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NZ GEOMECHANICS NEWS

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SPECIAL FEATURE AWARDS

GEOMECHANICS LECTURE
GEOMECHANICS AWARD
STUDENT POSTERS

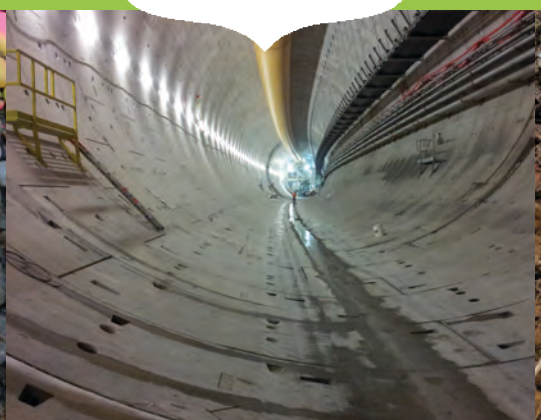
PROBLEMS WITH CPT INVESTIGATIONS

POST DISASTER GEOTECHNICAL RESPONSE IN HILLY TERRAIN

DETERMINATION OF SITE PERIOD FOR NZS1170.5:2004

HORIZONTAL TO VERTICAL SPECTRAL RATIO: APPLICATIONS AND LIMITATIONS

NZGS PHOTO COMPETITION WINNERS



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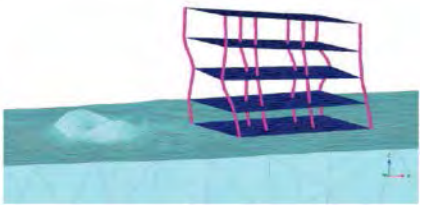

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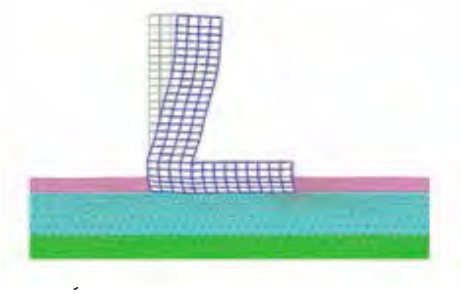

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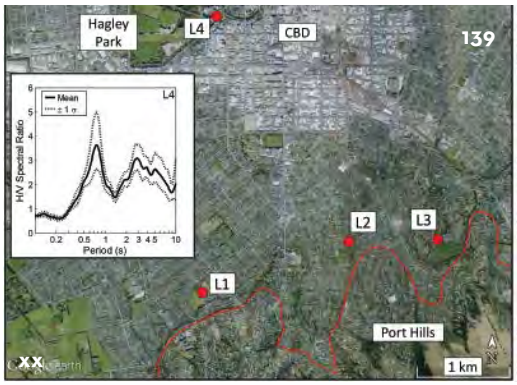


NZ Geotechnical Society
2015PHOTO COMPETITION
Show us your award winning style and tell us which award you think you should get – e.g. Best in Show, Darwin Award, Academy Award, Bravery Award details PAGE 146

COVER IMAGE: . Main image by Hamish Cattell, an NZGS member from Christchurch, 'Penstocks to Tekapo B Hydro Station'. Andreas Giannakogiorgos (Driving Art) Hamish Maclean (Waterview tunnel) and Martin Wilson (Tunnel 2 Johnsonville line).

Charlie is the Chief Geotechnical Engineer at MWH in Christchurch. Educated as a civil engineer in Dublin and an Engineering Geologist at Imperial College in London, he has worked on dam and tunnel projects in Africa, oil and gas projects in the North Sea, hydroelectric power stations in Pakistan and the UK. He moved to New Zealand in 2003 to work on Project Aqua, and spent seven years working with URS in their Christchurch office before moving to MWH in 2011.

Charlie Price
Chair, Management Committee

THIS IS THE 89th edition of Geomechanics News, and I hope every reader will gain some gem of wisdom from it. In recent years it has gained in stature with a number of excellent technical papers, and this version is notable in this respect with the publication of two award winning papers. The first of these is the Geomechanics Lecture, this year presented by John Wood, which is primarily a presentation rather than a 'paper', and has already been presented around the country by John, culminating in ANZ2015. The second is the reprinting of the Geomechanics Award paper, an award given by NZGS every three years to the author(s) of the paper adjudged to be the 'best' published paper by a member during the previous three years. Both of these are outstanding contributions, and I commend them to all our members.

In March the Society Office holders had their two yearly shuffle of positions on the Management Committee, with Gavin Alexander stepping back from the Chair and passing the mantle to me, and Tony Fairclough moving into the Vice-Chair and Treasurer role. Both of these roles require a significant amount of dedicated input, over a combined period of four years, and the increasing role of the Society in facilitating the publication of guidelines and involvement in industry leadership bodies has resulted in Gavin further extending that commitment during his period as Chair. He has done a huge amount of work in 'keeping the ball rolling' during that four year period, and the society owes him a debt of gratitude for that. He will no doubt relish a reduction in time commitment as he steps out of the Chair role, however, his role on the Management Committee has not altogether expired, as he has accepted some ongoing challenges in the form of NZGS representation on National Industry bodies.

CONFERENCES

We seem to have be going through a fairly intensive period of conferences in recent years, with the ANZ conference in Melbourne in 2012, followed by our Queenstown conference in 2013, the ANZ in Wellington, just a few months ago, and the fast approaching 6th International Conference on Earthquake Geotechnical Engineering (6ICEGE) to be held in Christchurch in November this year, for which we are one of the supporting 'Partners'. This will be a major international event, with many of the global leaders in earthquake geotechnical engineering present, and I would suggest that you add this to your calendar now.

The ANZ conference, held in Wellington in February, was as high a quality and every bit as polished as we have come to expect of such events of ours in recent years. Guy Cassidy is to be congratulated on his organisation of the event, and doubly so as the conference has again made a substantial surplus which is returned to the benefit of the society. We are now beginning to think about the next Symposium, which will be NZGS's 20th, to be held in 2017, and have appointed Pierre Malan to be the convenor.

INTERNATIONAL SOCIETIES

The ISSMGE board were our guests at the ANZ2015 conference, and by all accounts were suitably impressed by arrangements made for them, and by that small part of the conference that they were able to attend, as most of their time was taken up in Board Meetings.

We were pleased to hear that Chris Massey has been invited to give a keynote lecture to the 13th International ISRM Congress on Rock Mechanics, which will have taken place in Montreal by the time you read this. Chris's lecture deals with the performance of rock slopes during the Canterbury Earthquake sequence, and is supported by the Society.

We are now in the process of nominating some further members to Technical Committees of the ISSMGE. This will be concluded in the next couple of months.

GUIDELINES

Much work has continued to be put in to the development of this series

of guidelines in recent months, and publication of some of the modules is now imminent, with at least two, plus an update to Module 1, expected to happen by the end of this year. The series of Guidelines is now also supported by MBIE.

Nick Harwood has been active in authoring the geotechnical aspects of an update to the NZSEE 'Red Book' guideline, 'Assessment and Improvement of the Structural Performance of Buildings in Earthquakes (AISPBE)', and this NZSEE document (supported by MBIE) is also expected to be published in 2015.

OVERSEAS VISITORS

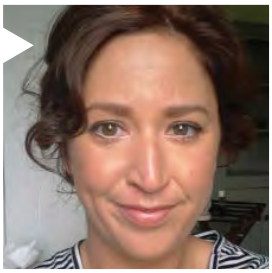
We have had two eminent lecturers from the UK provide excellent presentations this year already. Prof Jim Griffiths, Plymouth University, presented his 14th Glossop lecture earlier in the year, and in May we were privileged to have Professor Guy Houlsby of Oxford University give us fresh renditions of his 54th Rankine lecture in Auckland, Wellington and Christchurch. Both of these presentations were extremely well received by members. We have sponsored similar Rankine lecture tours jointly with the ICE and AGS on regular occasions in recent years, and hope to continue this fruitful liaison in the future.

NZGS will be at the 6ICEGE in Christchurch in November.

Please come and say hello, meet some of the exhibitors and of course, have a great time at the Conference. Don't miss this big international event in beautiful Canterbury!

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

THIS SPECIAL AWARDS edition of NZ Geomechanics News is a celebration of our society and our members. We will share the technical highlights from the ANZ 2015 conference, the NZGS Geomechanics Award, the NZGS Geomechanics Lecture, and the NZGS Student Awards.

The quality of the award winners and their papers reproduced are testament to the strength of our membership. This bumper-sized edition includes a number of longer articles than usual to circulate as much of this high-quality work as possible – and our digital edition is longer still with great technical pieces we couldn't fit into our printed magazine. Check out these extended versions online, on iPad and on Android tablets for fantastic extra content.

