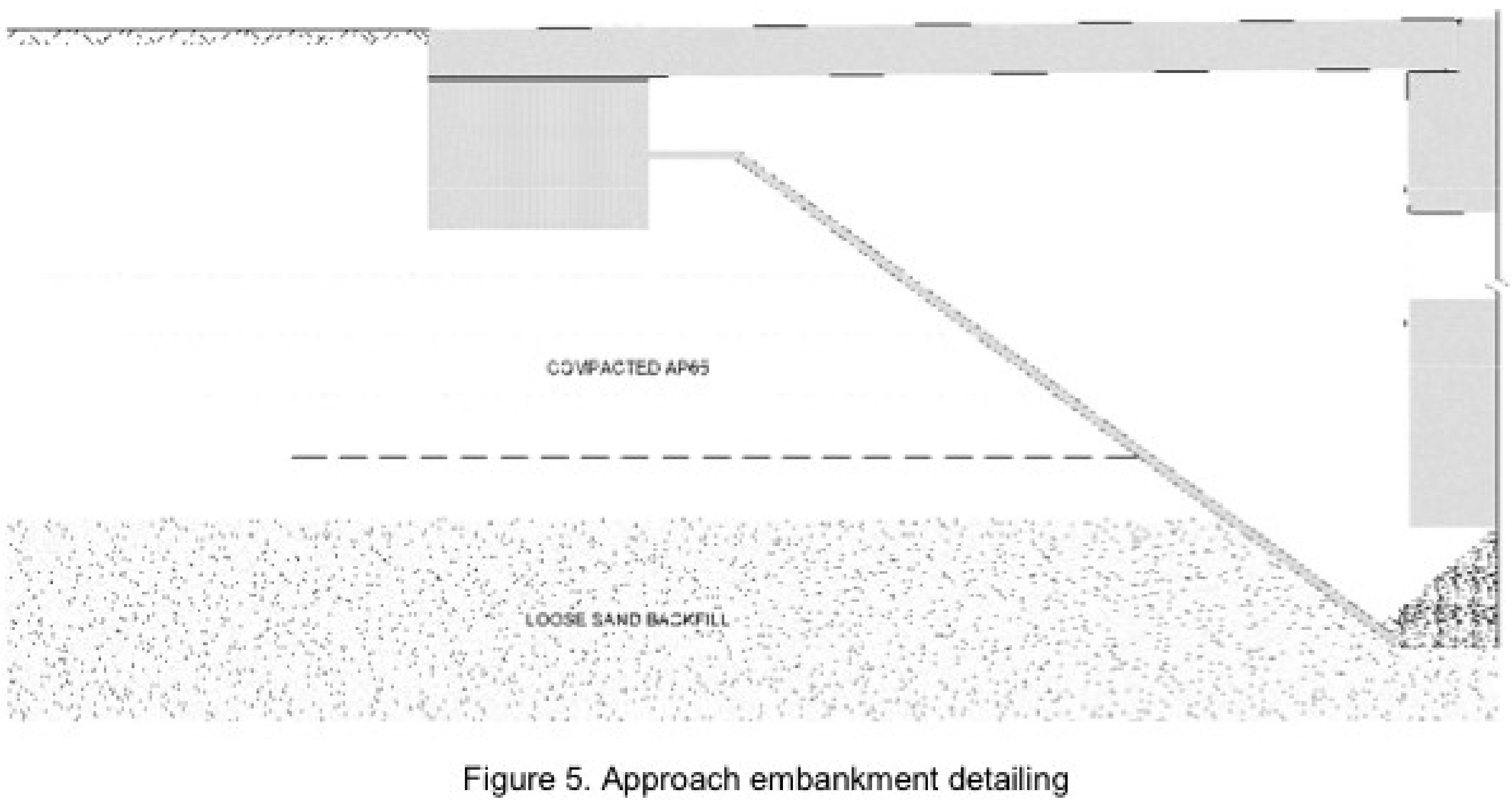

Figure 5. Approach embankment detailing

variable nature of the rock was considered during construction and as such a 1 m contingency of casing was ordered for every pile. The ongoing seismic activity posed a considerable risk to the construction of the replacement bridge. Geological and Nuclear Sciences (GNS) estimated a 9% probability of a magnitude 6.0 to 6.4 occurring in the Canterbury region in the following year (Opus, 2012). During construction, the bridge is more vulnerable to seismic activity, particularly prior to installation of the bridge deck, as loads cannot be transferred to the pier piles as per the design philosophy. The construction sequence was initially programmed to reduce the exposure time and hence the risk to the piles. This was achieved through constructing the pier piles first, as they are further from the river banks and less exposed to lateral spread loads, followed by the eastern abutment piles, which encountered rock at a significantly shallower level and as such were shorter piles, then construction of the western abutment piles. In addition to this sequencing a frame system was designed to prop the abutment piles off the pier piles (Sadashiva et al., 2014). These props were designed to minimise movement of the piles during construction and needed to be installed prior to driving the casing into rock.

6. Piling Operations

Piling for the replacement bridge began in June 2013 at the upstream pier pile, Pile 7 and concluded with the final pour of the upstream eastern abutment pile, Pile 10, at the end of April 2014. While piling was completed two months sooner than initially programmed it wasn't without its challenges.

The variation in ground conditions encountered during investigations was validated during piling operations with information obtained on site indicating some basalt layers dipping up to approximately 45°. Variations in the rock layering encountered, compared to that expected based on the site investigations, resulted in four of the 10 piles having an additional borehole drilled through the base of the pile excavation to prove the founding layer. At one pile in particular the top of the rock level, a critical aspect to the behavior of the bridge under seismic loading, was so much lower than that modelled during design that a revised model had to be analysed to assess the effect on the bridge behaviour.

The large diameter of the pier pile casings made driving a challenge. Piling through the basalt layer allowed excavation of the basalt up to 1 m in advance of the casing. However, due to the uncertainty of the stability of the agglomerate, excavation was only permitted 0.3 m in advance of the casing through these materials. Pile driving through the strong basalt layers required large driving forces which caused damage to the pile driving equipment, as a result the contractor mobilised larger plant. Pile driving through the agglomerate material was less of an issue as the casings were able to be vibrated into the ground relatively easily with minimal vibrations in the surrounding land.

A structural requirement of the design was for good contact between the pile casing and the ground, particularly for the length of pile embedded in rock. To achieve this, grouting of the pile casings was attempted using internal grout tubes and an external method using Tube-A-Manchette (TAM). The purpose of the grouting was to fill the void between the pile casing and the rock formed during excavation of the pile. While there was a significant focus on developing an appropriate method for TAM grouting with the contractor during the design and planning stages it was identified this method would need to evolve as grouting proceeded and more information and experience was gained. A specialist sub-contractor was engaged to undertake the grouting operations. Grouting initially involved internal grouting through tubes installed with the pile reinforcing cage prior to multiple stages of TAM grouting (Figure 6).

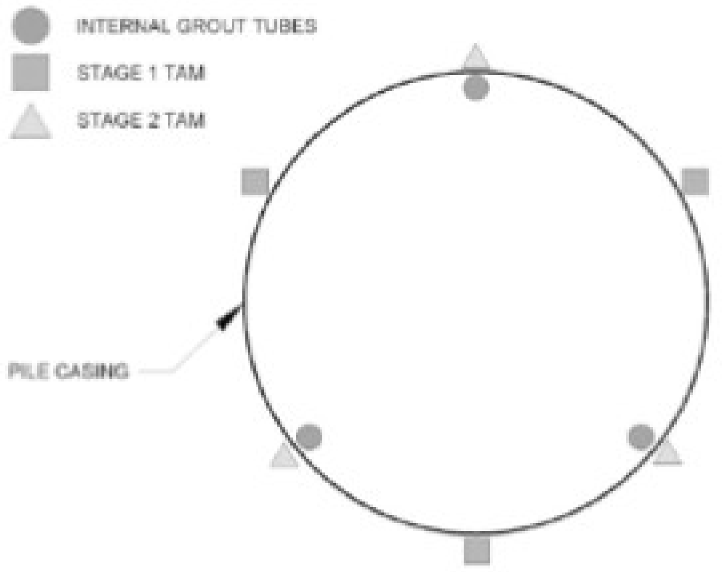

Figure 6. Internal grouting and TAM locations

As internal grouting appeared ineffective during grouting of the first two piles it was stopped and all grouting for the remaining eight piles was undertaken using TAM grouting. TAM grouting was conducted externally with the TAMs installed in boreholes drilled down the outside of the casing once the pile had been poured. Typically six TAMs were installed per pier pile and four TAMs were installed per abutment pile. TAM grouting was undertaken in two stages with half of the TAMs installed per stage and grouted over two to three days on average to ensure refusal was reached. Refusal was based on the pumping pressure reaching a maximum limit set prior to grouting commencing and was taken to indicate the void between the casing and the surrounding rock was full of grout. The volume of grout pumped through each node per day was limited to minimise the risk of losing excessive volumes of grout into the surrounding ground. Maximum grout volumes per node and maximum grouting pressures were selected based on contractor experience and experience gained through the Ferrymead Bridge TAM grouting. These maximum volumes and pressures were the key adjustments made during the grouting operations.

7. Conclusions

Seismic strengthening and widening of the Ferrymead Bridge began one week prior to the September 2010 earthquake, with minimal resulting damage, and works continued. Extensive damage to the bridge and the seismic strengthening and widening works occurred as a result of liquefaction and lateral spreading caused by the Christchurch earthquake. Due to the damage and ongoing seismicity it was decided to demolish and replace the existing bridge.

A proposed replacement bridge form was developed and additional site investigations undertaken to assist with design. Significant geotechnical issues considered during design of the replacement bridge include the soils susceptible to liquefaction and lateral spreading, variable ground conditions particularly the volcanic rock, the influence of the existing bridge abutment piles and the ongoing seismic activity in the area. Solutions to reduce the risk from these issues involved careful consideration of the form of the replacement bridge, with smaller diameter and fewer piles at the abutments, incorporating a landspan to reduce lateral spread loads, removal of a select number of existing abutment piles and careful sequencing and propping of the piles during construction to minimise the risk posed by ongoing seismicity.

Construction challenges encountered during piling operations include the variable ground conditions, which required close monitoring and communication with the contractors, difficulties associated with driving the large diameter pier piles and undertaking pressure grouting to fill the void between the pile casing and the excavation.

Piling works were completed in April 2014, with bridge construction and associated approach works due to be completed in July 2015.

Given the knowledge gained from this project, if another structure, with similar risks and constraints, arose I would employ a similar strategy for the site investigations. This would include boreholes at each of the proposed pile locations, a further borehole at each of the approach embankments, minimal CPTs in the overlying sand and laboratory testing of the rock, with stress and strain measurements.

The only other change worthy of note would be to the grouting methodology. TAM grouting of the piles into the surrounding ground appeared to be effective. However, in the future I would adjust the methodology in the following ways:

- only attempt to grout using external TAMs;
- consider the overburden pressure at the depth being grouted when deciding on a limiting grouting pressure;
- consider the piling contractors methodology when deciding on limiting grout volumes and the likely void space required to be filled with grout; and
- in areas where grout take is high, use more attempts with smaller grout volumes with the aim of sealing the surrounding fractured ground, rather than larger volumes.

The extent of these site investigations and modified approach to grouting should allow adequate design and construction outcomes whilst contributing to an appropriate spread of risk across the entire project team.

8. Acknowledgements

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CALIBRATION OF THE STANDARD PENETRATION TEST FOR LIQUEFACTION ASSESSMENTS

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ABSTRACT – The standard penetration test (SPT) is frequently used in ground investigations as a basis of assessing the relative density and liquefaction potential of granular soils. SPTs are popular due to the fact that they have been used for a long time and a number of accepted empirical correlations for application in design are available. These correlations are based on older SPT hammers which typically had efficiencies around 60%. More modern SPT hammers have much higher efficiencies. These efficiencies need to be measured and allowed for prior to using empirical design correlations. SPT tests are also known to have poor repeatability with variations in energy transfer from different rigs, hammers, operators and other factors. This paper discusses current SPT calibration practices in New Zealand and improvements that could be made. The relevance to a specific site investigation in Wellington is also discussed.

1. Introduction

1.1. Background

The 2010 Canterbury Earthquake and the subsequent aftershocks were a brutal reminder of the importance of creating earthquake resilience in New Zealand. The safety of people is paramount and requires a built environment that is specifically designed to protect lives from this high earthquake hazard. Liquefaction, lateral spreading and ground subsidence during the Canterbury earthquakes caused extensive damage to land and property. Because of this, earthquake effects have been brought to the forefront of people's minds when building, buying or developing property.

A significant sum of money is being invested in developing an appropriate level of resilience. The cost to land and building owners is greatly affected by the outcomes of site liquefaction assessments. Quality ground investigations are crucial to developing options that both provide an acceptable level of earthquake resilience while not being overly conservative resulting in unnecessary costly works.

1.2. Soil penetrometer testing methods

Soil penetrometer testing is commonly used in New Zealand as the basis for assessing the relative density and liquefaction potential of soils. Insitu tests include cone penetration test (CPT) and standard penetration test (SPT) which are commonly available in New Zealand.

CPT has been the preferred method of penetrometer testing for the Christchurch recovery. CPT data is recognized as generally more consistent and repeatable than SPT data. Their relative speed and continuous log make them a more preferable option. However, at some sites the CPT is unable to penetrate through dense sands and gravels.

The SPT is able to penetrate through dense soils and is commonly used in places such as Wellington where this is required. SPT is also popular due to the fact that it has been used for a long time, it is well known and understood and it is inexpensive. Accepted empirical correlations for application in design are available. It also provides samples to visually classify the soil type and/or undertake laboratory testing. A limitation of SPT is that it has relatively poor repeatability with variations in energy transfer from different drill rigs, hammers, operators and other factors.

1.3. SPT correction factors

The output of the SPT test is the value SPT N. This is the measurement of how many SPT hammer blows it takes to push an SPT sampler 300mm through the soil. The effort required for penetration of the soil is used to evaluate the soil's density.

To use SPT data for liquefaction assessments, correction factors are applied to the measured value of SPT N to standardize it. Various correction factors are applied for standardization including energy ratio, borehole diameter, SPT rod length, sampler type and overburden pressure. The energy correction factor takes the efficiency of energy delivered by the SPT hammer and standardizes it to a hammer with a 60% efficiency (SPT N_{60}). This efficiency is used as it was typical of the first SPT hammers. Different types of hammers are currently used. Idriss and Boulanger (2008) reported that the actual delivered energy of current hammers can vary by 30-90%. When the energy efficiency of a trip hammer is not known, it is common practice to assume 60%.

The SPT N_{60} is also normalized to 1 atmosphere effective overburden pressure to become SPT $N_1(60)$. This allows for SPT values measured at different effective stresses to be compared.

In this paper the recommendation of various international standards for SPT practices for the energy correction factor are looked at and compared to current practices in New Zealand. The variation of energy transfer ratios in a sample of calibrations of New Zealand hammers is assessed and the effects of this on a Wellington liquefaction assessment are discussed.

2. Review of requirements for the energy correction factor in SPT standards

Different countries refer to different standards for undertaking the SPT test. Recommendations from the standards used in United States and the international standard provided by the International Organization for Standardization (ISO) are looked at as these standards have been updated in the last few years and are a good representation of quality SPT practice.

2.1. American standards

The following three relevant American standards are available:

- ASTM D6066-11 (2011) which covers the use of penetration testing for evaluating liquefaction potential.
- ASTM D4633-10 (2010) which provides standard test methods for energy measurement for penetration testing.
- ASTM D1586-11 (2011) which provides a standard test method for SPT.

2.1.1. ASTM D6066-11

Recommendations from this standard include:

- For safety hammers, the energy transfer correction ratio can be assumed on smaller investigations.
- Automatic, trip and donut hammers need to have measured energy transfer for the make and model used to perform measurements. Test method D4633 to be used to measure energy transfer.
- Field monitoring to be undertaken of hammer impact velocity and periodic drill rod energy measurements for critical jobs, such as large ground improvements and liquefaction studies associated with expensive structures.
- For routine foundation investigations, visual confirmation of drop heights developed from known operational characteristics is sufficient.
- Automatic hammers are the preferable type of hammer.

2.1.2. ASTM D4633-10

Recommendations from this standard include:

- Yearly calibration of SPT hammers. It is also desirable to calibrate prior to starting major critical projects.
- The force velocity method is the only fundamentally correct method of measuring energy content.

2.1.3. ASTM D1586-11

This standard reports that:

- Variations in SPT N-values of 100% or more have been observed when using different standard penetration test apparatus and drillers for adjacent boreholes in the same soil formation.
- Field experience indicates that when using the same apparatus and driller, SPT N-values in the same soil can be reproduced with a coefficient of variation of about 10%.

2.2. International standards

ISO 22476-3 (2011) is the international standard for field testing with SPT. This standard recommends:

- The calibration of energy transfer should be made annually and after any changes, repairs and modifications to the driving equipment.
- The energy ratio of the equipment has to be known if the SPT N-values are going to be used for the quantitative evaluation of foundations or for the comparisons of results.
- Force-velocity method is the recommended method used to measure energy.

3. New Zealand practice for the SPT energy correction factor

3.1. NZS 4402 (1988)

NZS 4402 (1988) is the New Zealand Standard that covers the use of SPT to determine the penetration resistance of a soil. No conversion from SPT N value to SPT N_{60} is given in this standard. Energy measurements are not mentioned in this standard but it is stated that the results obtained from hammers with self-tripping mechanisms are not comparable with the results from other driving methods such as the rope and cat-head. Details of correction factors to the SPT N value are required to be stated.