Also presented on the front cover of this edition are the winners of our 2014 photo competition. The overall winner Hamish Cattell, an NZGS member from Christchurch, who submitted the stunning image 'Penstocks to Tekapo B Hydro Station'. The quality of the other entries meant we needed three runners-up. These, presented left to right on the cover, were Andreas Giannakogiorgos (Driving Art) Hamish Maclean (Waterview tunnel) and Martin Wilson (Tunnel 2 Johnsonville line). We have prominently displayed these photos on our new NZGS banners, which were first used in ANZ 2015 Conference.

The technical strength of our society is set to continue with the 6th International Conference on Earthquake Geotechnical Engineering (6ICEGE) being held in Christchurch in November this year and plans already in place for the NZGS 20th Symposium to be held in 2017. In addition, the Australian Geomechanics Society has announced that it will again bid to host the International Conference on Soil Mechanics and Geotechnical Engineering. Having narrowly missed out on the 2017 conference to South Korea they now aim to bring the quadrennial conference to Sydney in 2021 with the support of the NZGS.

With such a strong line-up of local and international conferences we are well set to continue producing the high standard of papers presented at the ANZ conference.

All this work takes a huge amount of voluntary effort from the authors of papers to conference organisers, committee members and local branch co-ordinators. There are far too many to mention them all, but special mention should be made of Guy Cassidy and his committee who worked tirelessly to organise a truly wonderful ANZ 2015 conference in February, and to Misko Cubrinovski and his committee who are already flat out organising 6ICEGE. In addition to this more glamorous work there is a huge effort going on at the moment to update or create guidelines for geotechnical engineering practitioners with numerous people involved under the leadership of Kevin Anderson and Tony Fairclough. I cannot overstate how much effort goes in to organising such a large programme of work, and Gavin Alexander, our past Chair, must now be somewhat relieved to be able to step back a little. If you meet any of these people, please take a moment to thank them for all the work they have put in to making our society a smooth running and technically robust machine.

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.

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News – In Brief

More on geotechnical data and BIM in NEXT ISSUE

MACCAFERRI NAME CHANGE

Maccaferri NZ Ltd, founded in 1988, has announced that it will be changing its name to Geofabrics New Zealand Ltd after Geofabrics Australasia Pty Ltd has increased its shareholding from 85% to 100% in February.

Geofabrics announced that their people, products, contacts and support services remain unchanged.

NZGS and MBIE move forwards with Geotechnical Guidelines

MBIE in partnership with the New Zealand Geotechnical Society is pleased to advise that the following guidelines are currently under development. The guidelines, the structure of which is outlined below, will substantially add to the material available to support the engineers, developers, and others in the New Zealand building industry.

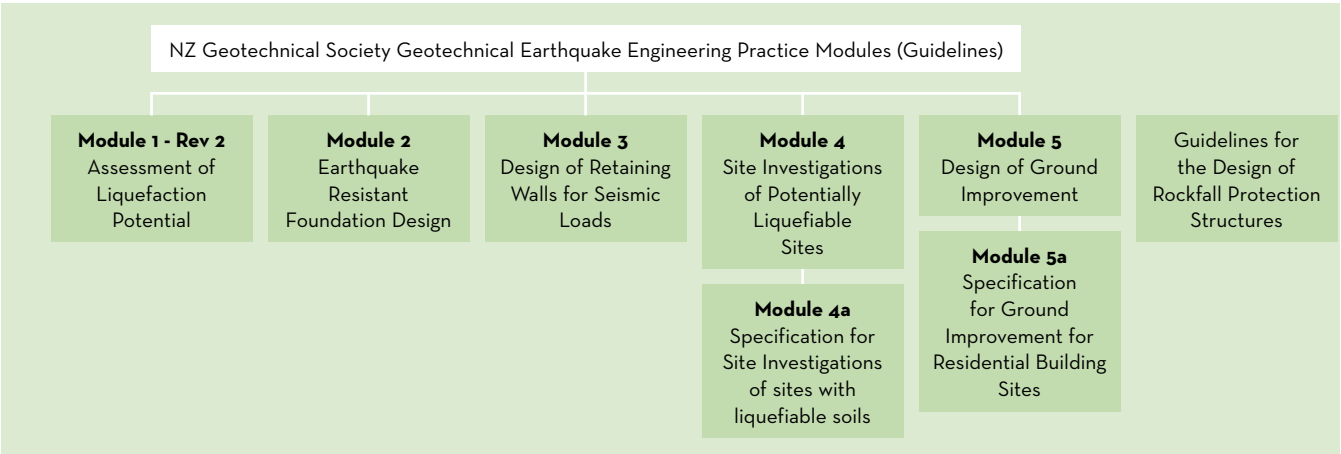
Module 1 is an update to the existing to the existing Module 1 guideline published in 2010 and incorporates recent research and lessons from the Canterbury earthquake sequence. Modules 2, 4, 4a, 5 and 5a are in direct response to the Canterbury Earthquakes Royal Commission recommendations. The rockfall protection structure design guideline is being developed as a national guidance document and builds on the guideline prepared by Christchurch City Council in 2012 in response to the rockfall hazard in the Port Hills caused by the Canterbury earthquake sequence. With the exception of Module 5 the balance of the guidance documents are expected to be issued progressively throughout 2015. Module 5 is likely to be issued in 2016.

AUCKLAND GEOTECHNICAL DATABASE

In April Watercare Services Limited introduced the Auckland Geotechnical Database at the Auckland Infrastructure and Procurement Technology Workstream. Based on the Canterbury Geotechnical Database, the Auckland Geotechnical Database has been established to facilitate the sharing of geotechnical data across Council, Utilities, Transport Agencies, consultants and contractors.

Utilising a common "cloud" platform, users are able to access geotechnical data and in return contribute their geotechnical data for the greater benefit of all organisations. Long term the Auckland Geotechnical Database will be amalgamated into a National Geotechnical Database.

Access to the Auckland Geotechnical Database is via a login at https://agd.projectorbit.com.



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KNOW YOUR GEOLOGIST
Name the engineering geo team on Clyde Power Project, in summer of 1991-92
Answers next issue

HARKER UNDERGROUND

One of New Zealand's largest tunnelling contractors and a part of the Hawkins Group, is going through a period of transformation with two new senior appointments and a re-brand underway.

In February Harker announced the appointment of Rory Bishop as General Manager and Matt Mules as National Operations Manager. Rory's specialist knowledge and extensive involvement in large-scale tunnelling projects led to significant roles on critical, high profile projects in Europe – in particular, managing the extension of the Rail and Metro Lines to the new Terminal 5 at Heathrow Airport, while Matt brings over ten years of experience from a number of complex, high-profile, global projects.

Tim Lancaster has moved into a new role as Commercial Manager and Mike Harker will continue to lead the Bid Team.

"It's an exciting time for Harker and for our industry," says Rory. "We're known locally for our trenchless construction technology and, where the opportunities arise, plan to take this experience and capability offshore."

Harker

ENGEO Incorporated, a firm of more than 200 engineers, geologists, environmental scientists, hydrologists and field representatives, has announced the promotion of ten leaders to higher levels of service. ENGEO president Uri Eliahu reported, "With expected growth comes the need for additional strong leaders, and ENGEO announces five new Principals and five new Associates. These individuals exemplify ENGEO's values, integrity, client-service ethic and commitment to technical excellence. They combine those traits with great enthusiasm to seize opportunities and make great things happen." New Zealanders Greg Martin and Guy Cassidy have been promoted to Principal, while Joseph Gray has been made an Associate.

ENGEO
Expect Excellence

HoleBASE SI UPDATE

Keynetix have announced that the latest update to their flagship HoleBASE SI software includes 'scheduling tools' that allow engineers to electronically schedule geotechnical or environmental testing intended to transform the way consultants and laboratories communicate.

"The new features in HoleBASE SI are going to change the way our clients and their laboratories work together" said Roger Chandler, Managing Director of Keynetix. "For years laboratory staff have been transferring test results using AGS data format but these new additions make it much easier for the engineer to schedule their testing within their data management system, significantly reducing transcription errors by laboratory data entry staff."

Laboratories are able to send clients electronic schedule instructions, including the type of testing they offer and any additional requirements required for each test. This reduces the number of follow up calls the laboratory needs to complete to finalise the testing schedule.

Although AGS data is now commonly used as the data transfer format of choice by engineers and laboratories, there are often problems with all 4 AGS sample references being maintained and returned to the engineer. The addition of these features will significantly reduce occurrences of this problem.

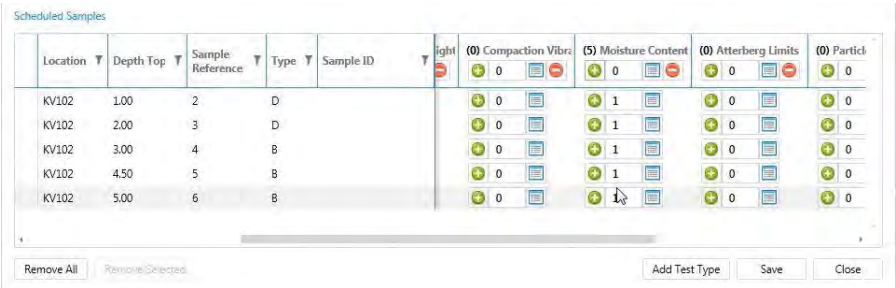

Fig: Laboratory scheduling tool in HoleBASE

★ AMANDA BLAKEY RESIGNS

It is with regret that we announce the imminent departure of Amanda Blakey from the NZGS management team. She will be returning to her roots as a planner, working with her husband Richard as an independent planning consultant. Amanda became NZGS secretary in 2008 and has since been instrumental in keeping the Society running efficiently.

She has worked tirelessly behind the scenes acting as the first point of contact for members, maintaining international links with ISSMGE, ISRM and IAEG, supporting branch events and presentations, organising courses – basically anything too challenging for the rest of the committee. Perhaps her most important role has been providing continuity in a constantly changing committee. Amanda has patiently and kindly helped train the new committee members, taking time to show each one how to make the most of their time on the committee and avoid previous mistakes.

She is also instrumental in the production of this Bulletin. Her enthusiasm, organisation and kindness will be greatly missed by us all.

There has been phenomenal growth in Society membership, events and conferences, and even an International Award (ISSMGE 'Outstanding Member Society') during Amanda's time. Wthout her dedication and diligence the society would not have achieved what it has done in that period. We all wish her the best for her future.

A recent advertisement on the NZGS website for a job vacancy attracted over 50 applications.
The advertisers (employers) were impressed and a bit overwhelmed (understandably). If you have a position to advertise in your business and would like the pick of the crop – don't forget that for a very reasonable fee ($75/month plus GST) – you can advertise on the NZGS Website. Please contact secretary@nzgs.org

CALL FOR CONTENT:

DECEMBER 2015 GEOMECHANICS NEWS SPECIAL FEATURE ON GEOTECHNICS AND BIM.

Please contact the editors if you have a relevant technical or news article that could be included. Ideally these would include technical articles on the integration of geotechnical data into BIM or geotechnical data management, and relevant project case-studies.

We are also keen to receive general short project updates on any interesting geotechnical project (ideally around 100-500 words with photos) and other technical and industry news.

Academic News

UNIVERSITY OF CANTERBURY

NEW SENIOR LECTURER IN ENGINEERING GEOLOGY

Dr. Clark Fenton has recently joined the team in Engineering Geology from the Department of Geological Sciences from Imperial College London, where he was programme leader for the Masters in Engineering Geology for the past 10 years. Clark is a Registered Geologist in California, a Fellow of the Geological Society of London and also brings with him over 10 years of industry experience, having worked for URS and Woodward Clyde in California prior to his appointment at Imperial. Clark studied at the University of Glasgow as a structural geologist and brings his expertise in that field to bear into his research and technical work. His key research interests include offshore foundations, fault rupture hazards, palaeoseismology and seismic source characterization, the influence of geological history on the mechanical behaviour of mudrocks, and the engineering geology of frozen ground. We are looking forward to his contribution to the research and technical expertise in New Zealand in these areas as well as to his contributions to teaching into the Professional Masters of Engineering Geology.

PROFESSIONAL MASTERS OF ENGINEERING GEOLOGY

This year is our first cohort in the new 12-month Professional Masters of Engineering Geology (PMEG). This degree is a professional postgraduate course of study delivered with an 8-course programme over approximately 8 months, followed by a 4-month dissertation. It includes a combination of lectures, tutorials, labs and field trips in different proportions for different courses. Students in the PMEG form two broad groups: students with a BSc who wish to obtain a postgraduate degree and persons from industry completing individual courses as part of their Continuing Professional Development (CPD) requirements for Professional Engineering Geologists or Chartered Professional Engineers. The students are currently identifying their dissertation project topics and we encourage collaboration with industry partners. In the past two years we have had successful collaboration with Aurecon, Davis Ogilvie, Department of Conservation, Environment Canterbury, Fugro, Geotech Ltd., Golder Associates, Pells Sullivan Meynink, and Tonkin and Taylor. We hope to see continued and increased interest in collaborations with these student projects.

RESEARCH SUMMARY

Evaluation and field calibration of a 3D numerical rockfall model for hazard assessment, **Louise M. Vick** *(PhD, submitted April 2015)*

Data from the Port Hills has been used to calibrate and test a state-of-the-art three-dimensional rockfall model. The model calibration from earthquake data has been tested against anthropogenic rockfalls to establish the influence of rotational and translational velocity in a rockfall, as well as changes in soil moisture content. The thesis establishes the importance of rigid-body modelling, as well as considering different soil parameters in hazard analysis.

The influence of alteration and lithology on rock properties and its relationship to drilling optimisation, **Latasha Wyering Templeton** (PhD, completed April 2015)

The properties of the hydrothermally altered rocks from the Ngatamariki, Rotokawa and Kawerau geothermal fields were measured through laboratory testing. Differing mechanical behaviour due to changes in physical properties through secondary mineralisation is evident as the samples in our study from the shallow, low temperature regions of the geothermal fields have lower UCS when compared to samples from deep, high temperature regions. These results provide evidence that mechanical rock properties are changed as a rock undergoes hydrothermal alteration. The Alteration Strength Index was developed to estimate rock strengths based on different geological characteristics to assist with selecting drill bits and optimise drilling operations.

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1-4 NOVEMBER 2015

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6ICEGE will provide not only the usual conference opportunities for exchange of ideas, presentation of novel findings and project achievements, but will also highlight our first-hand experience of the devastating Christchurch earthquakes and efforts towards the rebuilding and recovery of the city.

www.6icege.com

REGISTER NOW FOR THE 6ICEGE

The world's premier Earthquake Geotechnical Engineering Conference.

EARLYBIRD REGISTRATION CLOSES ON 3 AUGUST 2015

FOLLOWING ABSTRACT SUBMISSION, nearly 400 papers have been 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.

To accommodate the significant interest in the conference, we have decided to extend the duration of the conference to three and a half days, starting the 6ICEGE at noon on 1st November (Sunday) and closing it in the evening of 4th November 2015 (Wednesday). A provisional conference programme is now available on the conference website.

Three prestigious named lectures have been confirmed as part of the conference program:

- 5th Ishihara Lecture – Presented by Prof. Kokusho (Chou University, Japan)
- 2nd Schofield Lecture – Presented by Prof. Kutter (UC Davis, USA)
- 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).

A long list of the most prominent local and international names in the area of earthquake geotechnical engineering will deliver lectures at 6ICEGE. The program includes nine keynote lectures, 25 theme lectures, 2 discussion sessions, five special sessions, and 10 mini-symposia in addition to oral and poster presentation sessions.

Registration and accommodation information is available on the conference website, along with details of the technical tours before and after the conference. Please visit **www.6icege.com** for more information.

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Raito

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Letters to the Editors

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.

IT WAS VERY pleasing to read the articles on ethics and professionalism in the December 2014 issue of Geomechanics News. Ethical behaviour, or even accepting that such a thing exists, appears to be steadily declining in today's world. The business community, especially that involved in financial affairs, and our politicians, are leaders this decline. Even the very concept of an action being morally right or wrong is disappearing. People get caught doing what is clearly wrong and they excuse themselves by saying "everyone makes a mistake from time to time" or "we all make bad decisions (or choices) sometimes". What such people are actually doing is something they know is morally wrong and hope they will not be caught.

Ethics and morality are closely linked, but it is useful to distinguish between them in the following way: Morality involves clear principles or precepts of what is right and what is wrong, while ethics involves the application of these precepts to practical situations. Moral teachers, and I suspect (or hope) most readers, will agree that the following are morally wrong:

- Blatantly telling lies, especially with the clear intent to deceive someone.
- Blackmail
- Bribery, especially of government officials.
- Murder
- Rape etc

Moral precepts are straightforward, but applying them in practice is another matter. Readers may recall the film "Rwanda Hotel" which did the rounds in 2004. It tells the story of Paul Rusesabagina, a Rwandan Hutu married to a Tutsi who saves his wife and thousands of Tutsis from the Hutu genocide using a combination of bluster, lies, threats, and bribes. Eventually, he resorts to blatant blackmail using some clever lies, and leads a large number of Tutsis to safety.

So while lies, bribery, and blackmail may be morally wrong in principle, in practice there are situations, such as that above, where their application is problematical, giving rise to the term "situation ethics".

DETERMINING RIGHT FROM WRONG. Is there a sound basis for judging right from wrong? Some people will give a very positive answer and others the opposite. The following examples illustrate two extreme views, both taken from the NZ Herald of 15 July, 2014. On page A24 an Anglican vicar is reported as saying "the Bible teaches that the only rightly ordered sexual union is that between a male and a female, within the context of marriage". This simplistic, black and white view, is commonly referred to as the "prescriptive" approach, and is often held by people with strong religious beliefs.

On the previous page of the same Herald, there is an article by Bob Jones in which he claims that morality and immorality are simply a reflection of the values of society at a particular time and place, and they have no universal validity. He states that "throughout the 1000-year Greek and Roman civilisations, the offences of Jimmy Saville, Rolf Harris and company were acceptable behaviour, so too was the priests and scoutmaster's paedophilia".