3.2. NZGS guidelines

The New Zealand Geotechnical Society (NZGS) has prepared draft guidelines for assessing liquefaction hazards (NZGS, 2010). This guideline states that the SPT limitations include the potential to overlook susceptible soil strata, relatively poor repeatability and operator dependence. It advises that results should be carefully interpreted and corrected according to the recommendations of Youd et. al. (2001).

Youd, et. al. (2001) recommend that measurement of the hammer energy is undertaken frequently at each site where the SPT is used. Where measurements cannot be made, careful observation and notation of the equipment and procedures are required to estimate an energy

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correction factor for use in liquefaction calculations. ASTM 1586-99 standard (now superseded) is recommended for SPT testing procedures.

Youd, et. al. (2001) note that even when procedures are carefully monitored to conform to established standards that some variation in energy transfer occur because of minor variations in testing procedures. Measured energies at a single site indicate that variations in energy ratio between blows or between tests in a single borehole typically vary by as much as 10%.

3.3. MBIE Christchurch rebuild guidelines

Guidelines prepared by New Zealand's Ministry of Business, Innovation & Employment (MBIE) for the Christchurch rebuild, require properly energy rated equipment to be used for ground investigations on TC3 land (land categorized as requiring site specific engineering design).

3.4. Earthquake Commission (EQC) geotechnical investigation project

An extensive post-earthquake geotechnical investigation was undertaken in Christchurch. According to Fairclough and Ashfield (2013) the investigation area covered approximately 28,000 residential properties. SPT tests were undertaken where ground conditions caused shallow refusal of CPT. SPT hammer efficiency was identified to be an important input for using SPT test results in liquefaction assessments. Several SPT hammers were tested in accordance with the recommendations of ASTM D4633-10.

Fairclough and Ashfield (2013) concluded that SPT hammer efficiency should be measured on each rig deployed to ensure reliable and robust analysis is completed.

3.5. Current frequency of SPT hammer energy calibration in New Zealand

Information on SPT hammer efficiency measurements from a sample of drilling companies operating in New Zealand was collected for this paper. The frequency of calibration was inferred from:

- Information verbally supplied by the companies.
- A comparison between the date of calibration certificates provided for SPT hammers and the date they were requested by an engineer for a sample of projects.

A wide range of calibration frequencies was found, with no typical frequency. Out of the drilling companies that measure the energy efficiency of their hammers, the most frequent calibration was every 3 months and the least frequent was 7 years. Not all drilling companies in New Zealand had calibrated their SPT hammers.

4. Variation between energy measurements of New Zealand SPT hammers

SPT hammer calibration measurements of a sample of New Zealand rigs was collected. The sample consisted of 7 automatic hammer calibrations and 16 trip hammers. 13 results were carried out following ASTM D4633-10 standard. 10 results were undertaken under 'refusal conditions' and therefore do not follow ASTM D4633-10 standard. However, they still use the force-velocity method. For the purposes of this paper the different calibration methods has not been considered. Therefore this sample provides only an indication of the energy variation. Results are shown in Table 1.

A significant range of energy transfer ratios is found (48% to 91%). Automatic hammers in this sample were found to have a smaller range than trip hammers.

Fairclough and Ashfield, (2013) reported that during the EQC geotechnical investigation project, SPT hammer calibrations were undertaken in accordance with ASTM D4633-10. For a sample of 8 automatic hammers an average of 85.4% was found with a range of 52.5% to 101.3%. For a sample of 6 trip hammers an average of 65.9% was found with a range of 56.1% to 85.6%.

Some of these SPT calibration test values from the samples in the Fairclough and Ashfield (2013) paper are included in the sample investigated in this paper. The values not included were not available for this paper.

5. Case study – The importance of SPT hammer energy on a specific liquefaction assessment in Wellington

5.1. Background

This case study is of a typical low-lying site in the Wellington area. The importance of the variation in energy transfers of SPT hammers to this case study is considered.

The Wellington region is situated within highly folded and faulted sandstone, siltstone and mudstone.

Table 1. SPT hammer energy measurements for a sample of New Zealand hammers

Hammer Type	Average energy transfer ratio	Coefficient of variation for energy transfer of hammers	Minimum energy transfer ratio	Maximum energy transfer ratio
All hammer types	72%	16.6%	48%	91%
Automatic hammers	85%	6.4%	74%	91%
Trip hammers	67%	14.2%	48%	86%

In the subject area, thick deposits of marginal marine sediments comprising mainly sands with lesser silts and gravels overlie the greywacke bedrock.

The site is located in a low lying area with uplifted greywacke hills on either side. Groundwater is encountered approximately 3m below ground surface. A number of active and inactive faults are close to the site.

During ground investigations for this site SPT tests were carried out. A CPT test was also attempted but was unable to be completed due to the dense ground conditions near the surface. The first 17m of marginal marine deposit was found to to contain layers of fine to coarse sand with gravel. This sand had varying density, with SPT N values ranging from 12 to 50+. Refer Table 2 for soil profile at this site.

5.2. Liquefaction trigger

Idriss & Boulanger (2008) presents the mechanism of liquefaction trigger and the factors that affect whether liquefaction occurs or not. Seismic liquefaction occurs when excess pore pressures are generated in loose, saturated, cohesionless soil and low-plasticity silt during earthquake shaking, causing the soil to undergo a temporary partial to complete loss of shear strength. Such a loss of shear strength can result in ground failures, while the post-cyclic loading reconsolidation of the soil leads to ground settlements. Whether a soil liquefies is dependent on several factors including the intensity and duration of ground shaking, soil density, particle size, and groundwater location. Soils which are susceptible to liquefaction require a certain level of earthquake shaking (trigger) to cause them to liquefy. Denser soils require more intense and/or longer duration of shaking (higher trigger) than less dense soil.

Methods have been developed to correlate SPT N values to earthquake shaking intensity required to trigger liquefaction, for example Idriss & Boulanger (2006). These methods can be used to assess the return period of an earthquake that could be expected to trigger liquefaction at a site. Refer Figure 1 for an example of an evaluation of liquefaction triggers at the case study site.

Table 2. Soil profile for case study

Layer No.	Description	Depth to top of layer (m)	Layer thickness (m)	SPT N (blow/300mm)
1	FILL	0.0	0.0 – 0.5	-
2a	MARGINAL MARINE DEPOSITS Fine to coarse sand with gravel (medium dense)	0.3 – 0.5	2.5 – 3.5	12 – 32
2b	MARGINAL MARINE DEPOSITS Fine to coarse sand with gravel (dense/very dense)	3.0 – 4.0	10.0 – 14.0	30 – >50
2c	MARGINAL MARINE DEPOSITS Fine sand with gravel (dense to medium dense)	13.0 – 17.0	0.4 – 3.0	17 – >50
3	MARGINAL MARINE DEPOSITS Sandy silt (stiff/very stiff)	15.3 – 18.0	14.0	9 – 30
4	MARGINAL MARINE DEPOSITS Organic peat layer (Firm to stiff)	31.0	2.0	-
5	MARGINAL MARINE DEPOSITS Inter-bedded layers of fine to medium sand & silt with some sand (dense)	33.0	7.5	45 – 46
6	RESIDUAL SOIL Silt with some gravel (Hard)	40.5	4.5	33
7	BED ROCK Greywacke	45.0	–	–

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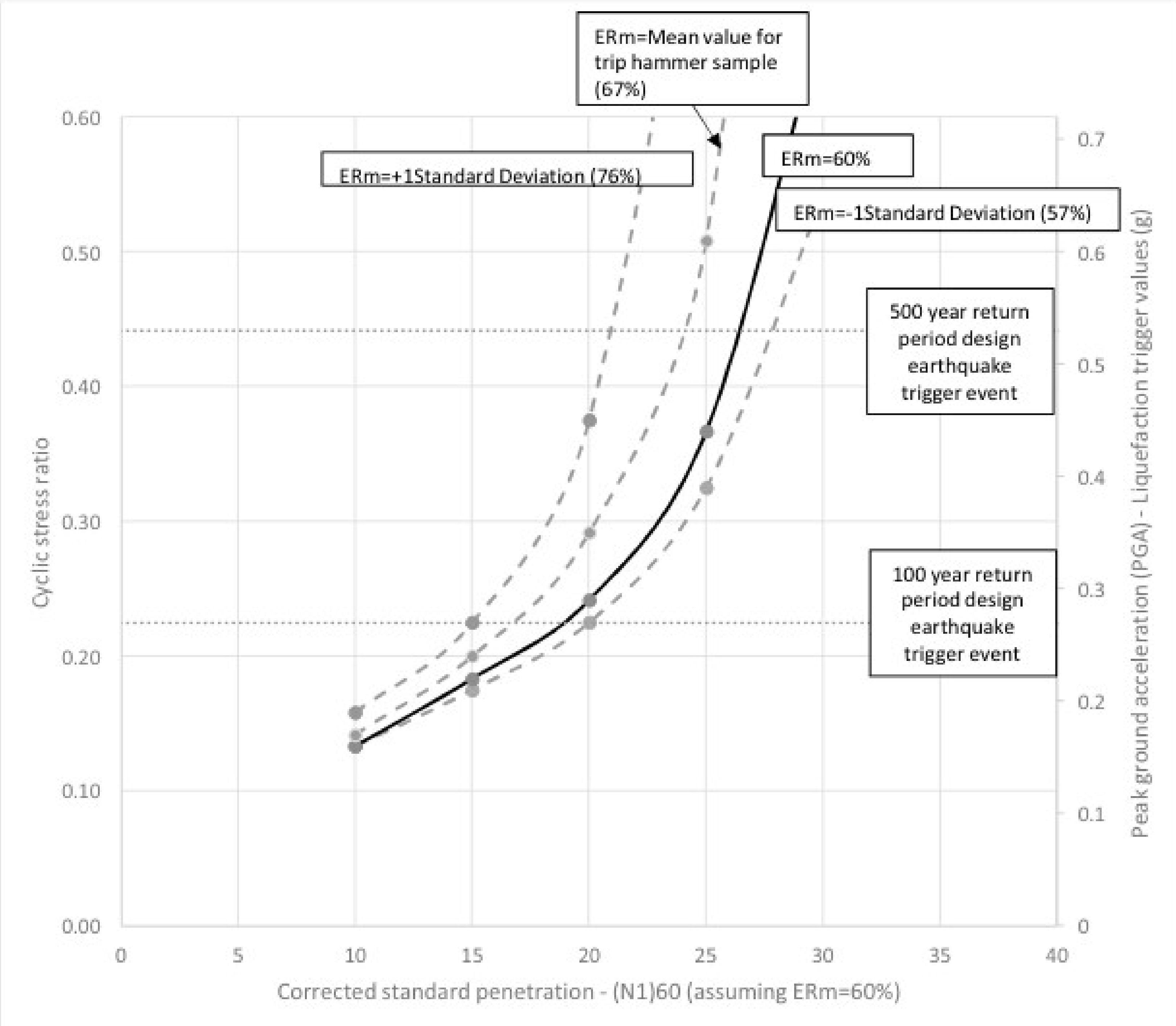

Figure 1. Liquefaction trigger values for a part of the Wellington marginal marine deposit at case study site. [Based on Idriss & Boulanger (2006), Fines Content ≤ 5%; Earthquake Magnitude = 7.5; δ'_{vc} = 1atm]

5.3. Importance of SPT hammer energy variation for case study

Layer 2a from the soil profile (fine to coarse sand with gravel) is potentially liquefiable. This layer has an SPT N range of 12-32, with typical values of 17-25.

Figure 1 shows that using the Idriss and Boulanger (2006) method with the typical assumed energy transfer ratio of 60% has different implications for the range of SPT values found in Layer 2a.

For an SPT N value of 12 and an assumed energy transfer ratio of 60%, the peak ground acceleration (pga) to trigger liquefaction is assessed to be 0.18g. There is only a minor difference in the assessed pga to trigger liquefaction if the SPT hammer has a different energy transfer ratio than the assumed 60%.

For an SPT N value of 17 and an assumed energy transfer ratio of 60%, the pga to trigger liquefaction is assessed to be 0.24g. There is a moderate impact on the assessed pga to trigger liquefaction if the SPT hammer has a different energy transfer ratio than the assumed 60%. If the SPT hammer energy transfer ratio was 67% the pga to trigger liquefaction is assessed to be 0.27g. Assuming an energy efficiency of 60% for this situation could lead to unnecessary conservatism in the liquefaction assessment. For a 100 year design earthquake event, this is the difference between triggering liquefaction and not.

For an SPT N value of 25 and an assumed energy transfer ratio of 60%, the pga to trigger liquefaction is assessed to be 0.44g. There is a large impact on the assessed pga to trigger liquefaction if the SPT hammer has a different energy transfer ratio than the assumed 60%. If the SPT hammer energy transfer ratio was 67% the pga to trigger liquefaction is assessed to be 0.62g. For a 500 year design earthquake event, this is the difference between triggering liquefaction and not.

The current practice of assuming an energy efficiency of 60% for a trip hammer is shown to be generally conservative as 60% is close to one standard deviation below the mean energy for trip hammers. However efficiency values as low as 48% were recorded and for values as low as this, assuming 60% would lead to an unconservative assessment. Liquefaction triggering could be determined as unlikely to occur when the opposite should be predicted.

A hammer energy transfer ratio is shown to make a difference to this case study for SPT values from 17 to 25. This is due to the larger spread in predicted pga values to trigger liquefaction evident in Figure 1 for higher SPT values.

The hammer efficiency for this case study was measured and found to be 63% and so assuming an energy efficiency of 60% had little impact on the design conclusions.

6. Conclusions

The following conclusions have been drawn:

- The energy efficiency of SPT hammers varies widely.
- A wide range of SPT hammer calibration frequencies exists in New Zealand.
- The New Zealand Standard for SPT states that different hammer types are not comparable.
- International standards recommend at least annual calibration of SPT hammers and project specific calibration for critical projects.
- Energy correction can have a significant impact on liquefaction assessment especially for SPT values between 17 and 25.
- When no energy correction is available the current practice of assuming 60% is generally conservative. However as efficiency values as low as 48% were recorded, uncalibrated hammers should not be used to prevent unconservative liquefaction assessments. For assessments that lead to expensive remedial works, confidence of the hammer efficiency could lead to considerable cost savings.

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Geotech Drilling Ltd, Griffiths Drilling Ltd., McMillan Drilling Ltd., Websters Drilling and Exploration Ltd were helpful in providing information used in this paper.

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A WELL GeoCONNECTED DESIGN: A HARD ROAD TO A SOLID FOUNDATION

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ABSTRACT – The NZ Transport Agency's Waterview Connection project is one of the largest and most important infrastructure projects to be undertaken in New Zealand. The project will see 5km of 6 lane motorway constructed through Auckland's Western suburbs, linking two existing motorways to complete a ring route around the city. The project centrepiece is a 14m diameter Earth Pressure Balance Tunnel Boring Machine, used to minimise the impact on surrounding communities. A considerable amount of complex and challenging surface works must also be completed at both ends to connect the tunnels into the motorway network. Geotechnical engineers for the Well-Connected Alliance have contributed to the design of the tunnel, bridges, trenches, embankments, diversions (stream, road and utilities), ancillary buildings and spoil disposal. One of the major geotechnical design challenges was to integrate detailed design packages with other designs that were still at concept stage. Often geotechnical designs were constrained by the needs of other design packages. Having to interface with another Alliance delivering the adjoining SH16 Causeway upgrade added to the challenge and shaped many of the geotechnical design issues on the project, including the design of a 90m long by 4m high retaining wall.

1. Introduction

The Waterview Connection project will see 5km of six lane motorway constructed through Auckland's Western suburbs. This will link State Highway 20 (SH20) to State Highway 16 (SH16) and complete a ring route around the city.

Half of the motorway link will be constructed underground using a 14m diameter Earth Pressure Balance Tunnel Boring Machine (EPB TBM). This will minimise the impact on the communities surrounding the project.

The first part of Auckland's South-western motorway network was constructed in the 1980's. The project has extensive planning history with consultation for the project occurring in the early 2000's.

The project will be delivered through an alliance model. The Well-Connected Alliance (WCA) comprises of the New Zealand Transport Agency, Fletcher Construction, McConnell Dowell, Obayashi Corporation, Parsons Brinckerhoff, Beca and Tonkin and Taylor.

This paper will provide a project overview, a brief summary on some of the major geotechnical works and a discussion on the challenges of designing a Mechanically Stabilised Earth (MSE) retaining wall.

2. Project Overview

Currently all traffic travelling through Auckland is reliant on State Highway 1 (SH1) to provide a motorway connection between north and south Auckland. During peak hours, congestion on SH1 cause significant delays to commuters, freight and tourists.

The western ring route will provide an alternative motorway for through traffic and alleviate congestion on SH1.

Figure 1. Location of the Waterview Connection.

2.1. Construction Zones

The project is geographically divided into three construction zones the southern zone, the tunnel and the northern zone.

2.1.1. Southern Zone

The southern zone extends from the end of SH20 at the Maioro Street interchange through to the tunnel portal openings. The construction work comprises of:

- a connection to SH20;
- an expanded Maioro Street interchange;

- a motorway and pedestrian overbridge;
- The southern approach trench (SAT) to launch the TBM; and
- Realignment of Oakley creek and other major underground infrastructure.