Neither of these approaches is sound. Consider Bob Jones' approach. The victims of the offences described no doubt suffered equally in all periods of history, and considered what was done to them was wrong. The offences were acceptable only in the eyes of those in power, almost invariably male adults.

Turning back to the prescriptive approach of the Anglican vicar, it is based on a claim that a book written two thousand or more years ago reveals the laws of God. Other religions also make similar claims for their holy books. Using

holy books in this way leads to a legalistic approach to moral questions, and comes up with different answers depending on which religion is consulted.

The above discussion suggests a basis on which to make ethical judgements, that is, to consider the effect of an action on individuals, or on society at large. Put simplistically, an action cannot be considered right, or ethical, if it harms individuals or society. This is also a little too simplistic, as an action may harm some individuals and greatly benefit others. Thus some sort of balance has to be sought, which may be easy in some situations but not in others.

Although critical remarks were made above about religious beliefs and a legalistic approach to answering moral questions, it is still true that the core moral teaching of most, if not all, major religions has much in common. This core teaching is that we humans should be as concerned for the welfare of our fellow human beings as we are for our own. "Treat others as you would like them to treat you" (the so-called **Golden Rule**) or "Love your neighbour as you love yourself" are well known precepts from the Bible. These simple concepts are found in all the world's religions in some form.

ETHICAL ISSUES IN THE WORKPLACE

Ethical questions inevitably arise in all types of work. In some cases, the ethical issue is simply one of delivering a quality service at an appropriate price. In other cases, ethical questions are almost inherent to the nature of the work. Typical examples of the latter are the advertising and public relations professions. Advertisers are primarily in the business of persuading us to buy a particular product. Many advertisements are aimed at getting people to buy things they don't need and probably can't afford, and the claims made about a product are often misleading. Is this ethical?

Several years ago a major bank in New Zealand conducted an aggressive advertising campaign to get people to take out loans, regardless of whether they would benefit from them, or afford them. Fortunately, the staff had much higher ethical values than their employer and they staged demonstrations outside branches. It is reassuring to know, despite what I said earlier, that ethical values are still held by many people in our society, and they are prepared to stand up for those values.

The December 2014 edition of *North and South* had an interesting article titled "Who Do You Trust", which looks into the operations and ethics of the public relations industry. It is rather like the advertising industry, but even less trustworthy. As the writer of the article states, the PR industry is very influential in today's world, and operates under very "flexible" ethical boundaries. PR companies promote themselves with statements like:

- "the art of reputation management"
- "value added communication outcomes"
- "strategic management of reputations and outcomes"

The PR Institute of New Zealand (PRINZ) has a code of ethics and members can be disciplined for breaking it, but only a few complaints have been received and no penalties imposed on any of its members. However, only about a quarter or third of PR firms are members of PRINZ, so most have no ethical restraints.

Developers would come very close to the top of my list of organisations devoid of ethical scruples. The leaky building disaster is often blamed on technical factors, but behind the technical detail lies the fact that it was due primarily to the lack of ethical scruples of developers. The big name developers were using every ploy they could to get architects, engineers, and builders to take shortcuts to reduce costs. Sadly, these professions, along with local authority staff, gave in to the developers to varying degrees. When the disaster became apparent the developers were able to claim that they engaged qualified architects, engineers, and builders.

ETHICS OF THE PROFESSIONS, INCLUDING CIVIL ENGINEERING

Ross Roberts, in the conclusion to his article on professionalism, makes the very valid point that a profession should be more than an advanced trade, and those claiming to be professionals should have higher ethical values, and that "a certain degree of altruism or selfless service is expected". I think most of us would aspire to this ideal, but in today's world it is the "free market" that we are expected to embrace in the belief that it will deliver all the best outcomes. Why anyone believes this is hard to understand - the free market has no ethical values, it is unpredictable, and seems to create and totter from one financial crisis to the next. But I must return to the question of professionalism.

Professions such as medicine, law, and engineering have codes of ethics which require their members to conduct themselves within specific

ethical boundaries. Engineers are in a somewhat different position to the medical and legal professions in that the latter have very rigorous ethical codes and are regulated by government statute. Constraints on engineers are far less rigorous - they can practice without being members of IPENZ or subscribing to any particular ethical code, other than that which arises from common law. I once heard engineers described as "functionaries", that is they perform necessary technical functions at the behest of their masters, who have a dominant say in how engineers go about their work. Engineers are not in control of their profession in the way lawyers and doctors are.

The IPENZ Code of Ethics gives some good guidelines. This code, quite rightly, states that the engineer's responsibility is threefold:

(1) to his or her client
(2) to society
(3) to the environment

This is a tall order, and it is not surprising that engineers are virtually forced to give preference to one or other of these responsibilities at the expense of others.

SPECIFIC ETHICAL CHALLENGES FACED BY CIVIL ENGINEERS.

1. Should geotechnical engineers accept briefs for site investigations when the client is only prepared to pay a sum less than that needed for an adequate investigation?

The simple answer is no, but this assumes that an "adequate investigation" is easy to define. I think there is a general impression that site investigations are becoming less rigorous than they once were, and this is largely due to competitive bidding for site investigations, and consequent lowering of standards.

2. Can an engineering project ever be good engineering if it does not clearly benefit society? In other words, can it be judged purely on its technical content without regard to its purpose, or its inevitable side effects?

An extreme example of this question is whether the gas chambers used by the Nazis were good engineering simply because their design served its purpose magnificently, maximising the daily output, and minimising the gas usage? Closer to home we can consider Sky City, which is predominantly a gambling facility. Can this ever be good engineering? Or, more specifically, is the benefit a gambling facility brings to society greater than the harm it does to countless individuals? Many people quite rightly view casinos as a means by which the rich and powerful extract money from those who are unwise, gullible and often poor. I was told at the time Auckland casino was first designed that a structural engineer resigned from the company designing it because he viewed the whole project as unethical. The Sky City tower may be clever engineering, but the sole reason it was built was that it helped the successful applicant obtain the right to build the casino. I personally cannot accept that the sky tower is good engineering, for the simple reason that it is a dominant part of an institution that does more harm to society than good.

3. Should consultants pay bribes on order to obtain contracts?

Some New Zealand consultants have paid bribes in order to gain contacts in Asian countries, while others would not pay bribes under any circumstances. I once heard a philosophy lecturer say that if bribery is a normal part of the local culture, then it is not the place of outsiders to condemn it. This is a feeble argument, for while it may be part of local culture, the only people in favour of it are those who benefit from it. The bulk of the population are normally totally opposed to it.

4. Is a mining project that will encroach into a national park acceptable?

This is one of any number of questions that can be raised in relation to engineers' responsibility for protecting the environment. Public concern for the environment is steadily increasing and engineers ignore this trend at their peril. The start of the modern environmental movement and its associated discipline of "Environmental Ethics" is generally attributed to a book called "Silent Spring" by Rachael Carson, published in 1967. In New Zealand, it was the Tongariro and Lake Manapouri power schemes that ignited public concern for the environment. The original Manapouri scheme proposed raising the lake level by up to 30m and merging it with Lake Te Anau. The "Save Lake Manapouri" movement began in 1959 and eventually forced the government to abandon plans to raise the lake level. Civil engineers worldwide have lost a lot of respect because of a perception that they do not really care about the environment and put economic gain ahead of environmental protection. This is often the case because they are subservient to the wishes of the organisations that employ them.

CONCLUSION

The view expressed by Ross Roberts in his article is very important. There is little point in claiming to be professional if this does not also entail subscribing to a higher ethical code than that often accepted by society in general.

Introduction

MEMBERS OF THE New Zealand Geotechnical Society are among the best geotechnical professionals in the world. In this issue of Geomechanics News we're celebrating some of their greatest recent achievements. The NZGS presents a number of prestigious awards to members who have made the most impressive technical contributions at a range of levels, starting with our Student Awards.

2014 NZGS STUDENT AWARDS

The New Zealand Geotechnical Society Student Awards are presented to recognise and encourage student participation in the fields of geotechnical engineering and engineering geology. The Student Awards evolve each year, and most recently took the form of a poster competition. For these awards students of a recognised tertiary institution in New Zealand are required to submit an abstract for their poster to register for the award and then prepare an A1 size poster that clearly and concisely presents their work. The posters were judged and ranked by a panel of three judges, then displayed at an Auckland branch meeting where NZGS members were also able to submit their vote for the best 3 posters. The member vote was added to the scores of the judges. Posters were judged on technical content, layout, and overall poster appeal. The winner of the best poster received a $1000 prize, with second and third place receiving $500 and $300 respectively. The winners were announced to the branch members on the day of the poster display as:

- 1st Place - Chris Baker & Innes Duncan - Can Chemical Alteration Reduce Long Term Productivity in Deep Geothermal Reservoirs?
- 2nd Place - Darshan Pradhan & Timur Sibaev - Effect of near-fault ground motion on bridges including soil-foundation-structure interaction
- 3rd Place - Yi Lu & Yuan Hong - Numerical Modelling of Iron Ore Liquefaction

Congratulations go to all the entrants on their excellent posters. First and second place posters are reproduced in this edition of Geomechanics News; unfortunately confidentiality requirements prevented the printing of the third place poster.

2015 ANZ CONFERENCE AWARDS

During the ANZ conference each paper was judged by a panel of five experts, and four awards presented for the best technical papers. For the best paper there were 35 papers shortlisted by the judges, who then attended the presentations of each of the ten finalists to select their best over-all paper. The winners were:

- Best Paper: Rob Hunter – "Development of horizontal soil mixed beams as a shallow ground improvement method beneath existing houses".
- Runner up: Julian Seidel – "Overview of the role of testing and monitoring in the verification of driven pile foundations".
- NZ Best YGP: Brendon Bradley – Site specific hazard analysis for geotechnical design in New Zealand".
- AU Best YGP: Udeshini Pathirage – "Reducing the risk of acidic groundwater through modelling the performance of a permeable reactive barrier in Shoalhaven floodplain".

About thirty of the papers at the ANZ conference were selected for presentation by poster rather than aural. These were all entered for the People's Choice Poster Awards. Delegates voted during the conference, and the results were announced in the closing ceremony:

- 1st Place: Maxim Millen – "Earthquake induced rotation and settlement of building foundations".
- 2nd Place: Christopher Robson – "Engineering geology and stabilisation of the 2011 landslide which closed SH3 in the Manawatu Gorge, New Zealand".

Maxim Millen's poster was a development of his 'The Rise of SFSI' graphic published in the December 2014 edition of Geomechanics News.

2014 NZGS GEOMECHANICS AWARD

The New Zealand Geotechnical Society Geomechanics Award is made to the society member or members producing the adjudged best published paper during the previous three years. The winning paper will be that considered to be distinguished in its contribution to the development of geotechnics in New Zealand. The NZ Geomechanics Award is bestowed on the author(s) of papers that are distinguished in their contribution to the development of geotechnics in New Zealand.

An NZGS judging subcommittee comprising Charlie Watts, Ann Williams and CY Chin recently concluded that the 2014 Geomechanics Award be given to Tam Larkin & Chris Van Houtte for their paper "Determination of site period for NZS1170.5:2004" published in the New Zealand Society for Earthquake Engineering Bulletin, Vol. 47, No. 1, March 2014.

The judges reported that the paper "responds directly

Above: Change text to "Rob Hunter (centre) receives his award for best paper at the ANZ 2015 conference from Darren Paul (Australian Geomechanics Society Chair, left), and Guy Cassidy (ANZ 2015 Conference Chair, right)

Above: Julian Seidel is runner up for best paper in the ANZ 2015 conference

Above: Udeshini Pathirage receives her award for best Australian YGP paper.

Above: Brendan Bradley receives his award for best NZ YGP paper.

Above: Tam Larkin receives his award on behalf of himself and Chris Van Houtte from Gavin Alexander at an NZGS Auckland Branch meeting presentation in June.

to a long-standing issue in geotechnical practice, clearly written and explained, which may contribute to changes in NZS1170. It provides geotechnical practitioners invaluable guidance on the determination of shear wave velocity of layered sites and the fundamental site period. Comparisons of various methods tabled in NZS1170 and the accuracy of these methods are made. The assessment of site period is critical due to implications on establishing spectral accelerations for design. The contribution made by this paper both internationally (as methods tabled in NZS 1170 are also adopted internationally) and locally is significant and distinguished."

The NZGS Committee congratulate Tam and Chris on this significant achievement, and encourage all members to read this paper. It is reproduced in this issue of Geomechanics News to reach the widest audience possible.

After receiving the news of their paper's selection, Tam Larkin wished to convey his thanks to the Society, "Thank you for that wonderful news. It is a real honour to receive such an award from the Society. It is especially heartening to see such recognition going to a current post-graduate student. It is useful to recognise the very significant contribution of these folk to the development of civil engineering in New Zealand and beyond. Without these people most of the research ideas in the minds of the engineering community would remain just that – an "idea". Recognition by the Society of the value of their efforts will spur young engineers on in the days to come."

Three further papers were identified as of very significant value. Where permission was available these have been re-printed in the digital edition of Geomechanics News. The abstracts are presented on the following pages:

NZGS GEOMECHANICS AWARD
RUNNER-UP
See digital edition for full paper

Management and documentation of geotechnical hazards in the Port Hills, Christchurch, following the Canterbury earthquakes

D. F. Macfarlane & M. D. Yetton

PROC. 19TH NZGS GEOTECHNICAL SYMPOSIUM

This paper outlines the scope of work undertaken by the Port Hills Geotechnical Group (PHGG) to identify, document and assist with the management of geotechnical hazards in the Port Hills area of Christchurch following the 22 February 2011 earthquake. PHGG was a group of consultants formed during the state of emergency and subsequently contracted by Christchurch City Council (CCC). The Group consisted of geotechnical engineers and engineering geologists drawn from Aurecon, Geotech Consulting, GHD, Opus and URS, with support from Canterbury University, Bell Geoconsulting, SKM, Geoscience Consulting and Meridian Energy at different times.

PHGG began as a group dedicated to documenting instability in rock slopes (cliff collapse and boulder roll scenarios) and ground cracking to identify high risk areas and dwellings that should not be occupied as well as identifying areas suitable for protective works and liaising closely with CCC and the public. Following the Civil Defence phase, the role transitioned into working closely with GNS Science and CCC to assist with the development of life-risk models and risk zones while concurrently defining and supervising work packages for interim protective works and more permanent remedial works for key Council infrastructure and parkland.

The PHGG was a very successful team of dedicated professionals that developed excellent working relationships internally and worked collaboratively with its key stakeholders (CCC and GNS). A large part of the reason for this success was that the individuals and companies involved put the job first and focussed on achieving the best possible outcome for the city and its residents.

NZGS GEOMECHANICS AWARD
RUNNER-UP
See Engineering Geology for full paper

Patterns of movement in reactivated landslides

C.I. Massey, D.N. Petley, M.J. McSaveney

ENGINEERING GEOLOGY 159 (2013)

The primary aim of this research was to study the relationship between landslide motion and its causes, with reference to large, slow moving, reactivated translational rock slides. Surface displacements of the 22 · 106 m3 Utiku landslide, in central North Island, New Zealand were measured using continuous GPS (cGPS), for three years. The nature of the movement of such slides has often been difficult to determine because of poor temporal and spatial monitoring resolutions. After removal of tectonic plate motion, the temporal pattern of the landslide's surface motion could be understood to arise from irregular episodes of faster (up-to-21 mm/day) and slower (up to 26 mm/yr) post-failure landslide displacement, and seasonal cyclic displacements of about 20 mm/yr–10 mm per half year in alternating directions. Intervals of faster motion gave rise to displacements of between 10 and 120 mm per event. Faster displacement was associated mostly with basal sliding (mechanism 1), involving deformation within a thin clay seam as recorded by borehole inclinometer surveys. Slower surface displacement involved permanent internal deformation of the larger landslide mass, consisting of plastic deformation within the landslide body and/or slip along existing internal planes of weakness, and slip on the slide base (mechanism 2); it accounted for up to 26 mm/yr of displacement at a mean angle of about 49° from the horizontal, indicating that the slide mass was thinning as it moved down slope. Seasonal cyclic displacements were synchronous with changes in pore pressure, suggesting that it is a shrink/swell process (mechanism 3) associated with wetting and recharge of groundwater during the wetter winter months, leading to a downslope movement, and soil shrinkage leading to upslope rebounds during the dryer summer months. The brief periods of faster displacement were triggered by seasonal peaks in pore pressure, linked to long periods (12 to 20 weeks) of increased precipitation and lowered evapotranspiration. Faster displacement, however, was not arrested by lowering pore pressure or by any other monitored factor. Similarly, periods of slower displacement did not correlate with pore pressure changes, or with any other monitored factor. This study has shown that the annual movement pattern of a reactivated landslide is a combination of these processes that generate a complex overall movement record. The field measurements showed real variability arising from variations in rainfall and pore pressure, which were overprinted with measurement noise that may mask some other processes.

NZGS GEOMECHANICS AWARD
RUNNER-UP
See digital edition for full paper

Assessment of Liquefaction-Induced Land Damage for Residential Christchurch

S. van Ballegooy, P. Malan, V. Lacrosse, M.E. Jacka, M. Cubrinovski, J.D. Bray, T. D. O'Rourke, S.A. Crawford, H. Cowan

EARTHQUAKE SPECTRA: FEBRUARY 2014, VOL. 30, NO. 1

Christchurch, New Zealand experienced four major earthquakes (Mw 5.9 to 7.1) since 4 September 2010 that triggered localized to widespread liquefaction. Liquefaction caused significant damage to residential foundations due to ground subsidence, ground failure, and lateral spreading. While liquefaction effects were expectedly severe in some suburbs, there was little to no damage in other areas, where more serious effects were expected based on existing liquefaction vulnerability criteria. These damage variations indicate that existing liquefaction vulnerability criteria do not capture fully the consequences of liquefaction. This paper first presents some general features of liquefaction-induced damage to land and dwellings in residential areas, and then examines the effectiveness of liquefaction vulnerability parameters in predicting/explaining the observed liquefaction-induced damage in residential areas of Christchurch. The Liquefaction Severity Number (LSN), a new parameter, is presented and discussed using results from 5,500 CPT and validated regional groundwater models.