2.1.2. Tunnel Zone

The tunnel zone extends from the Alan Wood Reserve in Owairaka to Waterview adjacent the existing SH16 motorway. The construction work consists of

- Two twin bored tunnels 2.4km long and 13.1m in diameter;
- 16 cross passages connecting the tunnels; and
- Associated mechanical and electrical works

2.1.3. Northern Zone

The northern construction zone links the tunnels to the existing SH16 motorway. The construction work consists of

- Northern approach trench; and
- Four new ramps at the great north road interchange to connect the tunnels to SH16.

3. Overview of Geotechnical Works

The WCA geotechnical engineers have contributed to the design and construction of permanent works such as the tunnels, bridges, retaining walls, embankments and ancillary buildings.

3.1. Southern Approach Trench

The southern approach trench (SAT) is located in the Alan Wood Reserve Owairaka, which is part of the southern zone of works. The SAT is designed to provide the necessary space for reassembly of the TBM that had been shipped in pieces from the factory in Nansha, China. The SAT is approximately 400m long and 29m deep during temporary excavations at the portal sump.

The trench face forms the southern portal where the TBM will begin its first bored tunnel and complete the second bored tunnel.

The SAT is constructed on a circa 100,000 year old basalt lava field sourced from the Mount Albert volcano approximately 1km to the northeast (Wansbone et al, 2013). This strong basalt rock (UCS 25-120 MPa) overlays much weaker East Coast Bays Formation (ECBF) (UCS 1-5MPa) located at depth.

As the TBM is designed to tunnel exclusively through ECBF rock and Tauranga Group soils it was critical to accurately map the basal contact of the basalt lava flow in three dimensions. From a program of ground investigations, a 3D model was developed in 12D and imported into geometric software MX (Wansbone et al, 2013).

The SAT has two separate retention systems for the basalt and the ECBF. Due to the different rock mass types within the basalt, various support types were used to retain the basalt. For poorer quality rock a combination of rock bolts steel mesh and shotcrete was employed. For blocky columnar jointed basalt, fibre reinforced shotcrete was used in combination with rock bolts.

To retain the Tauranga Group and ECBF below the basalt, bored piles with up to three rows of multi strand anchors were used. These were installed through the reinforced capping beams and walers.

Piles were spaced at a maximum of three diameters with drainage and shotcrete lagging installed between piles. By designing the retaining walls as drained, reduced groundwater pressures significantly reduced the structural demands on the retaining wall. (Wansbone et al, 2013).

Figure 2. Aerial view of SAT looking north on 7 August 2013

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3.2. Bored Tunnels

The centrepiece of the project is the twin 2.4km bored tunnels. These tunnels are designed to accommodate three lanes of motorway traffic in both the northern and southern directions. The southern extent of these tunnels is the SAT in Alan Wood Reserve and the NAT in the north adjacent the Waterview Reserve. At their deepest the tunnels are approximately 45m below ground and have a bored diameter of 14.5m with an internal diameter of 13.1m.

The tunnels are linked together by sixteen cross passages. These cross passages provide storage for maintenance purposes, house tunnel electrical and mechanical systems and provide emergency exit pathways from the tunnels.

To minimize disruption to the surrounding residential neighbourhoods, bored tunnels using an earth pressure balance (EPB) TBM was selected as the preferred method of tunnelling. An EPB TBM is able to reduce ground surface settlements around the tunnel excavation area by maintaining the pressure on the excavated face. This is achieved through a pressurized compartment directly behind the cutterhead. Spoil from the excavation face is stored in this compartment and used to maintain pressure on the face of the excavation.

Figure 3. Temporary steel segements installed at cross passage opening.

The tunnel will be lined with 2m wide precast concrete segments that will be assembled within the tailskin of the TBM. The rings comprise of nine segments and a key. For the most part the tunnels pass through sandstone (ECBF) of varying strength and weathering, except at the northern portal where the tunnel will be partially within Tauranga Group soil (Well-Connected Alliance 2013c).

Depending on ground conditions and alignment depth, three different types of reinforcement were used within the segmental lining. The cross passages however are not lined with segments and will be mechanically excavated, in a series of advances. Temporary steel segments are installed at the cross passage opening and are then removed later to begin excavation. Shotcrete is applied to the excavated rock lining for support. In areas of poorer quality rock bolts will be installed to provide extra support.

3.3. Great North Road Interchange

A critical part of the project, aside from the tunnels, is the Great North Road interchange (GNRI). The interchange will connect SH 20 onto SH 16. The GNRI comprises of 4 ramps with associated earthworks embankments and retaining walls.

The typical geological profile at the GNRI area consists of Puketoka Formation (alluvium) overlaying weathered and unweathered ECBF. Approximately 28,000 to 30,000 years ago the nearby Mount Albert and Mount Roskill volcanoes erupted. The resulting lava from the eruptions ponded in the valleys of the Oakley Creek catchment. The basalt flows have formed a lobe of basalt in the interchange area that continues into the Waitemata Harbour (Well-Connected Alliance, 2012a).

The construction of these ramps comprises of pre-stressed super T beams integrally supported on cross heads with single piers and piles. The piles are 2.1m diameter bored piles. Where there is sufficient basalt, pad footings will be used instead. (Well-Connected Alliance, 2012b)

There are six retaining walls in the GNRI area. Retaining walls 501, 502 and 503 are mechanically stabilized earth (MSE) walls using stone strong blocks for facing. Retaining walls 601, 602 and 703 use a combination of soil nails, bored piles and ground anchors.

The MSE walls have been designed to transition traffic from the ramps onto SH16. Stone strong blocks were selected as the preferred facing method, as the constructors were able to install them efficiently. Tensar RE geo grid was used as the reinforcement behind the walls.

Figure 4. Aerial view of the Great North Road Interchange and northern retaining walls looking north.

4. RW 503 Design Challenges

Retaining wall 503 (RW503) is located on the western end of Ramp two (Ramp 2). Ramp 2 takes northbound traffic from the tunnel westbound on SH16. The ramp is unique as the traffic from ramp two will merge into the inside lane of SH16 which is referred to as "the flip option".

The retaining wall itself is 90m long and approximately 4m high. It overlaps the Ramp 2 abutment wingwalls and therefore the walls are not connected and are essentially two separate walls. It retains crushed basalt excavated from the SAT excavation.

The retaining wall is constructed with stone strong blocks which are 2.2m long, 1.1m deep and 900mm high and have an exposed aggregate facing. Two rows of Tensar Re geogrid are cast into the back of the block in 0.46m lifts. The grid placed behind the blocks is connected to the cast in grid via a bodkin joint. Due to the large size of the blocks and the gradually diminishing retained height, a special taper top block was developed that could vary in height between 1020mm and 100mm. The taper top block is placed on top of the topmost stone strong block and used to follow the gradient of the ramp down.

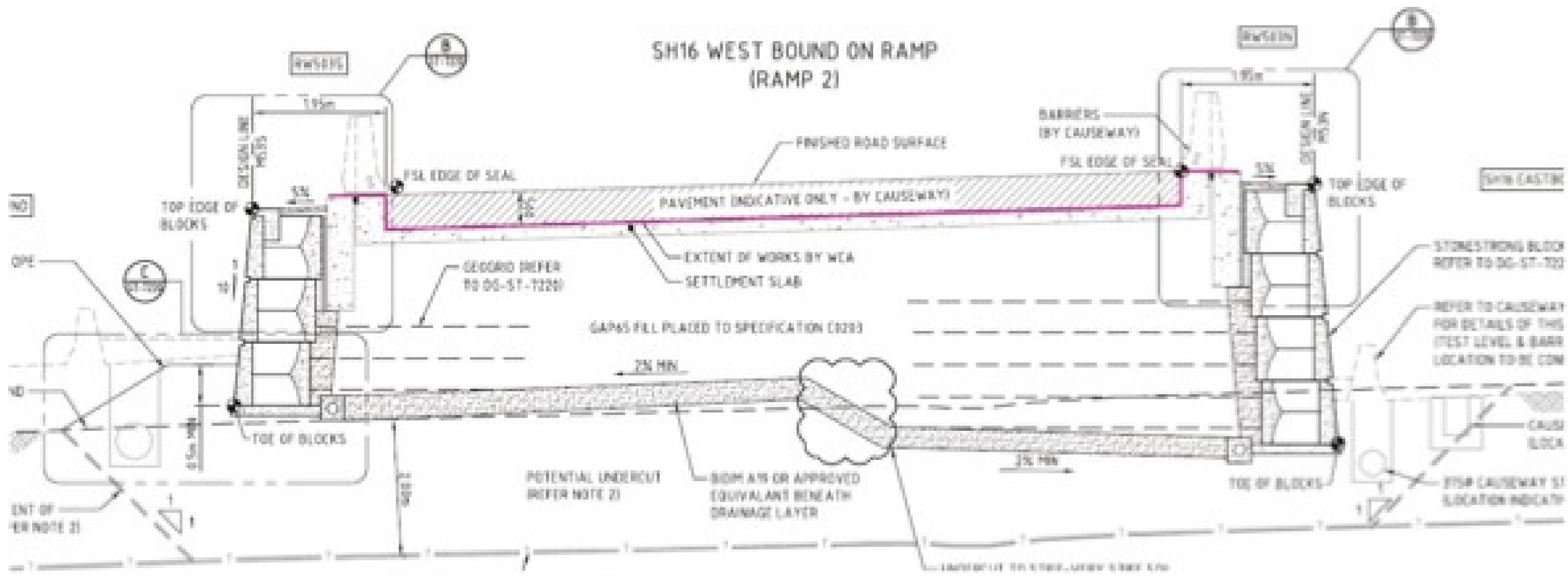

Figure 5. Showing a cross section through Retaining Wall 503.

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One of the major challenges on the project was being able to integrate design packages from different disciplines. Often design packages were at different stages of the design process. This makes the design process challenging when a detailed design package requires input from a design package still at conceptual stage. To overcome this, the WCA designers had to be proactive in engaging with other design processes.

Figure 6 Stone Strong block placement for RW502.

4.1. Flip Option

During Total outturn cost (TOC) Ramp 2 was proposed to enter SH16 on the left-hand side of the motorway like a conventional onramp. After TOC was completed it was then proposed that it was more appropriate for Ramp 2 to enter SH16 on the right-hand side of the carriageway. The main reason for this change was to prevent lane weave on SH16. The change in alignment required a retaining wall (RW503) to be built as the ramp landing was now located between the two SH16 carriageways. A challenge in the design of RW503 was the alignment and geometry of the retaining wall wasn't completely confirmed at the start of detailed design. This required close interaction with geometric designers to confirm the retaining wall string lines that were being used for the design of RW503 were accurate.

4.2. Retaining Wall 503 Settlement Issues

One of the challenges for the design of RW503 was confirming that the settlement in the carriageway meets the Project's Minimum Requirement of less than 15mm differential settlement between the settlement slab of the piled Ramp 2 abutment and the approach embankment. This proved quite challenging as the geological profile underneath the retaining wall was 5m of basalt overlying approximately 10m of Tauranga Group soils. Typically adopted analytical methods calculated significantly larger settlements than allowed for in the Minimum Requirements.

Due to RW503 being located in the carriageway of the existing SH16 getting investigation data was very challenging. Geotechnical drilling could only take place at night due to the high traffic volumes on the motorway. The drilling was also time-consuming because of the 5m of basalt above the Tauranga group. It required the drillers to change their drilling equipment once they had drilled through the Basalt and reached the Tauranga Group. This was to ensure representative samples of the Tauranga Group soil could be collected for laboratory testing. The specific borehole was completed at the proposed Ramp 2 abutment and push tubes were undertaken. Laboratory 1-dimensional consolidation tests could then be completed to inform the design process.

A Plaxis model was developed to estimate how the load of the approach embankment would be rafted through the basalt layer and onto the Tauranga group. This involved using Plaxis to provide an average vertical stress onto the top of the Tauranga Group soil. The model confirmed that the cumulative effect over the 25 year maintenance life would exceed these Minimum Requirements.

When the alliance requests a change to the minimum requirements, a departure must be raised. In the case of RW503 a departure was raised in respect to a requirement of less than 15mm differential settlement between the approach embankment and the abutment.

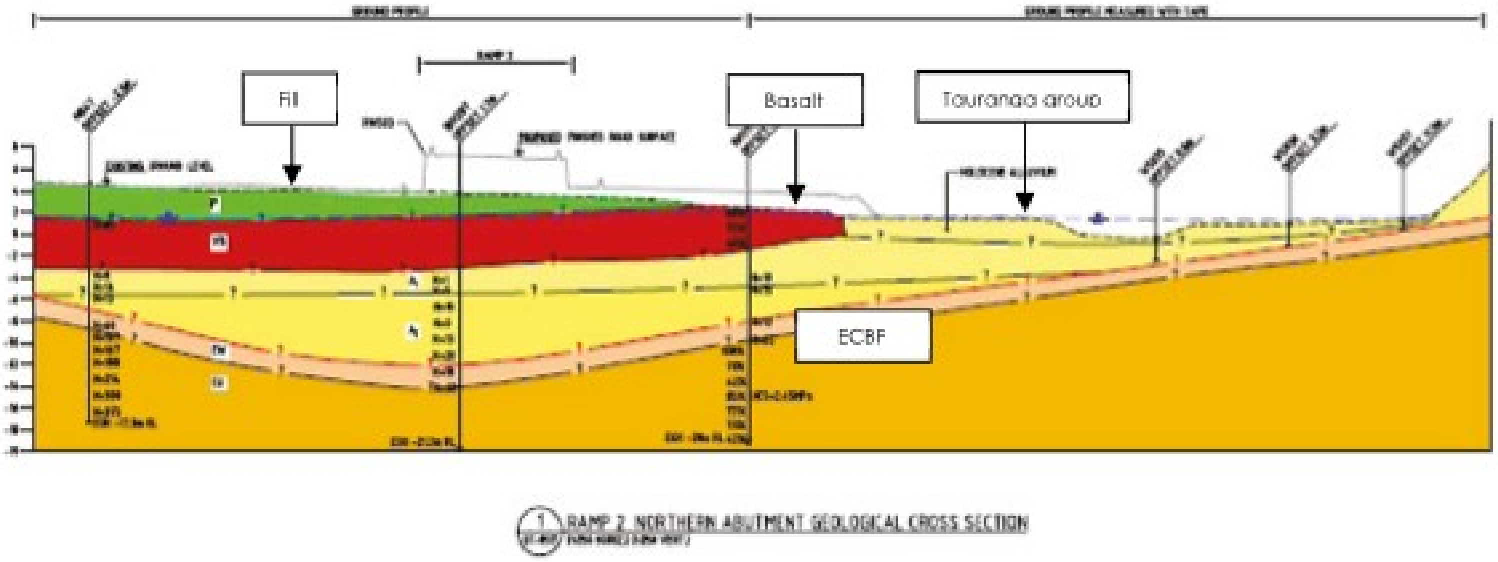

Figure 7 showing geological cross section for RW503

The solution that was presented in the departure centred on the ramp pavement known as Open Graded Porous Asphalt (OGPA). OGPA is resurfaced every 7 years at a maximum and during this scheduled maintenance, minor settlement can be corrected with the OGPA. The alliance could prove that there would be no more than 15mm differential settlement over the 7 year lifecycle of the OGPA and on that basis the departure was approved.

4.3. Interfacing with the Causeway Alliance

Another significant challenge was the integration of the design of retaining wall 503 with the Causeway Alliance (CA). The CA is responsible for raising and widening the causeway which is part of SH16. Retaining wall 503 was unique as it not only had a horizontal interface with the CA but it also had a vertical interface.The WCA was responsible for constructing the retaining wall while the CA was responsible for raising SH16 around the retaining wall and constructing the pavement, barriers and lighting of the ramp.

The Waterview Connection project required close interaction between different design disciplines. The retaining walls at the GNRI area are a good example requiring significant input from geotechnical, civil, structures, urban design, construction and maintenance.

This created challenges, as the project often involved working in confined spaces where design clashes between disciplines were common issues. It was crucial to have communication between the design disciplines and review work packages at the completion of each design stage. It also highlighted the importance of having sound review processes so that potential clashes did not go unnoticed. A good example of this was the stormwater drainage around retaining wall 503 by the CA.

The CA needed to install stormwater pipes at the toe of retaining wall 503. This was picked up early in our detailed design through providing the concept plans to the CA for comment. The pipes were to be located within close proximity. Due to the risk of the retaining wall toe collapsing into the open trench, bracing of the open trenches would be required. This would have been a significant cost to the CA and ultimately the NZ transport agency. By working closely with the CA it was possible to accommodate the stormwater pipes through changing the construction staging around the retaining wall. The stormwater pipes would be installed first followed by the retaining wall itself.

5. Conclusions

The NZ Transport Agency's Waterview Connection project is one of the largest and most important infrastructure projects to be undertaken in New Zealand.

The WCA geotechnical engineers have contributed heavily to the design and construction of structures such as the SAT, tunnels and the GNRI.

A significant challenge for design engineers on any large project is being able to integrate design packages from different design disciplines at different stages of the design process. To achieve this, it becomes crucial to be proactive in engaging other design disciplines and having sound design review processes in place before the design program begins.

6. Acknowledgements

The Author would like to acknowledge the New Zealand Transport agency and the WCA for permission to showcase this project.