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NZGS GEOMECHANICS LECTURE AWARD

The NZ Geomechanics Lecture is the premier award of the New Zealand Geomechanics Society, established to honour individuals who have made a notable contribution to New Zealand Geomechanics. It is presented by a person prominent in Geomechanics who can, in the presentation, contribute a statement of significance and value relevant to New Zealand. Previous presenters of the Geomechanics Lecture have been:

2011 David Bell – Geo-Logic and the Art of Geotechnical Practice
2007 Toan D V – A Geomechanics View on Heavy Duty Pavements
2004 Wesley L – Geotechnical Engineering in and out of the Ivory Tower
2001 Prebble W – Hazardous Terrain – an engineering geological perspective
1999 Sinclair T J E – Geotechnical Analysis – Fundamentals to Fractals
1996 Pender M J – Aspects of Geotechnical Behaviour of Some New Zealand Materials
1994 Berrill J B – Seismic Liquefaction of Cohesionless Sands
1992 Martin G R – Geomechanics – The Art and the Science
1990 Taylor D K – The Use and Misuse of Geotechnology in Civil Engineering
1987 Oborn L E – Thoughts on the Evolution of Engineering Geology in New Zealand
1984 Taylor P W – Geotechnical Engineering: Education and Practice in New Zealand
1979 Northey R D – The Acceptability of Geotechnical Risk
1975 Wroth C P A – Fresh Look at Damage Caused to Buildings by Settlement
1974 Ridley J W – The Economics and Correct Use of Natural Materials

Above: John Wood was presented with his Geomechanics Lecture Award during the ANZ conference

IN 2014 THE FIFTEENTH GEOMECHANICS LECTURE WAS PRESENTED BY JOHN H. WOOD.

John is well known to the geotechnical community for his work on the seismic design of various structures including retaining walls and reinforced earth structures. He has published many papers on testing, design, performance and strengthening of these structures. His thesis on earthquake induced soil pressures on structures was written early in his career and is still regarded as a standard reference.

John is a consulting civil engineer specialising in bridge design, structural investigation, soil-structure interaction and earthquake engineering. Before setting up his consulting engineering practice in 1986, he was Head of the Ministry of Works, Central Laboratories.

His recent work includes peer reviews of seismic strengthening proposals and seismic risk assessment for hydro power stations. He has carried out bridge strengthening design and peer review for the New Zealand Transport Agency, and research into the earthquake performance of underground structures, reinforced earth retaining walls and bridge abutments.

John is a Life Member and past President of the New Zealand Society for Earthquake Engineering. He holds post-graduate degrees in structural and civil engineering from both the University of Canterbury and California Institute of Technology

John has toured New Zealand presenting his lecture, and Part 1 of his written paper is published in this edition of Geomechanics News.

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15th Gomechanics Leture – Geotechnical Issues in Displacement Based Earthquake Design of Highway Bridges and Walls

John Wood is a consulting civil engineer specialising in bridge design, structural investigation, soil-structure interaction and earthquake engineering. Before setting up his consulting engineering practice in 1986, he wasHead of the Ministry of Works, Central Laboratories. His recent work includes peer reviews of seismic strengthening proposals and seismic risk assessment for hydro power stations. He has carried out bridge strengthening design and peer review for the New Zealand Transport Agency, and research into the earthquake performance of underground structures, reinforced earth retaining walls and bridge abutments. John is a Life Member and past President of the New Zealand Society for Earthquake Engineering. He holds post-graduate degrees in structural and civil engineering from both the University of Canterbury and California Institute of Technology

LECTURE SUMMARY

There is a growing emphasis on Displacement Based Earthquake Design (DBD) analysis for buildings, retaining walls and bridge structures. DBD is specified as the preferred design method for highway structures in a draft revision of Section 5 of the Bridge Manual (NZ Transport Agency) expected to be adopted in 2015.

For bridges and major retaining wall structures, the damping and deformations within their foundations and backfilling have a major impact on their displacement response. In the past, the geotechnical input for the design of structures has focused on investigating and defining the soil strength parameters. To implement DBD methods there is now a need to investigate and assess soil stiffness as well as strength and to focus more on soil-structure interaction analysis.

The lecture highlighted the influence of soil stiffness and damping on the earthquake response of retaining walls and bridges and discussed the effects of the uncertainty in these parameters. DBD design procedures were illustrated by examples from the presenter's research background on soil-structure interaction.

The lecture was presented in the following three parts:

1. Rigid, and flexible including outward sliding retaining walls
2. Stiff retaining walls and bridge abutments
3. Bridge DBD and pile foundations

This paper presents material discussed in Part 1 of the lecture. Material presented in Parts 2 and 3 will be published in the December 2015 issue of NZ Geomechanics News.

1. RIGID RETAINING WALLS

The pressures that develop on retaining walls during earthquakes are sensitive to the flexibility of the structural components of the wall and the ability of the wall to move outward as a result of permanent deformations in the foundation soils or ductility within the structure. It is important that any seismic analysis method or design procedure used for walls makes at least a gross recognition of the wall flexibility and deformation capability.

Many low walls are of cantilever type construction. In this type of wall lateral pressures from vertical gravity and earthquake forces will generally produce sufficient displacement within the wall structure to induce nonlinear behaviour or a fully plastic stress state in the retained soil. In more rigid free-standing walls, such as gravity (e.g. reinforced earth and crib blocks) and counterfort walls, a fully plastic stress state may develop as the result of outward movement arising from permanent sliding or rotational deformations in the foundation. In cases where significant nonlinear soil behavior or a fully, plastic stress state occurs in the soil, the assumptions made in the widely used Mononobe-Okabe method (Mononobe and Matsuo, 1929) of estimating earthquake forces on walls are valid and the method gives earthquake force predictions that are acceptable for design purposes. Estimates of the outward movement of these types of wall can be made using Newmark Sliding Block theory (Newmark, 1965 and Richards and Elms, 1979).

A significant number of retaining structures are not free standing or have rigid

foundations and therefore, even under severe earthquake loading, they may not displace sufficiently for a fully plastic active stress state to develop. Examples of these types of walls are; bridge abutments that may be rigidly attached to the bridge superstructure or founded on piles, basement walls that are an integral part of a building on a firm foundation, and closed culvert or tank structures embedded in the ground. For these walls the Mononobe-Okabe (M-O) assumptions are not met and earthquake pressures and forces may be underestimated by application of this method.

Analytical studies for rigid walls were carried out by the author in 1971 (Wood, 1973) and were motivated by the lack of an established design procedure for estimating earthquake forces on rigid walls and walls where the Mononobe-Okabe assumption of a fully plastic stress state is not appropriate. This work was used as the basis for design guidelines recommended by the New Zealand Society for Earthquake Engineering (NZSEE) for the earthquake design of bridges (Matthewson et al, 1980). These guidelines departed from other design recommendations in that an attempt was made to relate the earthquake pressures to the wall flexibility. Recommended rigid wall pressures and forces were significantly higher than given by the M-O method. Following this theoretical study shaking table experimental work was carried out at the NZ Ministry of Works Central Laboratories (Yong, 1985 and Wood and Yong, 1987) to provide a verification of the proposed rigid wall design forces. Design recommendations based on both the theoretical and experimental work were published in the NZ Road Research Bulletin 84 (Wood and Elms, 1990). These were similar to the earlier NZSEE recommendations.

1.1 Theoretical Solution

The theory of elasticity solution for the pressures on a rigid wall statically loaded by a uniform horizontal body force in an elastic soil layer (Figure 1) is summarized below. Details of this solution together with the solution for the related problem of horizontal dynamic forcing of the rigid boundaries are given by Wood, 1973. The static solution gives a good approximation to the dynamic loading case providing the dynamic forcing frequencies are less than the natural frequencies of the retained soil layer.

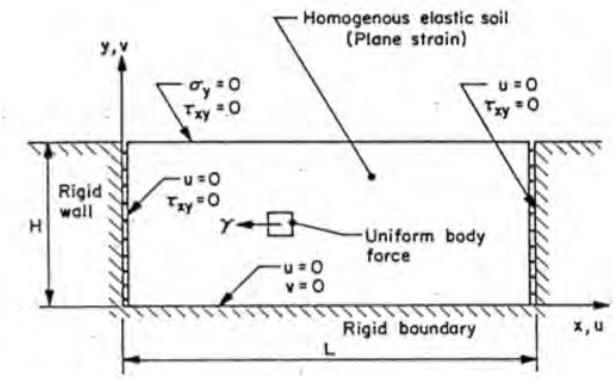

Figure 1:

The vertical end boundaries in Figure 1 represent rigid walls and the contact between the homogeneous linearly elastic soil and the wall is assumed to be smooth; that is, the vertical boundaries are assumed to be free of shear stresses. The lower horizontal boundary represents a rigid rock layer. A uniform static horizontal body force is assumed to act throughout the soil layer. For convenience, the magnitude of this body force is taken as the unit weight of the soil, and so represents the application of a static horizontal acceleration of one g. Under the assumption of plane strain, the equilibrium equations for a homogeneous, linearly elastic, isotropic soil layer are,

$$\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{xy}}{\partial y} + F_x = 0$$
$$\frac{\partial \sigma_y}{\partial y} + \frac{\partial \tau_{xy}}{\partial x} + F_y = 0 \qquad (1)$$

Where:

σ_x, σ_y = normal stresses in the x- and y-directions respectively

τ_{xy} = shear stresses in the x- and y-directions

F_x, F_y = body forces per unit volume in the x- and y-directions respectively

The stress-strain relations can be expressed as,

$$\frac{\sigma_x}{G} = k^2\frac{\partial u}{\partial x} + k^2\frac{\partial u}{\partial x} + (k^2 - 2)\frac{\partial v}{\partial y}$$
$$\frac{\sigma_y}{G} = (k^2 - 2)\frac{\partial u}{\partial x} + k^2\frac{\partial v}{\partial x} \qquad (2)$$
$$\frac{\tau_{xy}}{G} = \frac{\partial u}{\partial y} + \frac{\partial v}{\partial x}$$

Where:

G = shear modulus

$k^2 = 2(1 - \nu)/(1- 2\nu)$

ν = Poisson's ratio

Substitution of expressions (2) into (1) gives the equilibrium equations for the problem in terms of displacements. For the case of no vertical body force these equations are,

$$k^2\frac{\partial^2 u}{\partial x^2}+\left(k^2-1\right)\frac{\partial^2 v}{\partial x\partial y}+\frac{\partial^2 u}{\partial y^2}=\frac{\gamma}{G} \qquad (3)$$

$$k^2\frac{\partial^2 v}{\partial y^2}+\left(k^2-1\right)\frac{\partial^2 u}{\partial x\partial y}+\frac{\partial^2 v}{\partial x^2}=0$$

For $0<x<L$ and $0<y<H$

The boundary conditions are defined in Figure 1. By expanding the constant forcing term γin a Fourier series it is possible to derive a complete displacement solution for Equations (3). Substituting the displacement solution into the stress-strain relations gives a solution in terms of stresses throughout the elastic soil layer. The normal stress on the wall boundary can be ob·tained from this solution in the following dimensionless form,

$$\frac{\sigma_x}{\gamma H}=\frac{4}{\pi^2 k^2}\frac{L}{H}\sum_{n=1,2,3....}^{\infty}\frac{1}{n^2}\left\{2B_n\cosh ry+C_n\left(2ry+k'+3\right)e^{ry}+D_n\left(2ry-k'-3\right)e^{-ry}-k^2\right\}\cos rx \qquad (4)$$

Where:

$r = n\pi/L$

$k' = 3 - 4v$

The coefficients B_n, C_n and D_n are determined by satisfying the boundary conditions at y = 0 and y = H.

The force on the wall and moment about the base of the wall are found by integrating the normal pressure distribution on the wall and the moment of the pressure distribution about the base respectively.

Typical plots of the wall pressure distributions are shown in Figure 2 for a Poisson's ratio of 0.3 and the wall forces and moments are shown in Figures 3. The elastic solution is dependent on the value of Poisson's ratio but is independent of the other soil stiffness (and strength) parameters. In design application this eliminates the need to investigate the elastic modulus of the backfill.

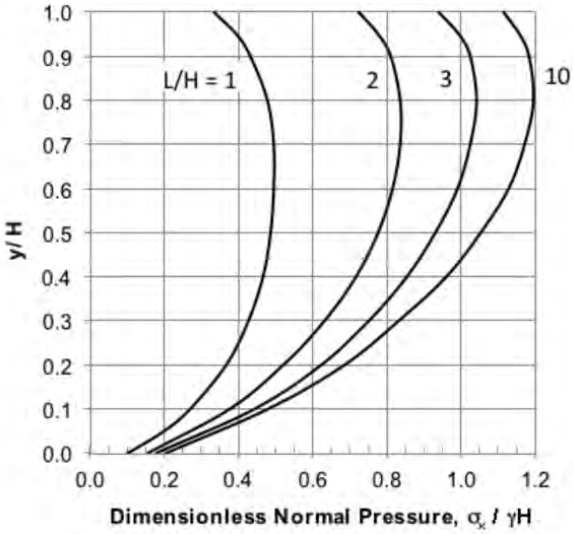

Figure 2: Pressures on smooth rigid wall for static 1-g horizontal body force

Finite element analyses for the rigid wall elastic soil problem showed close agreement between wall pressures for the analytical solution (smooth wall) with a fully bonded contact assumption. For the bonded case a stress singularity occurs at the top of the wall but this does not significantly change the force on the wall.

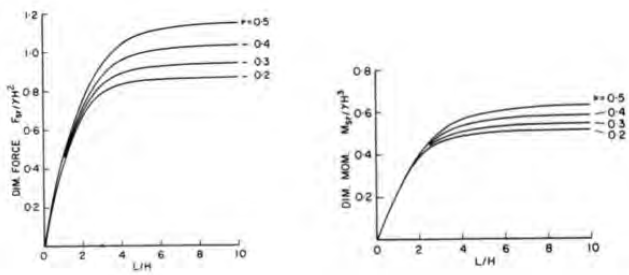

Figure 3: Force and moment on smooth rigid wall for 1- g static horizontal body force.

1.2 Experimental Investigation

Two series of rigid model wall tests were carried by Yong 1985 and Thurston 1986. In the first series, a dry sand material was used as backfill. This work was extended in the second series by also using moist sand and by carrying out a more detailed examination of the residual soil pressures that were found to develop as loose sand was

compacted by vibrations of the shaking table used to test load the model walls.

The testing was carried out using a sand box mounted on a shaking table driven in a single horizontal direction by a 250 kN capacity servo controlled electro-hydraulic actuator. The model wall consisted of a 25 mm thick aluminum plate, 0.6 m high by 2.24 m wide, mounted at one end of the 2.15 in long sand box. The wall was supported horizontally by eight load cells arranged in two horizontal rows. This support method ensured that the flexural deformations of the wall were negligible producing essentially rigid wall behaviour.

Details of the model wall and instrumentation are shown in Figure 4. Pressures on the wall were measured with five 19 mm diameter diaphragm type pressure transducers located on the vertical centerline of the wall. Four accelerometers were set up at various locations to record the motion of the table, wall and the sand mass.

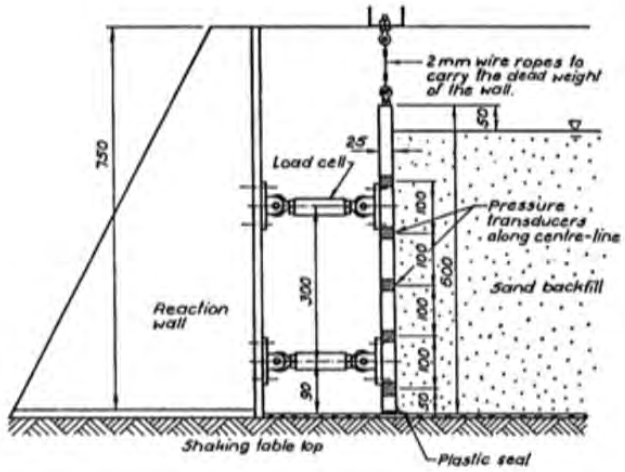

Figure 4: Model wall test arrangement

The backfill soil was uniform medium sand having a D_{50} size of 0.25 mm and a C_u of 2.3. The sand was placed into the sand box by showering from a screw conveyor which resulted in a very loose density state. After initial wall tests, a dense sand state was achieved by translational vibrations of the table. Measured densities for the very loose as-placed state and the maximum density state were 14.6 and 16.2 kN/m^3 respectively. The dense sand had a friction angle of 40°.

Wall pressures were measured in a series of tests using a 4 Hz sinusoidal acceleration input with the table peak acceleration gradually increased from 0.05 to 0.6 g. At the end of the loose sand tests the box was topped up with loosely placed sand and the complete sand volume compacted by 100 cycles of shaking at a peak acceleration of 0.6 g. Tests were also repeated for the sand in a dense moist condition with an average moisture content of about 18 %.

At the completion of the fixed frequency tests a further series of tests were performed sweeping the input frequency from 2 to 65 Hz at a constant peak acceleration of 0.3 g. This test provided information on soil amplification and resonance effects from dynamic loading.