The support of my colleagues, in particular Sian France (Associate – Hydrogeology, Beca) and Grant Newby (Technical Director – Geotechnical Engineering, Beca) has been much appreciated and key in the composition of this paper.

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PILE LOAD TESTING AND VALIDATION OF FOUNDATION DESIGN ON THE MACKAYS TO PEKA PEKA EXPRESSWAY PROJECT

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ABSTRACT – A regime of bored pile load tests was developed for validation of design assumptions around the capacity and behaviour of large diameter (2.1 – 3.0 m) bridge piles on the MacKays to Peka Peka (M2PP) Expressway project. Delivered by the M2PP Expressway Alliance, whose main partners are the NZ Transport Agency, Beca, Fletcher Construction and Higgins, the project is one of eight sections of highway improvements that form the Wellington Northern Corridor road of national significance, in the lower North Island of New Zealand. The expressway comprises 18 km of new four lane highway, crossing peat swamps and sand dunes. Given the high seismicity of the region, some of the ten expressway bridges are designated lifeline structures: designed to provide resilient emergency access into Wellington in the event of a natural disaster. This paper discusses the pile load test (Osterberg method) at the Waikanae River Bridge: measured vs. predicted performance, validation of design approach and considerations for later load tests.

1. Introduction

Large diameter (2.1 – 3.0 m) reinforced concrete bored pile foundations are being used to support the piers of the multi-span bridges on the MacKays to Peka Peka (M2PP) Expressway project. Four of these bridges have a mono-pile pier arrangement as shown in Figure 1 and, as per AS2159 (2009), are classed as a low redundancy system with little ability to redistribute loads. As such, the need to confirm design assumptions around the *axial capacity* of these pier piles was identified early on in the design.

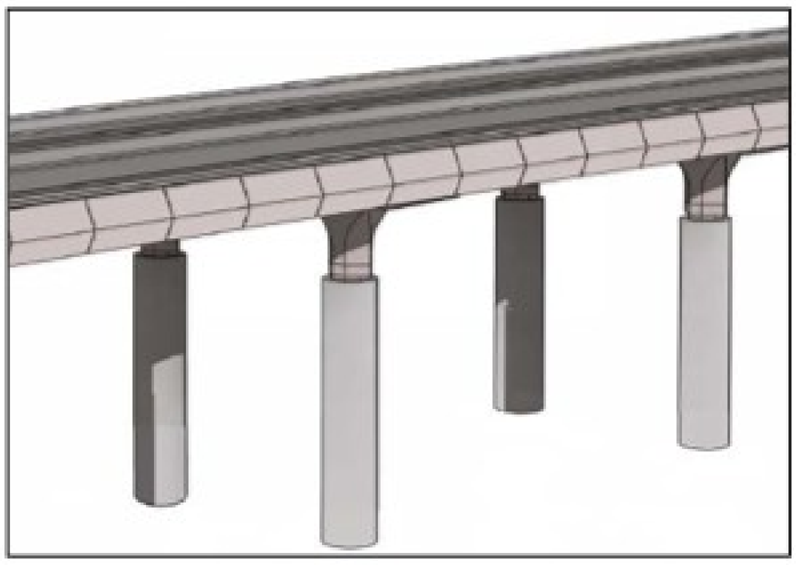

Figure 1. Simply supported, mono-pile bridge structural arrangement (decks not connected).

Three bi-directional load tests, undertaken using the Osterberg method, on sacrificial reinforced concrete bored piles have been proposed and located to reflect three different site soil conditions:

i) Waikanae River Bridge – founding strata comprises mostly gravels interbedded with silt,

ii) Te Moana Road Overpass – founding strata comprises partly sand and partly gravels interbedded with silt, and

iii) Wharemauku Stream Bridge – founding strata comprised entirely of sand.

The first of the load tests was undertaken on a 2.1 m diameter pile at the site of the Waikanae River Bridge on 11 and 12 March 2014. The results of this pile load test have been used to validate design assumptions and have provided valuable considerations for the two subsequent tests.

2. Project Description

The 18 km long, four lane M2PP Expressway is a realignment of State Highway 1 through the townships of Paraparaumu and Waikanae, located in the lower North Island of Zealand (Figures 2 and 3). The project is one of eight sections of highway improvements that form the Wellington Northern Corridor road of national significance; a strategic route key to the economic growth of the Wellington region and providing resilient emergency access to the capital in the event of a natural disaster. The NZ Transport Agency, Beca, Fletcher Construction and Higgins are the main partners of the M2PP Expressway Alliance, the consortium currently designing and constructing the route.

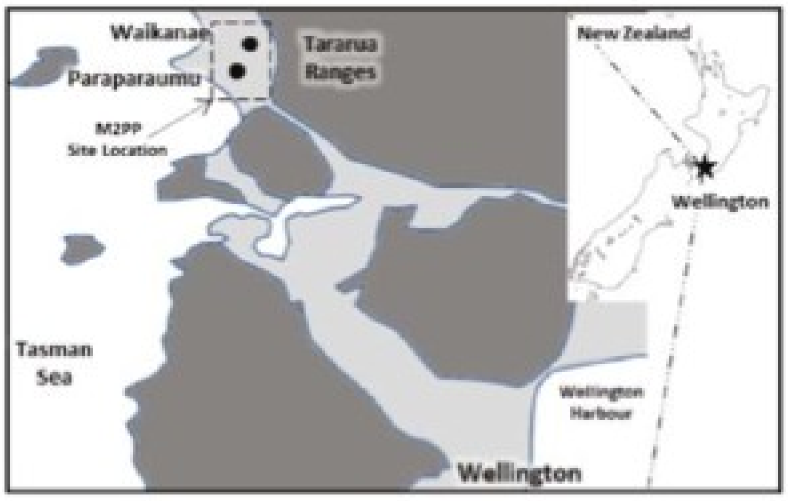

Figure 2. Site location.

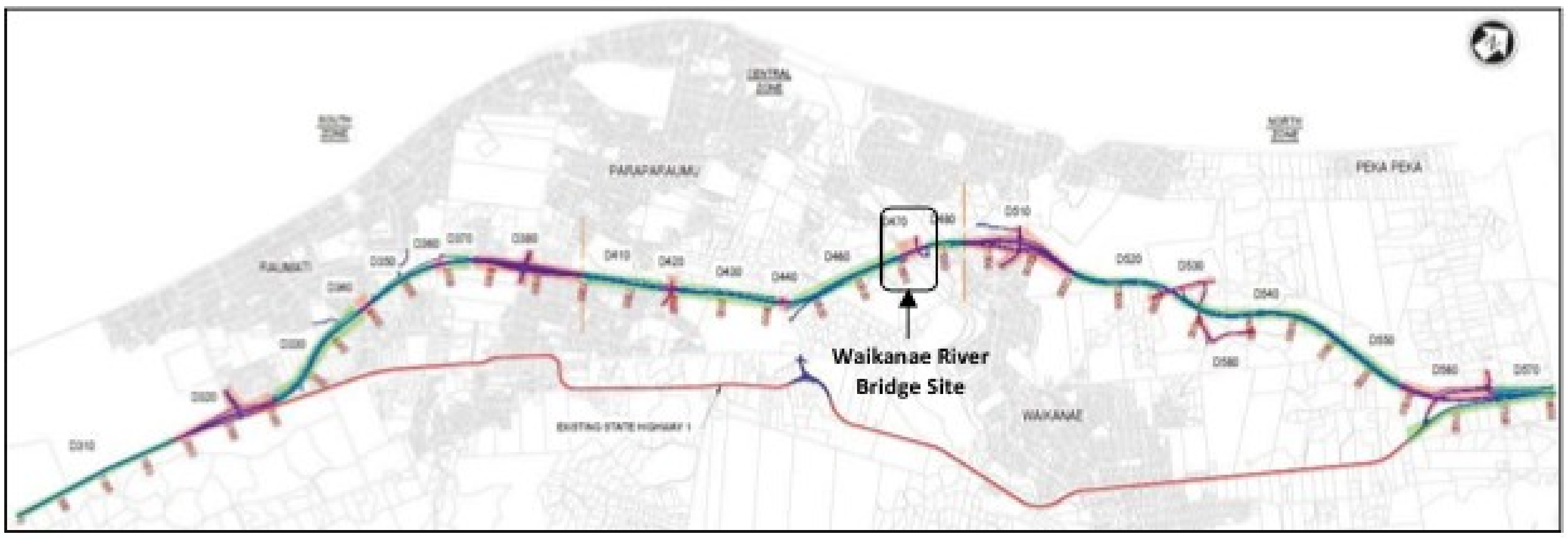

Figure 3. Expressway alignment and location of the Waikanae River Bridge.

3. Site Conditions

3.1. Geological Setting

The expressway route crosses the coastal plain to the west of the Tararua Ranges (Figure 3), in an area that has been shaped by repeated cycles of glaciation that have occurred in the past two million years. The Tararua Ranges, being steep greywacke (metamorphosed sandstone) hills formed through uplift and tilt of basement rock along northeast-southwest oriented faults, held valley glaciers during the glacial periods which resulted in severe erosion and generation of alluvial fans and floodplains. Eroded material deposited during glacial and interglacial periods along the coastal plain up until the last glaciation (some 10,000 – 15,000 years ago) form a thick wedge above the greywacke basement rock, known as the Pleistocene wedge. Deposited estuarine sands and gravels, dune sands, peat and river gravels above the Pleistocene wedge are Holocene-age materials.

The stratigraphic relationship between the materials encountered at this site is best illustrated in Figure 4.

3.2. Seismicity

The lower North Island of New Zealand is a seismically active region, producing large magnitude earthquake events. Although no major active faults are mapped passing directly through the expressway site, splinters of these faults are present. The design seismic event for bridge design has a 2500 year reoccurrence interval with a moment magnitude of 6.9, producing a peak ground acceleration of 0.68 g. All saturated granular Holocene deposits and medium dense to dense Pleistocene sands on the site are expected to liquefy under this high level of seismic shaking.

3.3. Ground Conditions at the Waikanae River

The site of the Waikanae River Bridge is underlain mostly by dense to very dense Pleistocene gravels interbedded with Pleistocene silt. The soil units present at the test pile location are summarised in Table 1 below, in the form of a general soil profile.

Table 1. Waikanae River Bridge soil sequence.

Layer	Depth (m bgl) [1]	Soil Unit	Deposition Type
1	0 – 4.7	Holocene sand / gravel	Interglacial (<10000 yrs)
2	4.7 – 12.8	Pleistocene sand	Interglacial
3	12.8 – 21.9	Pleistocene gravel	Glacial
4	21.9 – 22.6	Pleistocene silt	Interglacial (marine)
5	22.6 – 29.3	Pleistocene gravel	Glacial
6	29.3 – 31.0	Pleistocene silt	Interglacial (marine)
7	31.0 – 32.9	Pleistocene gravel	Glacial
8	32.9 – 34.7	Pleistocene sand	Interglacial
9	34.7 – 39.9	Pleistocene gravel	Glacial

[1] m bgl = meters below ground level.

Project-wide geotechnical parameters for the materials encountered beneath the expressway have been derived from correlations to numerous *in situ* tests, laboratory testing and, where appropriate, data from published sources. The interpretation of the pile load test is based on these established strength parameters and has not refined them specifically for the Waikanae River Bridge site.

4. Axial Pile Capacity – Design Philosophy

For the four bridges that have a mono-pile pier arrangement, the reinforced concrete bored piles are large in diameter (2.1 – 3.0 m) to accommodate the large horizontal seismic actions on the structure.

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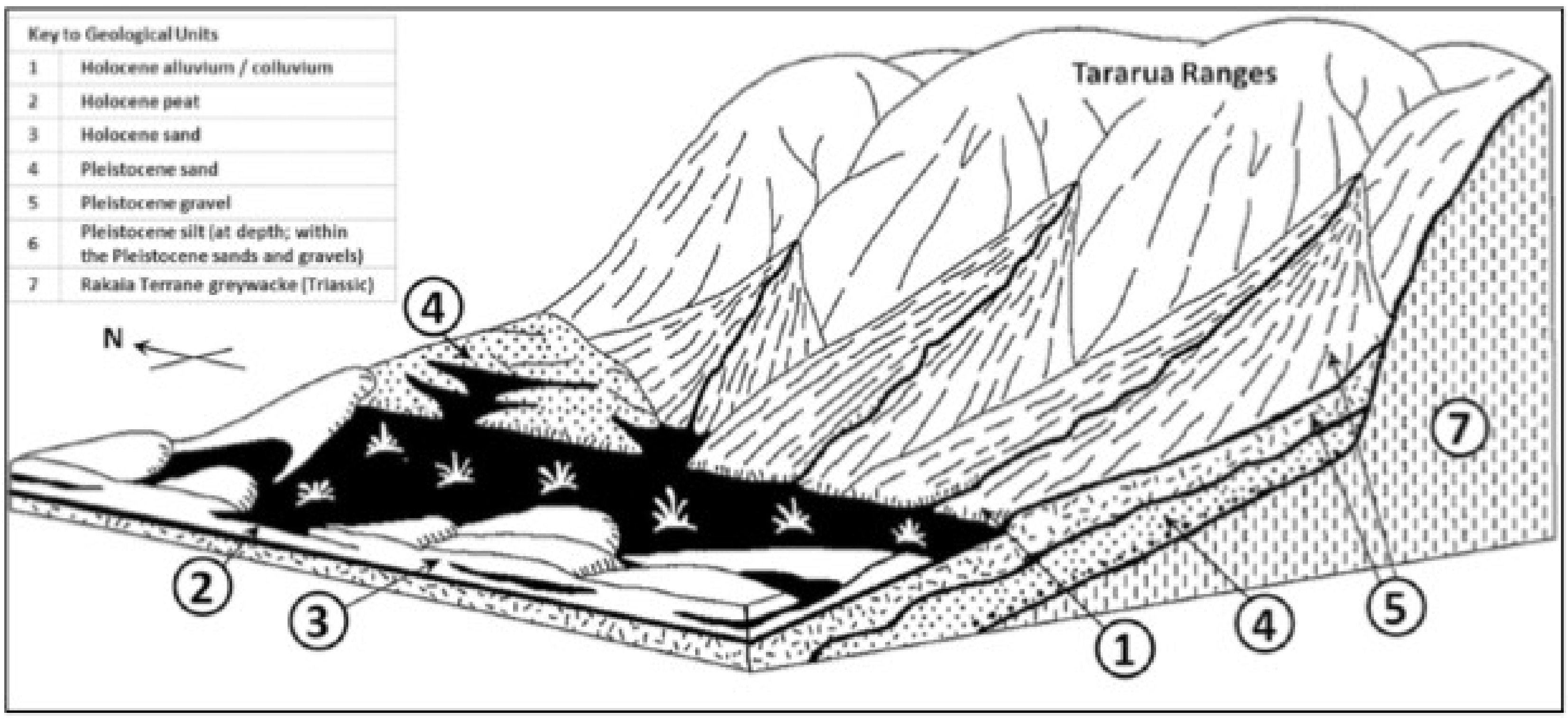

Figure 4. Isometric cross section of encountered materials at the M2PP Expressway and their relationship (modified from Maclean & Maclean, 2010).

The form of these multi-span bridges (separate decks for each carriageway, single pier per pile and simply supported deck; Figure 1) means that there is effectively no frame action. Critical axial load combinations are dead load in combination with live load and liquefaction induced negative skin friction.

Based on the scheme assessment for structures on the expressway, a five diameter clearance between the pile toe and any silt layers within the founding strata was to be provided (M2PP Alliance, 2011) for piles designed to derive their axial capacity in end bearing. This was to provide adequate stress reduction through the competent material before bearing onto silt layers.

From geotechnical investigations undertaken for the detailed design phase of the project, extending to greater depths than previous investigations, a greater prevalence of silt layers in the founding strata was identified at multiple locations along the alignment. The irregular distribution of the silt and hence difficulty in predicting their occurrence makes end bearing design difficult. Both avoiding silt layers within the stress bulb around the pile toe and providing certainty around pile founding levels made shaft resistance-only (skin friction) design an attractive option. Consequently, a skin friction axial capacity design has been adopted for determining the ultimate geotechnical capacity of bored pile foundations on the project.

The ultimate capacity, provided by skin friction alone, is computed using the following relationship:

$$V_u = V_{su} = \int_0^L C(\sigma_v^{'} K_s \tan\delta^{'} + \alpha s_u).dz \qquad (1)$$

The $\sigma'_v K_s \tan\delta'$ term is the adhesion against the pile surface from frictional soils and αs_u is adhesion from cohesive soils.

The self-weight of the pile is ignored in the calculation of capacity because it is assumed that this is satisfactorily carried in end bearing.

The way material properties are used in the design is affected by assumptions around:

i) The coefficient representing the horizontal effective stress in terms of vertical effective stress (K_s); taken as $K_s = 2.0$,

ii) The interface angle of shearing resistance (δ'); taken as $0.67\varphi'$ based on published correlations, and

iii) The adhesion factor (α); taken as $\alpha = 0.4$, based on published correlations, for all cohesive deposits whether Holocene or Pleistocene.

The design $K_s = 2.0$ is optimistic compared to the conventional $K_s = K_0$ (at rest earth pressure coefficient) relationship (Figure 5). The formation history of the materials that underlie the expressway is the basis for selection of this value: repeated cycles of glaciation, faulting, aging and cementation, resulting in overconsolidation of these materials.

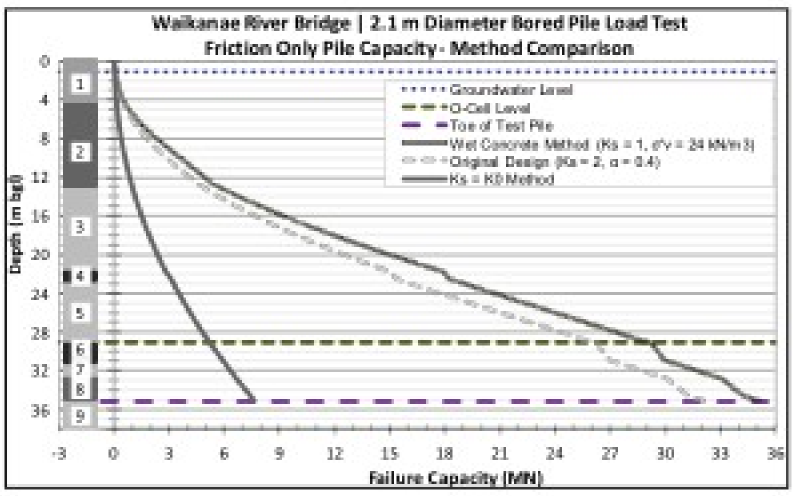

Figure 5. Estimation of skin friction capacity – method comparison (layering as per Table 1).

In terms of the load test, it is the product of K_s and δ' that is measured, though the relative proportions of K_s and δ' remain unknown. In the interests of consistency, only the K_s value has been selected for modification where a change in the product of K_s and δ' is warranted.

For reference, Figure 5 also includes the design capacity computed using the weight of wet concrete method (K_s = 1.0 and σ'_v = 24kN/m^3 inputs to Equation 1).

5. Test Pile Details and Loading Sequence

An sacrificial reinforced concrete bored pile – approximately 35.2 m long and 2.1 m diameter – was installed at the site of the Waikanae River Bridge by the M2PP Expressway Alliance and instrumented / tested by Fugro Singapore Pte. Ltd.

5.1. Instrumentation

An assembly consisting of two 510 mm Osterberg cells (O-cells) was provided at a single location near the toe of the pile (base of assembly at approximately 29 m bgl; three diameters above the pile toe). With reference to Figure 6, the following instrumentation was provided:

i) A pair of automated digital survey levels for measuring top of pile displacement;

ii) Seven levels of four vibrating wire strain gauges above the O-cell assembly, attached at 90° spacing within the reinforcing cage;

iii) Four telltale rods at 90° spacing monitored by displacement transducers (attached to the top of the pile) for measuring shaft compression above the O-cell assembly;

iv) Four displacement transducers for measuring expansion of the O-cell assembly;

v) Three levels of four vibrating wire strain gauges below the O-cell assembly, attached at 90° spacing; and

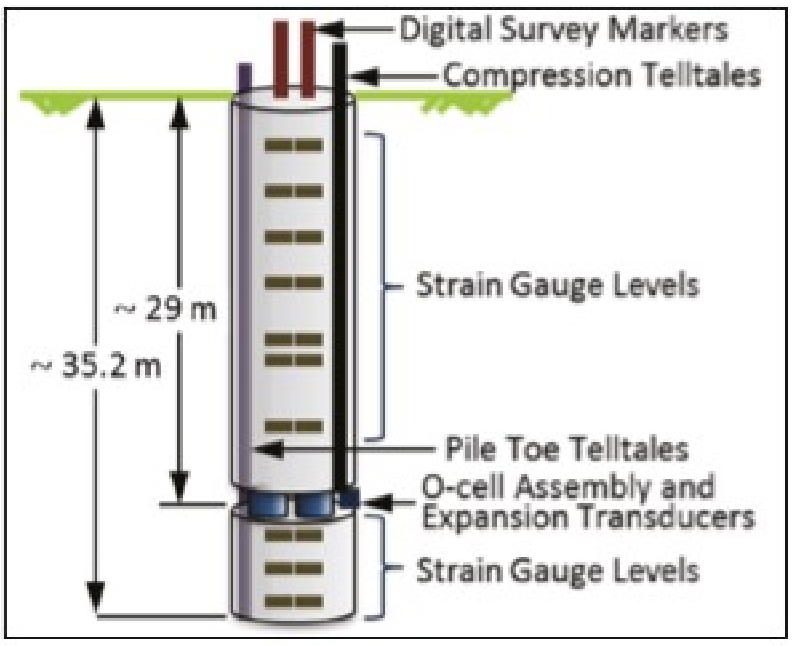

Figure 6. Indicative instrumentation arrangement (not to scale).

vi) Four telltale rods at 90° spacing monitored by displacement transducers (attached to the top of the pile) for measuring pile toe displacements.

5.2. Loading Sequence

A loading sequence was developed in general accordance with AS2159 (2009), adapted for use with an O-cell load test. Each successive load increment was held constant for either 20 minutes or until the creep criteria was met (0.5 mm per 15 minutes), whichever was longer. The load increment assessing pile serviceability (~10 MN) was held for four hours and the increment assessing the design ultimate capacity (~17.9 MN) for one hour.

Load increments were applied past the design ultimate capacity level with a view to assessing the failure capacity of the pile. For this load test, the limits of the equipment were reached at a maximum load of 28.73 MN. The ultimate load capacity of the pile was not reached.

6. Test Results

6.1. Above the O-cell Assembly

It is assumed that the O-cell does not impose an additional upward load until the expansion force exceeds the buoyant unit weight of the pile section above the O-cell assembly; calculated at 1.43 MN (based on an assumed 24 kN/m^3 unit weight for reinforced concrete).

At the maximum applied upward net load of 27.3 MN, the pile head moved 14.55 mm upward with an average shaft compression of 4.02 mm. Therefore, total displacement immediately above the O-cell assembly was 18.57 mm.

6.2. Below the O-cell Assembly

The O-cell applied a maximum downward load of 28.73 MN and at this loading, the displacement immediately below O-cell assembly was 49.78 mm. The pile toe was displaced downward by 48.05 mm and hence compression of the pile shaft in the lower portion was 1.73 mm.

7. Analysis of Results and Discussion

The results of the load test above the O-cell assembly have been interpreted and discussed in this paper; being relevant to skin friction axial capacity design.

It is worth noting that although this load test was undertaken to prove skin friction pile capacity, the results provide useful information for end bearing design considerations and also assessment of axial stiffness. These however, are beyond the scope of this paper.

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7.1. Determination of Pile Modulus

The conversion of strain to load within the pile, between each strain gauge level, requires a pile modulus (AE; cross sectional area of pile multiplied by concrete stiffness).

Derivation of pile modulus was a two-step process:

i) Using the laboratory concrete stiffness (reported to be 30.35 GPa on the day of the pile load test; AE = 105 GN) to determine the theoretical load at the O-cell level (46.6 MN), and

ii) *Pro rata* the laboratory-based pile modulus by the amount required to reduce the theoretical load in step i. above to the actual applied test load (28.73 MN) (an approximately 40 % reduction).

The reduced pile modulus derived in this manner for strain gauge interpretation was approximately 64.8 GN and agrees well with Fugro's interpretation which used a back-analysis taking into account O-cell loads and compression telltale measurements (Fugro, 2014).

Thus, in the load test, the observed stiffness of the concrete (on which the observed pile modulus relies) is less than the value determined from laboratory testing. The elastic modulus of concrete is a function of the strains imposed in the pile as a result of loading and the load applied to the pile reduces as a result of load shedding into the soil surrounding the pile shaft (Fellenius, 2001).

7.2. Comparison of Design and Measured Values

Three approaches had been taken in comparing design values with load test results:

i) Overlaying theoretical versus interpreted capacities,

ii) Overlaying theoretical versus interpreted rolling average cumulative skin friction capacities, and

iii) Overlaying individual theoretical and interpreted unit skin frictional capacities.

Analysis of unit skin friction values (approach iii. above) was selected as the most appropriate method of assessing if changes to design assumptions were necessary. The distributions of measured unit skin friction and calculated values are shown in Figure 7.

7.2.1 Interpretation for Frictional Soils

The key input to skin friction capacity derivation for granular soils is the combination of K_s and δ' (as defined and described in section 4).

In the current design philosophy, the unit skin friction values in the upper layer of Pleistocene sand (layer 2 in Figure 7, between 4.7 and 12.75 m bgl) are overestimated. A decreased K_s = 0.7 has been determined for this material to bring theoretical unit skin friction in general agreement with measured values.

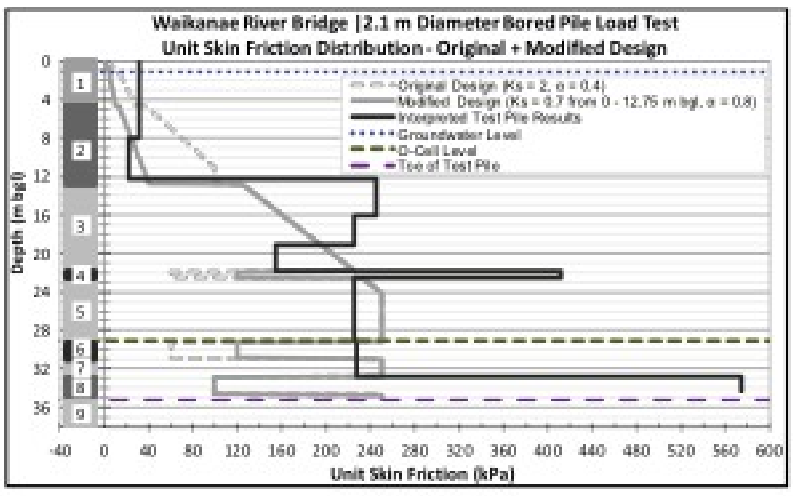

Figure 7. Measured, original design and modified design unit skin friction (layering as per Table 1).

It is likely that this upper layer of Pleistocene sand is not as heavily overconsolidated as the older, deeper Pleistocene deposits below 12.75 m bgl. In addition, the design K_s for Holocene materials have also been revised down to K_s = 0.7.

It is worth noting that the strain gauge level from which deductions around K_s have been made (Figure 7, ~12.3 m bgl) has been identified as potentially unreliable. It is located in the same material as the strain gauge at ~8.1 m bgl (Figure 7), in a zone with greater confining pressures, and yet indicates lower unit skin friction values. Considerations around strain gauge reliability and further investigation around the upper Pleistocene sands will be undertaken in the design of the next load test.

No change to K_s values for deep Pleistocene sands deposits are proposed based on the higher unit skin friction values calculated for this material between 32.9 and 34.7 m bgl (layer 8 in Figure 7).

In the Pleistocene gravels below 22.6 m bgl (layers 5 and 7 in Figure 7), measured unit skin friction is less than design estimates. Analysis of load steps preceding the ultimate indicated that unit skin friction was continuing to increase and therefore these soils do not appear to have reached failure.

7.2.2 Interpretation for Cohesive Soils

The unit pile skin friction for cohesive soils corresponds to the product of α (defined and described in section 4) and the undrained shear strength (s_u).

The results of the load test indicates an underestimation of either s_u or α, given that the product of these terms underestimates adhesion of the Pleistocene silt by a factor of at least two.

As per section 3.3, geotechnical parameters are not refined as part of this interpretation and hence the assumed α factor for Pleistocene silt has been increased to α = 0.8.

7.2.3 Interpretation of Capacity

An update of Figure 5, showing the theoretical capacity using the modified design parameters and

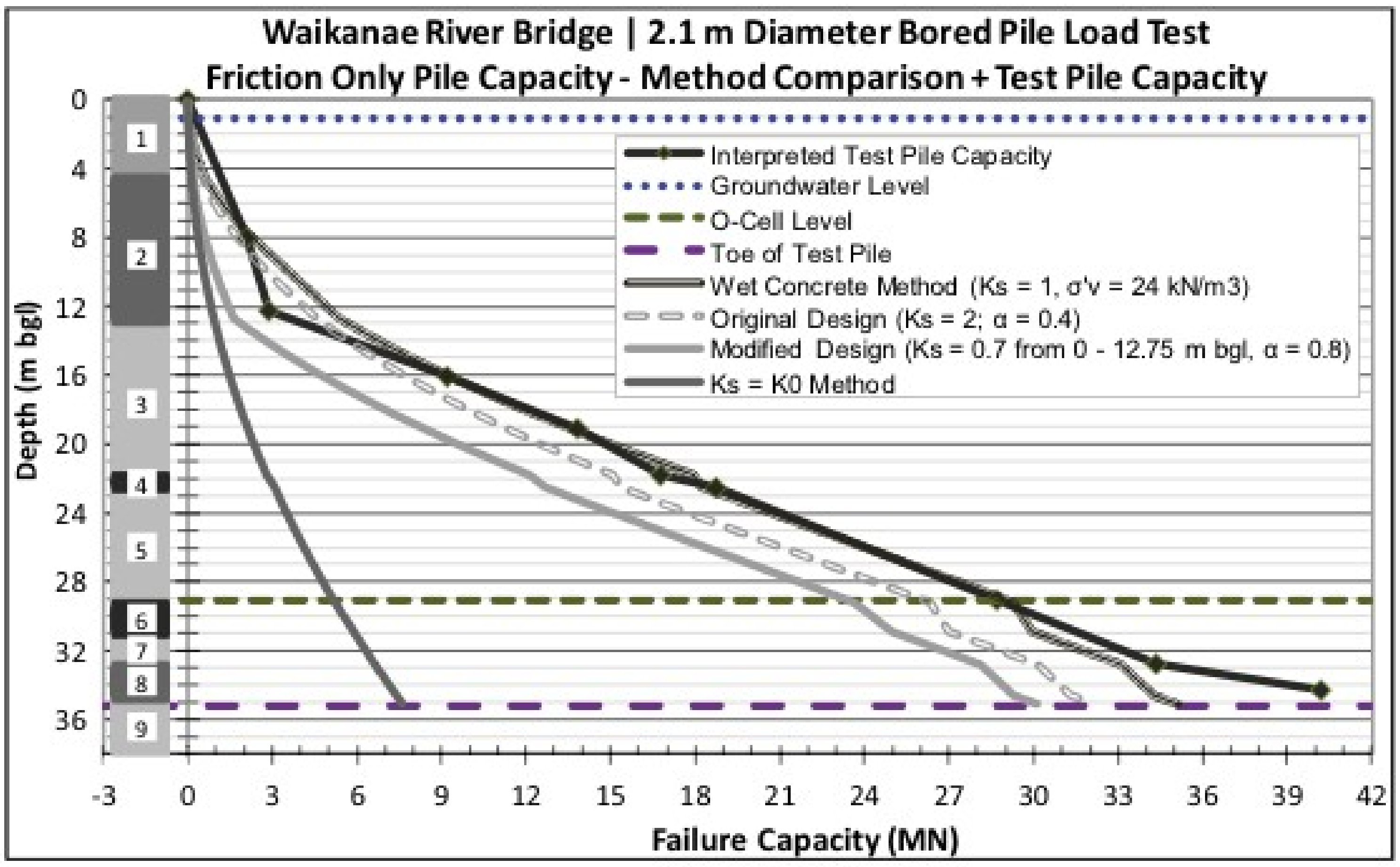

Figure 8. Estimated, modified estimated and measured skin friction capacity (layering as per Table 1).

results of the load test, is presented in Figure 8. The modified design provides a good curve fit with the measured capacity, compared to the original design or wet concrete method, and is more conservative.

It is worth noting that the increased rate of development of skin friction capacity between the lowest strain gauges is due to end effects and the influence of some mobilized end bearing.

8. Conclusions

8.1. Considerations for Design

Overall, the measured capacity of the test pile was greater than that predicted using the selected design approach for the project. Design assumptions are validated, with two modifications identified as part of the load test interpretation:

i) Adoption of a K_s = 0.7 for upper layers of Pleistocene sand and Holocene deposits at Waikanae River Bridge. The Pleistocene sand deposits at depth may continue to use a K_s = 2, and

ii) Use of a higher α = 0.8 value for the Pleistocene silt.

8.2. Considerations for Subsequent Load Tests

The next bored pile load test to be undertaken will be at the site of Te Moana Road Overpass. Greater thicknesses of Pleistocene sand are present at this location compared to at the Waikanae River Bridge site.

In order to better capture the capacity of the Pleistocene sand, a greater number of strain gauges is recommended within this unit and at the Pleistocene sand / gravel interface. Supplementary telltale compression rods with displacement transducers are also proposed. A two-level (push / pull) O-cell load test could also be considered.

Strain gauge reliability requires further consideration in the design of the pile load test at Te Moana Road Overpass.

9. Acknowledgements

The author would like to acknowledge the NZ Transport Agency and M2PP Expressway Alliance for permission to showcase this project and present the findings included in this paper.

The support of my colleagues, in particular Phil Clayton (Associate – Geotechnical Engineering, Beca), has been much appreciated and key in the composition of this conference paper.

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BIRDS HILL REMEDIATION FOLLOWING 2011 TASMAN FLOOD EVENTS – A CASE STUDY.

Rebecca RYDER
MWH New Zealand Ltd., Nelson, New Zealand

ABSTRACT – In 2011 674 mm of rain fell over a 48 hour period in Golden Bay, Tasman, triggering a devastating one in 500 year flood event. As a result several Golden Bay communities were isolated from the rest of the Tasman region and multiple lifeline networks were severed. Slope failure cut off crucial roading networks, flood waters caused extensive damage to waste water and water supply systems as well as communication and power line networks.

This case study focusses on the slope failure at Birds Hill on SH60 near Takaka, Golden Bay. Both lanes of the road were completely lost due to a slip below the carriageway. This failure required rapid reinstatement as SH60 provides the only road access to a number of towns north of Takaka. A temporary access road was cut into the hill-side, however a long term solution was needed.

A reinforced fill slope was selected as the preferred remedial solution. SlopeW modelling software was used to verify the design and assess the potential impact of geological and geometric variations to the design. The opportunity to realign was used to improve the safety of this section of road and provide additional space for cyclists. Remedial works were completed December 2013.

1. Introduction

Following persistent rainfall over 13-15 December 2011 in the Nelson and Tasman region (New Zealand) a number of landslides occurred along with widespread flooding damage.

The region was declared to be in a state of civil emergency on the 14th December (Wopereis, 2014). Teams of geotechnical specialists were deployed to inspect the affected properties and life-line structures around central and rural areas.

The heavy rainfall caused two ground failures on Birds Hill located along SH60 in Golden Bay, Takaka, New Zealand (Wopereis, 2014). One of these two noted failures was a smaller slumping failure and the other a more significant drop-out which removed the full road width, severing the only access route to the settlement of Collingwood, north of Takaka. This paper summarises the rainfall event before focusing on the large Birds Hill drop-out failure and subsequent design and remediation.

2. Rainfall Event 13-15 December 2011

The December 2011 rainfall caused a 1 in 500 year flood event in the Nelson and Tasman regions. Rainfall was not particularly heavy, but it was persistent over a 48 hour period. This weather system was caused by a trough stalled between a low in the Tasman Sea and a high to the east of New Zealand (Stephens, 2012). The recorded rainfall for various Tasman/Nelson regions is recorded in Table 1 below. The total rainfall for the event in Takaka was recorded as 674mm which is around a third of the regions normal total annual rainfall (Stephens, 2012). This excessive volume of rain fell within 48 hours which set a New Zealand record for coastal rainfall (Wopereis, 2014).

Table 1. Nelson/Tasman region rainfall recorded over 48 hour period, December 2011 (Tasman District Council, 2012).

Region/Zone	Rainfall Recorded over 48 hours
Takaka	674mm
Richmond	281mm
Nelson	300mm

3. Civil Defence Emergency Response

The heavy rainfall caused widespread flooding and landslips. Many residential homes were impacted and had to be evacuated. Some rural towns were temporarily cut off by surface flooding and land-slips. After the state of emergency was declared on the 14th December, the Nelson/Tasman Civil Defence response was initiated. As a significant part of this emergency response, geotechnical specialists, both from within the region and around New Zealand, were deployed to carry out inspections of properties impacted during the event.

Teams of geotechnical engineers and engineering geologists travelled from Nelson to the rural settlement of Takaka which had been badly affected with damage to many lifeline access roads. The Birds Hill Slip was quickly identified as a critical site which needed urgent attention.

As a result of the drop out on Birds Hill the only access route to Collingwood was lost (Stephens, 2012). This had economic implications as there are numerous dairy farms in the area requiring road access to export milk on a daily basis. Temporary single lane access was created by cutting into the hill side and side-casting fill material. The steep terrain prevented further widening of the road, so it was not possible to provide more than one lane using this temporary solution (MWH, 2012).

4. Birds Hill Slip – Geotechnical Assessment

Birds Hill is located in Golden Bay approximately 3km north of Takaka on the Takaka-Collingwood Highway / SH60. The road carriage-way was constructed in the 1960's. It traverses the lower hills crossing a series of ridges and gullys. It is likely that the cut material produced from the road excavation through the rocky bluffs was then re-used as fill material in the gullys (Wopereis, 2014). The toe of the hill merges with the flood plain that extends to the sea. An abandoned historic highway reportedly built around the 1870's is still visible at the toe of the slope. The land above the site is covered with regenerating native forest. The floodplain downslope is generally covered in pasture (MWH, 2012).

The underlying geology of the site is mapped by GNS Sciences and illustrated in the published geological map of Nelson Area (Geology of the Nelson Area, Scale 1:250,000, GNS Map 9, 1998). The map indicates that the site is generally underlain by the Onekaka Schist that is locally covered by the Motupipi Coal Measures. The Onekaka Schist exposed in cut batters alongside the highway is moderately to highly weathered. Clayey soils mantle the slopes in the vicinity of the sites. The flood plain at the toe of the hill is comprised of recent fluvial deposits.

The large under-slip at Birds Hill SH60 affected the full road width but did not impact the slope above the road. It was identified that the area which failed was an old gully which had been filled to allow the road to pass between the rocky spurs on either side (Figure 2). The failure appeared to have originated in the fill material, likely to have been placed during the construction of the road. The failure extended vertically to expose the Onekaka schist bedrock at the steepest part of the headscarp. It was also noted during the initial inspections that a services duct which ran along the back of the slip could have contributed to the failure. The services duct was observed after the slip event to be conducting water from further up the road. The duct had ruptured and water was spilling out at the head-scarp of the Birds Hill slip. This theory was not conclusive however, as the pipe may have been ruptured during the slip movement rather than contributing to the failure.

5. Site Investigation / Reconnaissance Data

Investigations carried out at the location of the main slip comprised 12 Dynamic Cone Penetration Tests completed at the top of the slip with six Scala Penetrometer tests and test pits on the face and toe of the slip. This information was used to develop a geological model. A bedrock profile was inferred based on the outcropping rock spurs and mapped geology above the slip. The four main geological units encountered during the site investigation were: Onekaka Schist, Motupipi coal measures siltstone, sandstone and clayey colluvium fill comprised of clay/sandy silt (MWH, 2012).

Groundwater was reportedly located between 1.0m and 3.0m depth at various points down the slip face. It is likely that insufficient drainage was a significant contributing factor to the slope failure (MWH, 2012). This assessment is based on the lack of culverts through the natural low point of the gully as well as the possibility of the services duct leaking water directly into the gully at the head-scarp of the slip.

6. Birds Hill Remedial Solution Concept

As this section of road is a state highway (SH60), the basis of design was controlled by NZ Transport Agency (NZTA) requirements. NZTA framework dictated the design life, seismic loading and factors of safety that had to be met in the analysis.

No lab testing was carried out on any of the geological deposits. Geological design parameters were based on literature and engineering judgment. Due to the high uncertainty associated with the design input information, sensitivity analysis was carried out during the modelling phase. This ensured that there was sufficient robustness in the design and that the model was not disproportionately dependent on a particular input parameter. The analysis explored the sensitivity of the model to geological material parameters, the inferred depth to bedrock and the inferred ground water level down the slope. These parameters were varied within a range of approximately +/-15% and were analysed individually to ensure that results could be interpreted correctly. It was verified that the design was sufficiently robust and would still maintain acceptable stability if the conditions were not as exactly as assumed for modelled purposes. If further site investigations or lab testing of the site materials had been carried out, the design may have been further refined at this stage, possibly reducing the cost of materials and therefore construction. However, the client preferred that a more robust design was adopted to account for the unknown conditions.

The final conceptual design consisted of an engineered fill slope providing sufficient width to reinstate the double lane carriageway, as well as space for cyclists and a berm.

The fill used to construct the slope was to be taken from the rocky bluffs above the road. This method of cut-to-fill was efficient and simple to monitor. This was particularly beneficial for the Birds Hill site which had limited storage space for fill materials. By removing material from the inside slope above the road it also allowed for improved traffic sight distance as the sharp corners were removed from the cut batters.

7. Model Development

The remedial design for Birds Hill was developed using the slope stability analysis software program SlopeW by Geo-Slope International Ltd. Physical geometry of the slope in cross-section was obtained from survey information captured immediately after the main slip occurred and again after single lane access had been reinstated. A simplified geological model was created based on the available site investigation data.

The gully where the slip failure occurred was assumed to be roughly v-shaped, with out-cropping bedrock mapped along two ridgelines. It was known that an engineered fill slope could be founded on bedrock at either edge of the gully. The worst case however was a section through the middle of the gully which had a thick layer of colluvium silt/clay overlying the bedrock. Bedrock was estimated to be much deeper in the middle of the gully as it was not encountered during site investigations. Using the mapped (visible) profile of the bedrock from above the road, along with the spurs of rock on either side of the gully below, the inferred bedrock profile indicated that through the middle of the gully bedrock was potentially up to 10m below the existing ground level. For this depth to bedrock, it was not financially viable to dig out and replace all the colluvium material. A fill slope design solution had to be found that would satisfy the design criteria without needing the slope to be excavated down to bedrock across the width of the slip.

Figure 1 is the design cross-section through the middle proposed fill slope footprint. This was identified as a 'worst case design scenario' through the deepest part of the gully as the colluvium is at an estimated maximum thickness of 10m over the bedrock. The design of the fill slope shown was verified to meet NZTA criteria based on this assumed 'worst case' where the slope toe is founded on colluvium. However the reality is that approximately one half of the engineered fill slope toe is founded on bedrock rather than on colluvium, at the edges of the gully. The sections of wall with a bedrock foundation were found to have a much higher FOS against significant failure than that of the design sections founded on colluvium (Figure 1). By limiting the width of the engineered fill slope which is founded on colluvium rather than bedrock, it is inferred that the stability of the slope has been maximized, while keeping within financial constraints of the project.

Using the SlopeW program, both circular and wedge failures were analysed. Circular failures gave the most critical slip planes, so these were used in the sensitivity analysis. The design process focused on the risk of failure to the road carriage-way. Some minor surficial failures below the road were identified as having a FOS lower than the design threshold of NZTA requirements; however these failures were very shallow, small in extent and did not impact the road carriageway or berm.

Subsoil drains were trenched in below the engineered fill slope face with cut-off drains running laterally across the width of the slip. Installation of this drainage network reduces the chances of future landslips as a result of high rainfall/flooding in the area.

In addition to the fill slope design, new drainage provisions and road layout design were implemented. Both provided improvement to the road, compared to its condition prior to the slip.

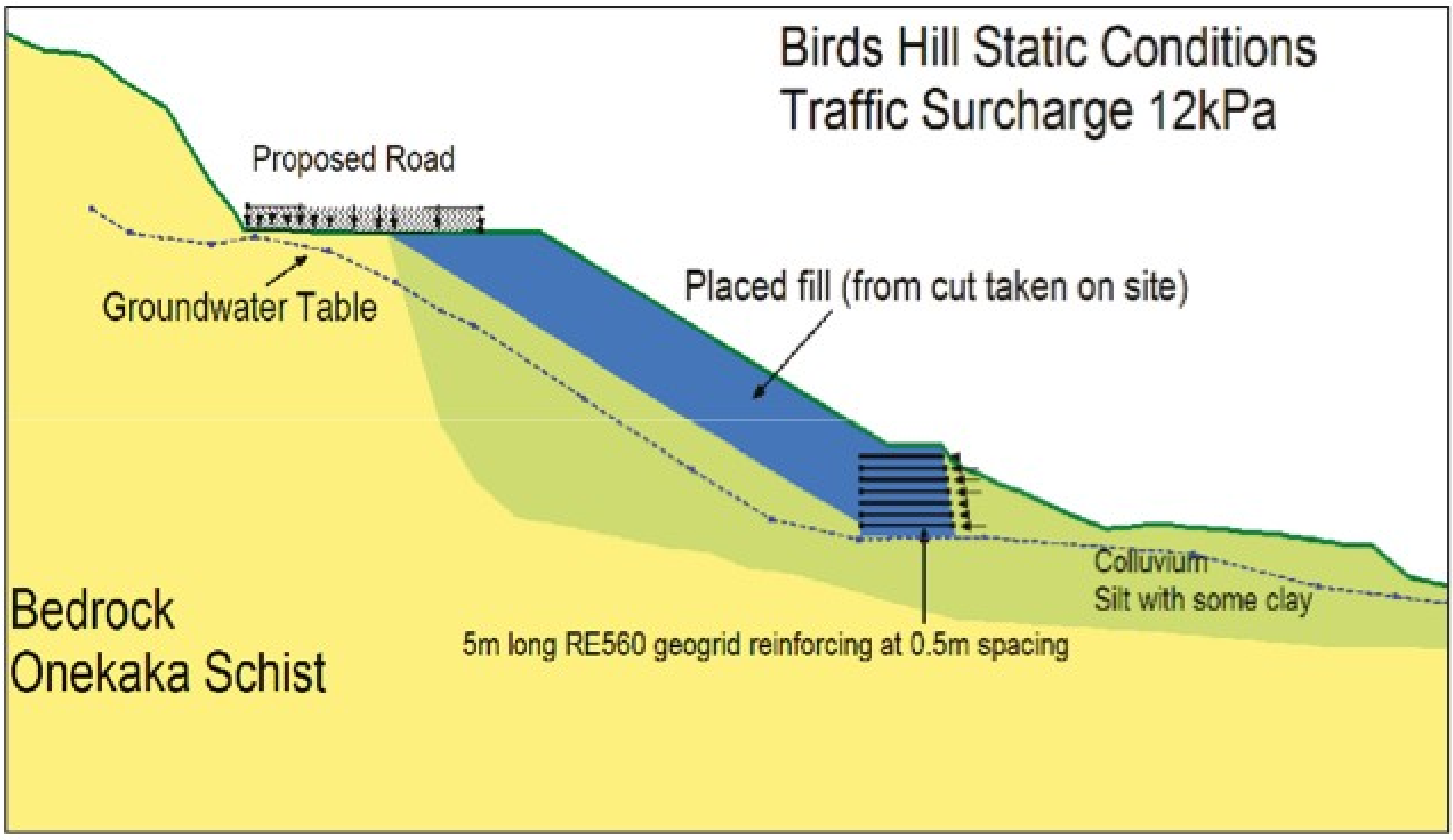

Figure 1. SlopeW model used for the analysis of remedial solutions at Birds Hill.

8. Fill-Slope Construction and On-going Works

Side-cast material had been cut and dumped downslope as part of the emergency works to provide the temporary single lane access through Birds Hill. To prepare the Birds Hill slip site for construction, excavation of the side-cast material was required to provide a stable foundation for the engineered fill slope. To reduce the likelihood of future instability or settlement all of the uncontrolled side-cast fill needed to be removed. The excavation proved to be difficult as it was not clear exactly where the boundary was between the *in-situ* colluvium and end-tipped fill material. Information gathered from survey data was used to verify that excavation was taken deep enough to remove all of the uncontrolled fill.

Originally, a long length of the road cut-batter through Birds Hill was to be trimmed during the construction phase of the engineered fill slope. This would have allowed good quality fill material to be selected from a large volume of cut. A change was made to the construction program that reduced the extent of the cut-batter excavation completed. This meant that fill material was suddenly limited in volume and the engineered slope had to be constructed out of whatever material was provided at that particular stage of the program. The cut material available was much more weathered, and therefore poorer quality, than assumed in the design model. This more weathered, weaker rock had to be used in the fill slope, or the project program would be jeopardized.

Sensitivity analysis provided valuable insight for assessing how a weaker fill source would impact the design. It was concluded that this change of fill strength posed a significant risk to the slope stability. It was agreed that this fill could be used at the toe of the slope, if sufficient reinforcement could be implemented to provide the additional strength required. The design was revised to include a reinforced soil block constructed with layers of geogrid at 0.6m center spacings, extending 5m horizontally at the toe of the slope (Figure 1). The reinforcement eliminated the critical circular failures which were predicted by the SlopeW model for the revised model with a weaker fill material.

To avoid having to include geogrid reinforcement up the full height of the fill slope, less weathered (stronger) material had to be sourced quickly without incurring a significant cost to the project. Alternative fill was located in nearby areas, however to allow the use of this new material additional compaction laboratory testing and plateau testing verification had to be carried out. Some storage and carting costs were also incurred by this change in fill source. The alternative fill sources had a distinctive weathering profile which ranged from highly weathered at the surface, though to moderately weathered deeper into the rock mass. The contractor then had to make an adjustment to the excavation methodology to account for this varying degree of weathering. The most weathered surficial rock was to be removed and dumped. Further cutting of the rock was then made in a way that mixed the highly weathered rock with less weathered rock before it was placed on the fill slope below the road. The SlopeW model was updated and re-analysed with revised fill parameters. By making these changes to the fill source and excavation methodology it was confirmed geogrid reinforcement was only required at the toe of the slope. Fill was monitored closely to ensure that the material was of an acceptable standard throughout the construction of the engineered slope.

To verify the compaction of the placed fill, Nuclear Densometer (NDM) testing was carried out. Compaction curves were completed on the various fill sources and plateau tests were carried out on site using the rock fill. It was found to be difficult to achieve the required compaction on site, despite the use of heavy compaction plant and applying a methodology verified through plateau testing completed on site. The fill was highly variable having been sourced from multiple locations and appeared to be very sensitive to moisture. This was a particular problem as the main construction period ran through wet winter months of 2012. Moisture content of the fill material had to be preserved by covering stockpiles on site and allowing time for fill to dry after a rain event. A significant amount of time was spent on site attempting to adjust moisture contents and repeating NDM tests to ensure the required compaction was achieved.

The re-construction of the road had to be coordinated with service providers. The construction works provided an opportunity for buried services in the road carriageway to be accessed and improved. Timing of such additional works had to be carefully planned so that the road could be safely trafficked while work continued. Land ownership issues also slowed the progress of the construction. Disagreements between NZTA and local land owners meant that excavation of the various fill sources was delayed while land boundary lines were discussed and defined. It was very important that all of these issues were dealt with as quickly as possible especially as local media was following the project and residents were eager to see the two-lane road reinstated.

As part of the Bird's Hill remediation, trimming of the road cut-batters was undertaken on either side of the main slip zone. During this additional work, a historic landslide was uncovered in one of the new cut-batters above the road (Wopereis, 2014). Despite numerous attempts to trim back the batter in this area, continuing instability and failure of the batter occurred. Geotechnical specialist inspection and instruction along with significant earthworks were required to ensure the safety of this area. As a result the cut batter had to be laid back on a very low angle to prevent on-going instability.

9. Conclusions

The value of investing time and funding in site investigations and laboratory testing was strongly emphasized through this project. Many of the design assumptions and construction issues could have been avoided with a greater emphasis on site investigation and testing early in the project life.

Changes to the construction program meant that the fill material was limited in volume and therefore could not be preferentially selected based on quality. To account for the use of poor quality fill geogrid reinforcement was added to the toe of the slope. Additional fill was later sourced from other near-by areas to prevent the need for geogrid reinforcement up the whole height of the engineered slope.

The additional time and expense of finding additional fill sources, carrying out the required laboratory/site testing and completing further design verification as a result of this was a significant cost to the project. However it was a necessary step to ensure that the engineered fill slope was constructed correctly while keeping to the project program and meeting the expectations of the client and road users.

Today Birds Hill is a much improved section of road (Figure 3) with a wide carriage-way shoulder and improved site distance for motorists and cyclists (NZTA, 2013).

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Figure 2. Birds Hill SH60 Slip, December 2011.

Figure 3. Birds Hill SH60 post-remediation construction, December 2013.

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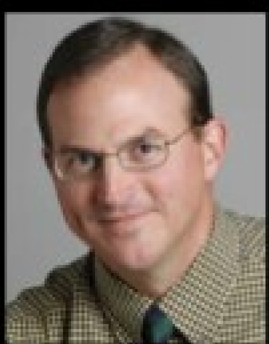

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The following webinars are proposed in the future:

Richard Jardine, UK, *Advanced laboratory testing in research and practice*

Peter Robertson, USA, *In-situ testing using the CPT*

George Gazetas, Greece, *Geotechnical earthquake engineering*

David Muir Wood, UK, *Soil behaviour and modelling*

Hugh St John, UK, *Deep excavations*

Mark J. Cassidy, Western Australia, *Combined Loading of Offshore Foundations (2012 E.H. Davis Memorial Lecture)*

An-Bin Huang, Chinese Taipei, *Landslide risk mitigation with a special emphasis on observation with optical sensors*

Prof. Misko Cubrinovski, Christchurch, NZ, *Impacts of liquefaction in the 2010-2011 Christchurch earthquakes*

Takeshi Katsumi, Japan, *Treatment of debris from tsunami debris after the 2011 gigantic earthquake in Japan*

Lyesse Laloui, Switzerland, *Multiphysical behaviour of soils*

Yoshinori Iwasaki, Japan, *Conservation of cultural heritage*; Erol Tutumluer, USA, *Transportation geotechnics*

Professor Ian Johnston, Melbourne, *Introducing Geotechnical Design to Shallow Geothermal Systems (2012 John Jaeger Lecture)*

Gabriel Auvinet, Mexico, Mexico City, *World capital of Geotechnical Engineering (title to be amended)*

Enio Palmera, Brazil, *Geotextiles in Geotechnical Engineering*

Kaushik Mukherjee, Malaysia, *Offshore pile design and driveability issues in soft clay.*

CORPORATE ASSOCIATES PRESIDENTIAL GROUP (CAPG)

Sukumar Pathmanandavel stated that the CAPG was committed to assisting in developing actions and activities that will enhance the commercial sector of the geotechnical profession and had a key objective of helping the geotechnical engineering profession towards the State of the Art in Geotechnical Practice. The following actions were planned:

- Categorise various areas of geotechnical and geo-environmental practices and develop CAPG view of both State of the Art (SOA) and State of the Practice (SOP);
- Select key areas of geotechnical and geo environmental practices for development by CAPG; and
- Think about the form of a Survey Questionnaire with the assistance of ISSMGE, Corporate Associates, technical and Board level committees and interested universities.

The intention is that, by the time of the ICSMGE in Seoul, the CAPG would have a document describing the State of Geotechnical Practice from around the world.

21ST ICSMGE 2021

On behalf of the Australasian region, the AGS will again be bidding for the opportunity to host the next International Conference on Soil Mechanics and Geotechnical Engineering, which is planned to be held in Sydney in 2021. The AGS is currently developing its conference bid and seeks support from members of the NZGS.

Mark Jaksa

Mark is Head of the School of Civil, Environmental and Mining Engineering at the University of Adelaide. Over the last 25 years Prof Jaksa's research at the University of Adelaide has concentrated on probabilistic methods, geostatistics, artificial intelligence, ground improvement, expansive soils and geo-engineering education. He has published over 125 journal and conference papers on these topics.

NEXT BOARD MEETING

The next ISSMGE Board meeting will be held in Wellington at the 12 ANZ Conference on Monday, Feb. 23 and Tuesday Feb. 24, 2015. At the beginning of the meeting, both the NZGS and the AGS are invited to give a 10-minute presentation on the state of the Societies and any aspects that they'd like to draw to the attention of the Board.

International Society for Rock Mechanics

NEWS RELEASE OF THE COUNCIL MEETING 2014 HELD IN SAPPORO, JAPAN

The ISRM Council meeting took place in Sapporo, Japan, on 13 October 2014. The key points included:

- Membership is now at a record high
- Four nominations were received for the most prestigious award of the ISRM, made once every four years at the ISRM Congress, in recognition of distinguished contributions to the profession of rock mechanics and rock engineering: Giovanni Barla (Italy), Gyula Greschik (Hungary), Richard Goodman (USA) and John Hudson (UK). Professor Hudson was selected by the Council to receive the award. He will deliver the Müller Lecture during the 13th International Congress of the ISRM, in Montréal, Canada
- After a competition between two excellent proposals presented by Brazil and India to host the 14th International Congress of the ISRM, the Council selected the Brazilian proposal, which has the cooperation of the National Groups from Argentina and Paraguay
- The Council selected the symposium Eurock 2016 "Rock Mechanics & Rock Engineering: From Past to the Future" in Cappadocia, Turkey, as the venue of the 2016 ISRM International Symposium, with the Regional Symposia ARMS9 in Bali, Indonesia, and Afrirock in Cape Town, South Africa
- The Council approved a Board proposal to create the ISRM Fund for Education, in order to further the ISRMs mission by enhancing education in Rock Mechanics and Rock Engineering. It will do this by planning, funding, coordinating and conducting educational activities for the benefit of the ISRM community. Priorities will include economically disadvantaged regions and group-wide initiatives. Activities might include training courses, workshops and seminars, continuing education programs for rock mechanics practitioners, development and dissemination of educational materials and endorsing higher education and professional certification programs. The ISRM Education Fund shall be sustained by seeking donations from organizations and individuals
- The Orange Book, produced by the ISRM Commission on Testing Methods, was published by Springer and is launched in Sapporo, during the ARMS8. This book is a collection of ISRM suggested methods for testing or measuring properties of rocks and rock masses both in the laboratory and in situ, as well as for monitoring the performance of rock engineering structures, published since 2007. It complements the Blue Book, with the previous ISRM suggested methods. It is available at reduced prices to ISRM members

ROCHA MEDAL 2015 WINNERS ANNOUNCED

The Rocha Award Committee selected the winner of the Rocha Medal 2015, Andrea Lisjak Bradley (Italy), for the thesis "Investigating the influence of mechanical anisotropy on the fracturing behaviour of brittle clay shales with application to deep geological repositories" and one runner-up, Dr Ahmadreza Hedayati (Iran) for the thesis "Mechanical and Geophysical Characterization of Damage in Rocks". The award will be given during the 13th International Congress, in Montréal.

David Beck

Dr David Beck is General Manager of Beck Engineering. David's research area for his PhD was simulation of instability and induced seismicity in deep level hard rock mining, and he now specialises in rock mechanics, fracture mechanics, induced seismicity, multi-physics simulation and large-scale 3-dimensional discontinuum non-linear numerical modelling.

6TH ISRM ONLINE LECTURE NOW ONLINE

The 6th ISRM Online Lecture was delivered by Prof. Herbert Einstein from MIT and is now online.

DEDICATION CEREMONY for ISRM Past-President John Franklin

A 1.5 tonne piece of black anorthosite is in the University of Waterloo Peter Russell Rock Garden, an award-winning display of large rocks from around Canada. The ceremony took place in May, and Dr Franklin's wife Kersty was in attendance, as well as various friends.

ISRM ROCHA MEDAL 2016 - Nominations to be received by 31 December 2014

The winner will be announced during the 13th ISRM Congress on Rock Mechanics in Montréal, Canada, in May 2015, and will be invited to receive the award and deliver a lecture at the 2016 ISRM International Symposium.

International Association for Engineering Geology and the Environment

THE EXECUTIVE AND Council Meetings of the IAEG were held on 13 and 14 September 2014 in association with the 12th IAEG Congress in Italy.

The budget for 2015 includes allowance of 7,500 Euros for online publication of video lectures and website maintenance. Keynote and invited lectures from the Torino Congress were videoed and can be accessed from www.iaeg.info.

THE BULLETIN

There was considerable discussion on the IAEG Bulletin, including:

- Whether abstracts should be published in up to three languages
- That all memberships should include a soft copy subscription to the Bulletin (the cost would be 13 Euros per year cheaper than the current cost of a hard copy bulletin)
- Establishment of sub-committee to evaluate the price impact of the above changes.

All of the above proposals were approved.

CONFERENCES

The IAEG Congress Torino was a great success, with:

- 872 full registrations; 137 1-day registrations; a total of 1051 registrations
- 1300 accepted papers (from 1800 abstracts)
- 27,000 Euros from the solidarity fund was used to accept 60 applications from low income countries
- 11 participants from NZ
- Full set of the proceedings (8 volumes, 1300 papers) is available on line from Springer for 2,500 Euros.

The 13th IAEG Congress will be held in September 2018 in San Francisco.

The India National Group of the IAEG will host the Asia Regional Conference of the IAEG in conjunction with its Golden Jubilee over the period October 27 – 29 2015 in New Delhi. The Executive and Council meetings of the IAEG will be held in conjunction with this meeting. Abstracts are due in August 2015.

Japan will also host an Asia Regional Conference of IAEG, over the period 26 - 27 September 2015 in Kyoto, titled "Geohazards and Engineering Geology" with themes around humid tectonically active Asian countries. Call for abstracts opens 1 September 2014 (http://2015ars.com/)

IAEG SUB40S

A young practitioners' commission was established during the IAEG Congress in Auckland in 2010, and while some activity followed, it lost impetus. A meeting of interested Sub40's was held at the IAEG Congress in Torino at which it was agreed that a young practitioners/ professionals/ researchers forum should be re-established. The group has the objectives of:

- Facilitating networking, international friendships and opportunities
- Transitioning to taking leadership roles in IAEG
- Pathway for younger people to bring ideas into IAEG.

Please contact New Zealanders Pedro Martins (pedro.martins@beca.com) or Louise Vick (louise.vick@gmail.com) for more information.

50TH ANNIVERSARY BOOK

2014 was the 50th anniversary of the IAEG. To mark this occasion, Sebastian Dupray (a past Secretary General of the IAEG), Ricardo Oliveira and Paul Marinos (past Presidents of the IAEG) and Carlos Delgado (current President of IAEG) put considerable effort into collating and editing a book that celebrates the last 50 years of the association and looks forward to the future of engineering geology. The book is available for purchase via the website.

NEW EXECUTIVE

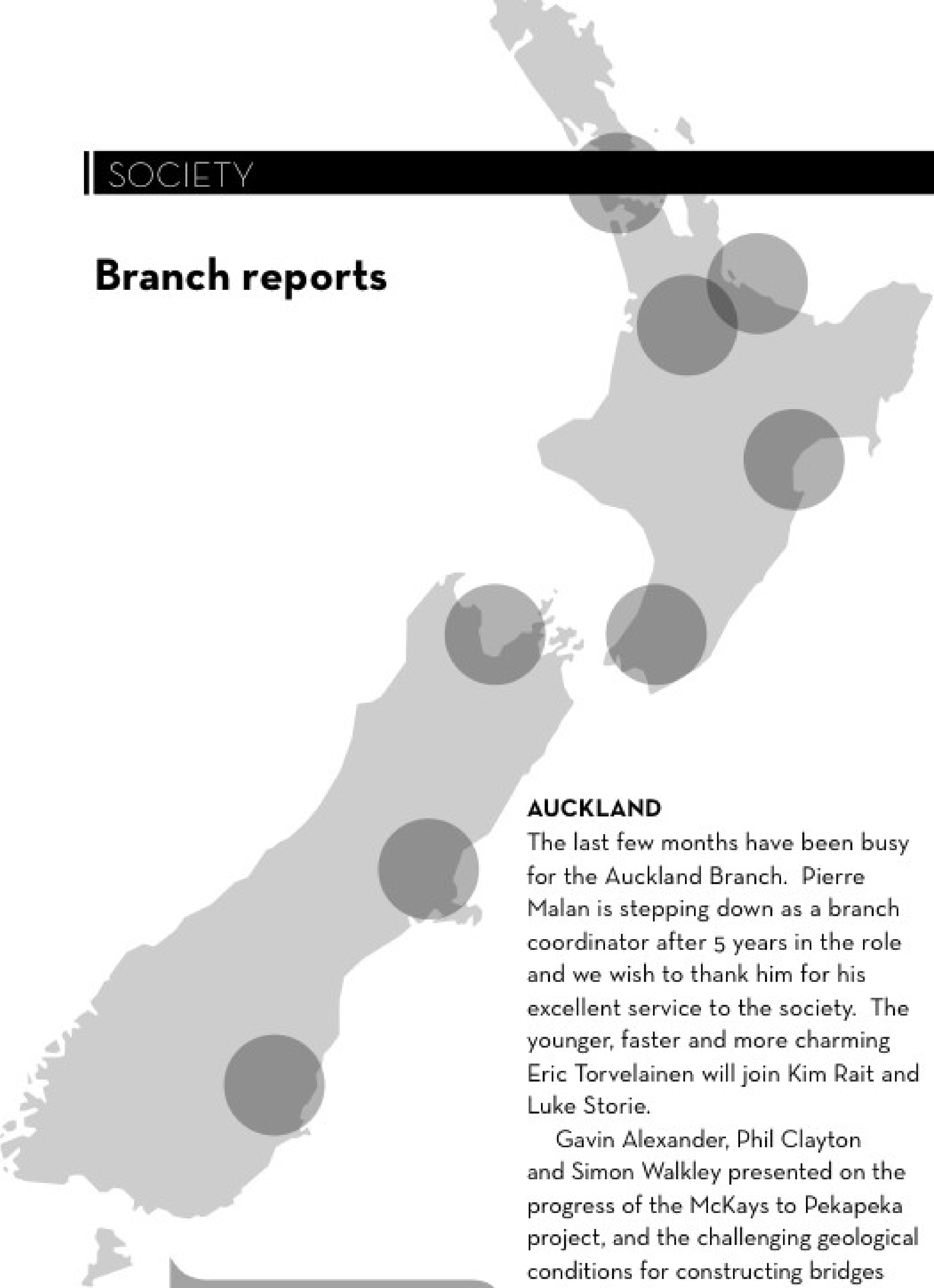
A new executive was elected. The new President is Scott Burns (United States); Scott is a past Vice President of the IAEG representing North America. Scott has enormous energy and has even lived and worked in New Zealand! Faquan Wu will continue in the role of Secretary General. There will be a new treasurer, Jean-Alain Fleurisson.

Mark Eggers will take on the role of VP Australasia. I would like to thank you for the opportunity to represent you on the Executive of the IAEG and wish Mark well in the role. David Burns will take on the role of New Zealand liaison.

Ann Williams

Ann has spent the last 17 years of her 22 year career with Beca. She is a past chair of the NZGS, Vice President representing Australasia on the IAEG and an associate editor of the Quarterly Journal of Engineering Geology and Hydrogeology.

Branch reports



NZGS ANNUAL GENERAL MEETING
★ *March 2015* ★
Members will receive notice of the AGM in February, and details will be available on the NZGS website. **www.nzgs.org**

AUCKLAND

The last few months have been busy for the Auckland Branch. Pierre Malan is stepping down as a branch coordinator after 5 years in the role and we wish to thank him for his excellent service to the society. The younger, faster and more charming Eric Torvelainen will join Kim Rait and Luke Storie.

Gavin Alexander, Phil Clayton and Simon Walkley presented on the progress of the McKays to Pekapeka project, and the challenging geological conditions for constructing bridges and embankments, particularly with peat present to cause headaches. A site visit was arranged to a Watercare detention tank in Kohimarama, supported by Hiways Geotechnical and Fulton Hogan. Warwick Prebble was scheduled to present in September, but this has been delayed to early 2015.

The quarter concluded with the Geomechanics Lecture, presented by John Wood. He looked at displacement based design of highway bridges and walls, and provided an interesting and insightful history and commentary on the topic.

WAIKATO

We have had a few great events recently in the Waikato which have been well supported. The local members were enlightened and entertained by a selection of junior geo-professionals trying out their presentation skills on 21st August. Topics included paleoliquefaction, centrifuge slope stability, tomos, coastal landslides and the Waikato expressway. We thoroughly enjoyed the recent geomechanics lecture in November and now, with the weather warming up (well it should be anyway) and earthworks re-starting we will be arranging to visit the Rangiriri section of the Waikato Expressway. We have a range of talks at various stages of organisation and are looking forward to keeping the branch active. We are also always keen to hear from someone who has an idea for a branch event. Don't be shy.

HAWKES BAY

The Hawkes Bay NZGS Branch has had a few successful events over the past few months, with the younger membership being especially active. A very well-attended co-sponsored NZGS and IPENZ presentation on Contaminated Land was held on 1 May. This was followed up in June by a good round-table discussion on the applicability of applying the MBIE Canterbury Guidelines to liquefaction analyses/remediation in the Hawkes Bay. Geoff Kell, in particular, had some good insight.

The Branch's final event for the year

will be the John Wood's Geomechanics lecture on 19th November. We plan to pick up meetings after the Christmas Holidays, but we donít have a specific event planned as of yet.

WELLINGTON

In the past six months a range of interesting presentations have been made, culminating in the recent 15th Geomechanics Lecture presented by John Wood.

A range of events have been identified by the branch coordinators for the upcoming year, but we would heartily welcome any suggestions for presentations or field trips. If you have any good ideas or would like to volunteer a venue or sponsor an event, please get in touch with one of the branch coordinators.

Upcoming events include:

- Timescales and impacts of post-closure flooding at an open pit mine To be presented by Chris Woodhouse Early 2015
- YGPC presentations and/or ANZ2015 practice
- The 12th Australia New Zealand Conference on Geomechanics (ANZ 2015) 22 to 25 February 2015

Lastly, Doug Mason has recently stepped down from his role of branch coordinator. We would like to thank Doug for all his hard work in organising and running Wellington branch events over the last 3 years, and wish him all the best for the future. Also we'd like to welcome Dolan Hewitt from Opus who has recently joined the team.

NELSON

In June we had a visiting presentation from Kevin Anderson of Aecom. He talked about the seismic design of retaining walls and new documents and guidelines available, together with the MBIE changes. This was a warmup for the following days training course on design implications. This was well received advice and initiated good discussion on local relevance.

Above: *15th Geomechanics Lecture in Napier on 19 November*

The November we hosted the 15th Geomechanics Lecture by John Wood. There was a great turnout and the lecture was of the highest class. Thanks John for your all inputs to the Geotechnical community in NZ.

The local YPG group is very active in Nelson which includes a number of geotechnical specialists. It is good to see them collaborating and holding regular meetings through the year. Rebecca Ryder from MWH was recently awarded a NZGS scholarship to attend and present at the September 10th Young Geotechnical Professionals Conference, in Noosa, Australia. Her presentation was a project profile on emergency repairs work on a landslide that closed a highway in Golden Bay. Rebecca spoke very highly of the experience and the quality of the other presentations. Well done Bec!

Upcoming events:

- Presentation on the Tahunanui Landslide by Marcus Lovell (T&T) early in 2015
- Presentation on Golden Bay Landslide by Rebecca Ryder (MWH early in 2015
- Site Visit to the Richmond High Level Reservoir early in 2015.

CHRISTCHURCH

22 October 2014 Prof. Ellen Rathje (Uni of Texas at Austin): Seismic Stability of Slope, New Developments and Incorporation of Uncertainty.
12 November 2014 Dr John Wood 15th Geomechanics lecture. It was great to meet John and I enjoyed his talk very much. UC Quake Centreís help to organise the venue and setting up last night was appreciated too. Personally, I was disappointed with the numbers that attended (very gratefully for those that did) we
might have had 20 in the audience.

No confirmed upcoming events this side of Christmas.

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

AUCKLAND

Luke Storie

Luke is undertaking a PhD at the University of Auckland on earthquake resistant design of foundations. He is investigating the response of buildings in Christchurch CBD following the earthquakes following on from research undertaken under the supervision of Professor Michael Pender. Previously, with a BE(hons) and BA, Luke was a Geotechnical Engineer at Coffey Geotechnics

luke.storie@gmail.com

Kim Rait

Kim is a Geotechnical Engineer with Beca Ltd. She completed a BSc(Hons) in Mathematics and Statistics at the University of Canterbury before working in accountancy for several years. Kim then returned to UC to complete a PhD in Geotechnical Engineering and has been working at Beca on various small projects over the last year while completing her thesis.

Kim.Rait@beca.com

Eric Torvelainen

Eric is passionate about soil stiffness, SSI and liquefaction. A Canterbury graduate, he works in T&T using numerical methods to solve complex problems, such as wind turbine foundations, bridges, multi-storey and in-ground structures.

ETorvelainen@tonkin.co.nz

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

WAIKATO

Kori Lentfer

Kori is a Engineering Geologist for Coffey Geotechnics. He graduated in 1998 with a BSc(Tech) in Geology, followed by Masters study at Waikato University and an MSc thesis in Engineering Geology from Auckland University in 2007. Kori has worked for consultants based in the UK, Europe and the Middle East.

Kori.Lentfer@coffey.com

Andrew Holland

Andrew is a Director of HD Geotechnical. He studied engineering at the University of Auckland, graduating in 2002.

Andrew's experience includes geotechnical investigation, assessment and design for infrastructure, buildings and development. Andrew is a Chartered Professional Engineer (CPEng).

Andrew@hdc.net.nz

BAY OF PLENTY

Matthew Packard

Matthew is a Senior Geotechnical Engineer with Coffey. He has completed a BSc degree in Earth Sciences at Waikato University and a University of New South Wales Masters of Engineering Science. His main areas of interest are soft ground conditions, liquefaction and settlement analysis, soil-structure interaction and complex retaining structures.

matthew.packard@coffey.com

HAWKE'S BAY

Riley Gerbrandt

Riley, a Geotechnical Engineer with Opus in Napier, immigrated to New Zealand from California with his family in late October 2011. Whilst it took him several months to get up to speed with the local geology, different codes/standards and some innovative Kiwi designs, he has come to thoroughly enjoy the New Zealand engineering consultancy space.

Riley.Gerbrandt@opus.co.nz

WELLINGTON

David Molnar

David is an engineering geologist at Aurecon Wellington. He has 6 years of experience in projects throughout New Zealand, notably NZTA's SH16 Causeway Upgrade and SH2 Muldoon's Corner Improvements, also KiwiRail's North to South Junction which won the 2012 Railway Technical Society of Australia (RTSA) Biennial Railway Project Award.

david.molnar@ aurecongroup.com

Aouyb Riman

Ayoub is a senior geotechnical engineer with more than 10 years of experience gained in several countries in the Middle East, Africa, Australasia and Europe. He has experience in the analysis and design of foundations, soil improvement and treatment, deep excavations, cut and cover tunnels, land reclamation, slope stability, seismic assessments

ARiman@tonkin.co.nz

Dolan Hewitt

Dolan is an engineering geologist with five years of experience. Dolan has worked in Western Australia in mine resource geology and planning. He now works for Opus and has been involved in geotechnical investigations and risk assessments for infrastructure and land development throughout New Zealand.

Dolan.Hewitt@opus.co.nz

NELSON

Grant Maxwell

Grant manages technical development for the MWH geotechnical team across the Asia Pacific region. He grew up in Nelson and has now returned home with a young family. Grant is especially interested in emergency responses and encouraging asset and community resilience to natural disasters. He has 16 years' experience working across NZ, Australia, Pacific nations and the UK.

Grant.J.Maxwell@ nz.mwhglobal.com

CANTERBURY

Edwyn Ladley

Edwyn is a Senior Engineering Geologist with Riley Consultants. He is currently completing a Masters in Geotechnical Engineering through the UNSW, Sydney. Since the Canterbury earthquakes Edwyn has been involved with investigations and rebuilds in the CBD, and advising insurance companies on geotechnical hazards and risks in the Port Hills.

eladley@riley.co.nz

Shamus Wallace

Shamus is an Engineering Geologist with Tonkin & Taylor. Since completing his degree in 2002 he has worked around New Zealand and spent time travelling overseas (a particular passion) and working in London (not so much a passion). Always happy to go off on a tangent when discussions turn to Geology.

swallace@tonkin.co.nz

OTAGO

David Barrell

David is a geologist and geomorphologist at GNS Science in Dunedin. South Island born and bred. Since joining GNS Science, he has specialised in Quaternary geology, landform evolution and landscape processes. David very much enjoys the mix of scientific research and applied geoscience that his work entails.

d.barrell@gns.cri.nz

Amanda Blakey
Amanda has been the NZGS Secretary since 2008. She works from home in Glendowie, Auckland. Amanda is a serial initiative-taker, enthusiastic about working with people and draws on a diverse range of experience. She balances practicality and hard work with a busy family life and lots of fun. She is passionate about travel and adventure.

Please remember to contact the Secretary (Amanda) 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 Amanda. 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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Elected Member	Guy Cassidy	Guy@nzgeoscience.co.nz
Elected Member	Ken Read	Ken.Read@opus.co.nz
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ISRM Australasian Vice President	David Beck	DBeck@beckengineering.com.au
ISRM NZ Representative	Stuart Read	S.Read@gns.cri.nz

EDITORIAL POLICY

***NZ Geomechanics News* is a biannual bulletin issued to members of the NZ Geotechnical Society Inc.**

Readers are encouraged to submit articles for future editions of NZ Geomechanics News. Contributions typically comprise any of the following:

- **technical papers which may, but need not necessarily be, of a standard which would be required by international journals and conferences**
- **technical notes of any length**
- **feedback on papers and articles published in NZ Geomechanics News**
- **news or technical descriptions of geotechnical projects**
- **letters to the NZ Geotechnical Society or the Editor**
- **reports of events and personalities**
- **industry news**
- **opinion pieces**

Please contact the editors (editor@nzgs.org) if you need any advice about the format or suitability of your material.

Articles and papers are not normally refereed, although constructive post-publication feedback is welcomed. Authors and other contributors must be responsible for the integrity of their material and for permission to publish. Letters to the Editor about articles and papers will be forwarded to the author for a right of reply. The editors reserve the right to amend or abridge articles as required.

The statements made or opinions expressed do not necessarily reflect the views of the New Zealand Geotechnical Society Inc.

NEW ZEALAND GEOTECHNICAL SOCIETY INC

NZGS Membership SUBSCRIPTIONS

Annual subscriptions cost $105 per member. First time members will receive a 50% discount for their first year of membership; and student membership is free. Membership application forms can be found on the website http://www.nzgs.org/membership.htm or contact the NZGS Secretary on secretary@nzgs.org for more information.

NEW ZEALAND GEOTECHNICAL SOCIETY INC

The New Zealand Geotechnical Society (NZGS) is the affiliated organization in New Zealand of the International Societies representing practitioners in Soil mechanics, Rock mechanics and Engineering geology. NZGS is also affiliated to the Institution of Professional Engineers NZ as one of its collaborating technical societies.

The aims of the Society are:

a) To advance the education and application of soil mechanics, rock mechanics and engineering geology among engineers and scientists.
b) To advance the practice and application of these disciplines in engineering.
c) To implement the statutes of the respective international societies in so far as they are applicable in New Zealand.
d) To ensure that the learning achieved through the above objectives is passed on to the public as is appropriate.

All society correspondence should be addressed to the Management Secretary (email: secretary@nzgs.org).

The postal address is
NZ Geotechnical Society Inc,
P O Box 12 241,
WELLINGTON 6144.

Letters or articles for NZ Geomechanics News should be sent to editor@nzgs.org.

MEMBERSHIP
Engineers, scientists, technicians, contractors, students and others who are interested in the practice and application of soil mechanics, rock mechanics and engineering geology are encouraged to join.

Full details of how to join are provided on the NZGS website http://www.nzgs.org/about/

National and International Events

2015

22-25 FEBRUARY, 2015
Wellington, New Zealand
ANZ 2015 Changing the Face Of The Earth
http://www.anz2015.com/

10-12 APRIL, 2015
Rotorua, New Zealand
New Zealand Society for Earthquake Engineering 2015 Conference
http://confer.co.nz/nzsee2015/

27-30 APRIL, 2015
Hammamet, Tunisia
16th ARCSMGE
http://www.cramsg2015.org/?lang=en

10-13 MAY, 2015
Montréal, Québec, Canada
The 13th International ISRM Congress 2015
http://www.isrm2015.com/

10-12 JUNE, 2015
Oslo, Norway
International Symposium on Frontiers in Offshore Geotechnics 2015
http://www.isfog2015.no/

15-17 JUNE, 2015
Rome, Italy
The 3th International Conference on the Flat Dilatometer
http://www.dmt15.com/

13-17 SEPTEMBER, 2015
Edinburgh, Scotland
XVI European Conference on Soil Mechanics and Geotechnical Engineering
www.xvi-ecsmge-2015.org.uk

24-25 SEPTEMBER, 2015
Island of Ischia, Italy.
4th Workshop on Volcanic Rocks and Soils. An ISRM Specialised Conference.
http://www.wvrs-ischia2015.it/

7-10 OCTOBER, 2015
Salzburg, Austria
EUROCK 2015 - 64th Geomechanics Colloquy, An ISRM Regional Symposium.
http://www.eurock2015.com/en/

13-16 OCTOBER, 2015
Rotterdam, The Netherlands
5th International Symposium for Geotechnical Safety and Risk ISGSR 2015
http://www.isgsr2015.org/

2-4 NOVEMBER, 2015
Christchurch, New Zealand
6th International Conference on Earthquake Geotechnical Engineering (6ICEGE)
http://www.6icege.com/

3-5 NOVEMBER, 2015
Brussels (Belgium)
2nd International Workshop on Geomechanics and Energy
http://www.eage.org/event/index.php?eventid=1250

9-13 NOVEMBER, 2015
Fukuoka, Japan
http://www.15arc.org/index.html

15-18 NOVEMBER, 2015
Buenos Aires, Argentina
Sixth International Symposium on Deformation Characteristics of Soils
http://conferencesba2015.com.ar/

26-28 NOVEMBER, 2015
Tirana, Albania
Geo-Environment and Construction European Conference

7-11 DECEMBER, 2015
New Delhi (NCR), INDIA
6th International Conference Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics

2016

1 MAY, 2016
Cape Town, South Africa.
African Rock Engineering Symposium. An ISRM Regional Symposium.

25 MAY 2016
Xi'an, China
GEOSAFE: 1st International Symposium on Reducing Risks in Site Investigation, Modelling and Construction for Rock Engineering
xtfeng@whrsm.ac.cn

1-6 AUGUST, 2016
New Delhi (NCR), INDIA
6th International Conference Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics
http://6icragee.com/6IC/

29-31 AUGUST 2016
Cappadocia, Turkey
EUROCK 2016, An ISRM Regional Symposium
http://eurock2016.org/

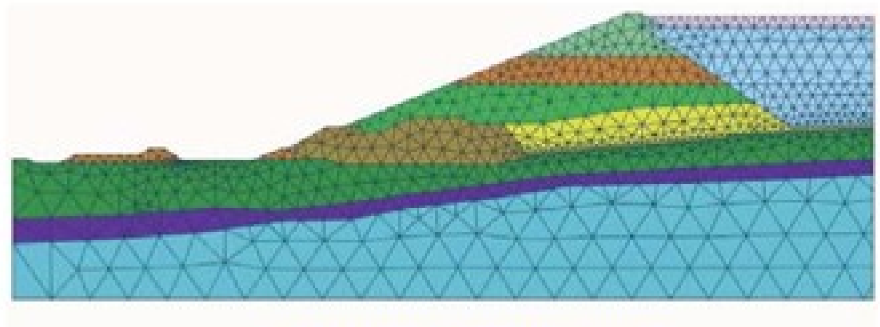
29 -31 AUGUST 2016
Ürgüp-NevŞehir, Turkey
EUROCK 2016 – ISRM European Regional Symposium
http://eurock2016.org/
1 October 2016
Bali, Indonesia
ARMS 9 – the 9th Asian Rock Mechanics Symposium, an ISRM Regional Symposium
rkw@mining.itb.ac.id

2017

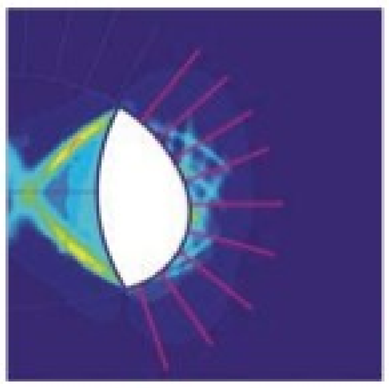
LOCATION: SEOUL, KOREA
19th ICSMGE-Seoul 2017 - Unearth the Future Connect Beyond

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

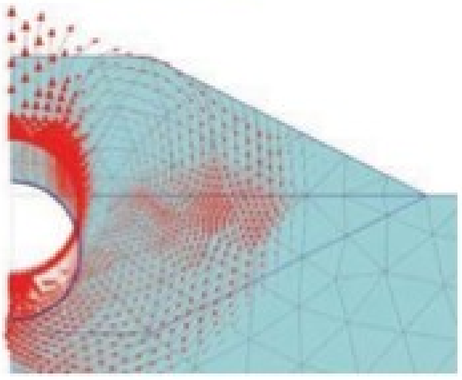
NZGS ANNUAL GENERAL MEETING
★ *March 2015* ★
Members will receive notice of the AGM in February, and details will be available on the NZGS website. **www.nzgs.org**



automatic mesh refinement



tunnel reinforcements



upward motion pipeline