1.3 Experimental Results

The incremental dynamic force on the wall for the constant 4 Hz sinusoidal shaking of the dense dry backfill is plotted in Figure 4. The force increment is plotted in dimensionless form by dividing the measured dynamic force by the unit weight of the backfill γ, the square of the wall height H and the acceleration coefficient C_o. For comparison, incremental dynamic forces from the theory of elasticity solution and the M-O method for a soil friction angle of 40° are also shown. There is good agreement between the experimental results and the theory of elasticity solution. As expected the M-O incremental force is considerably less than the corresponding experimental value.

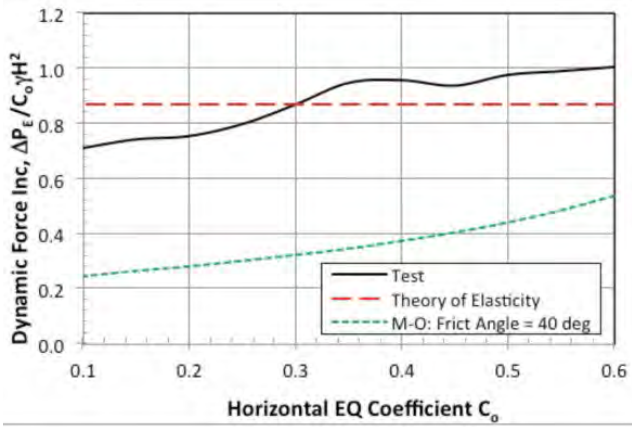

Figure 5: Comparison of dynamic force increment results for dense sand with theory and M-O.

The experimental force is a little less than the theoretical value at acceleration coefficient values less than 0.25 and a little higher at coefficient values greater than 0.35. Part of these differences was attributed to side friction in the sand box at low accelerations and sliding of the sand on the base at high accelerations.

Over the range of accelerations tested, the experimental centre of pressure for the dense and moist backfill was at an almost constant height of about 0.58 H above the base of the wall. This is in good agreement with the theoretical value of 0.57 H.

The dynamic force ratio on the wall measured during the frequency sweep carried out on the dense backfill is

shown in Figure 6. The plotted force ratio is defined as the dynamic increment of wall force at a particular frequency divided by the dynamic increment measured at the 4 Hz frequency (essentially the static force). The frequency has been made dimensionless by dividing the test frequency by the first mode frequency of an infinitely long soil layer (shear mode). For a Poisson's ratio of 0.4 and a Young's modulus of 2.2 MPa the first mode frequency of a 0.55 m deep shear layer with a unit weight of 16.2 kN/m3 (dense sand) is approximately 10 Hz. A distinct resonance peak was observed between dimensionless frequencies of 1 to 2.5 (test frequencies of 10 to 25 Hz).

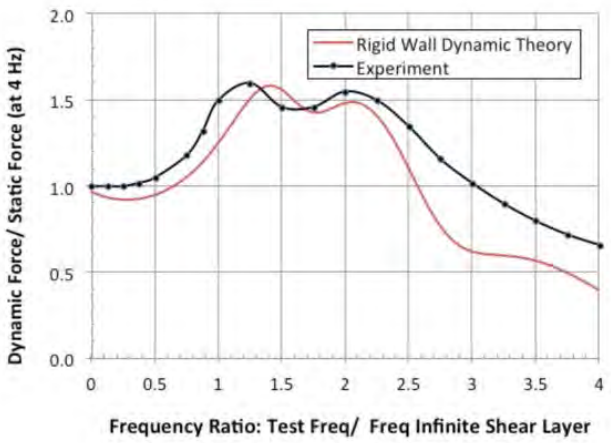

Figure 6: Comparison of test and theoretical dynamic force ratio for harmonic forcing

Also plotted in Figure 6 is the rigid wall dynamic theory of elasticity solution for 20% damping and a length over height ratio L/H of 3.0 (sand box L/H was 3.6). For harmonic forcing with angular frequency ω, the complex dynamic to static force ratio ($F_r'(\omega)/F_{sr}$) is (Wood, 1973),

$$\frac{F_r'(\omega)}{F_{sr}} = \sum_{n=1,m=1}^{\infty} \frac{F_{n,m}/F_{sr}}{\left\{\left(1-\frac{\omega^2}{\omega_{n,m}^2}\right)+2i\zeta_{n,m}\frac{\omega}{\omega_{n,m}}\right\}} \qquad (5)$$

Where:

$F_{n,m}$ = dynamic force component in mode n,m

$\omega_{n,m}$ = angular natural frequency of mode n,m

$\varsigma_{n,m}$ = damping ratio in mode n,m

The theoretical curve shown in Figure 6 was based on a summation of the lowest five modes (each with 20% damping) and a correction for higher neglected modes.

Figure 6 shows reasonable agreement between the test and theoretical results for harmonic forcing and indicates that high damping can be expected in retained sand layers during strong shaking. The Young's modulus value of 2.2 MPa assumed for the comparison is very low but nevertheless it may be a reasonable value for the very low confining stresses in the 0.55 m deep sand layer.

The strong shaking generated in the tests produced an increase in the static soil pressures recorded before and after the completion of the tests. For the dense case with peak accelerations lower than 0.3 g, the increase in the static force was less than 2%. The dynamic pressures generated on the wall during shaking cause small wall deflections and settlements. After each pressure pulse the wall does not return completely to its original position with the result that a residual pressure develops as the number of cycles or acceleration level is increased.

1.4 Design Assumptions

Figure 7 shows a comparison of the theory of elasticity solution (L/H=10, ν=0.3) with the design recommended pressure distribution on a rigid wall for a static horizontal acceleration of $C_o\gamma$. The design pressure distribution was recommended by both NZSEE (Matthewson et al, 1980) and RRU (Wood and Elms, 1990). The design curve is simplified and has the correct centre of pressure at 0.58H but it gives a pressure force approximately 10% higher than the theory of elasticity solution. Although the difference is not very great there is no valid reason for a conservative approach since small wall movements and wave scattering will reduce the pressures.

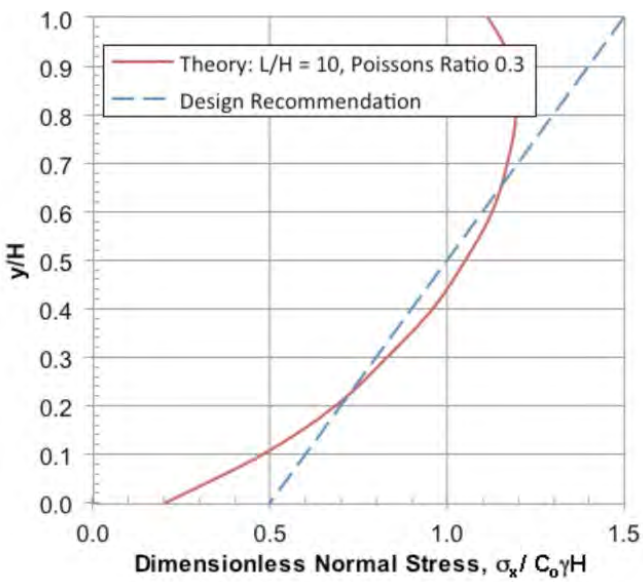

Figure 7: Comparison of theory of elasticity pressure distribution with design recommendation.

Earthquake wave scattering effects arising from the step in the ground surface between the toe and backfill of retaining walls (and embankment slopes) is known to cause reductions in the ground accelerations in the retained backfill. Figure 8 shows wave scattering reduction factors

published by Anderson et al, 2008 that were based on a finite element time history study of a uniform soil layer model located above a transmitting boundary subjected to the input motion. The scattering reduction factors are a function of the wall height and ground motion characteristics. They summarized their results in terms of Upper Bound (UB), Mid and Lower Bound (LB) spectral shapes. The LB spectral shape corresponded to a ground motion spectrum expected on weak rock or firm soil and the UB the spectrum expected on soft soil. Reductions from wave scattering increase as the wall height increases and as the spectral accelerations at long periods reduce. These reduction factors are applied to the peak ground acceleration (PGA) of the reference ground surface design spectrum to calculate the design force on the wall.

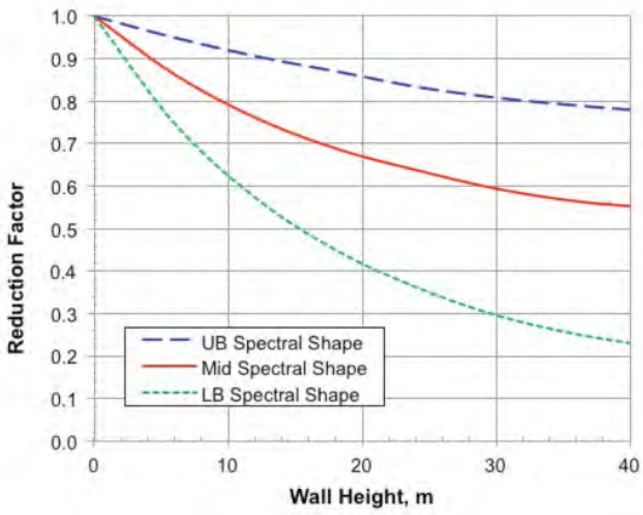

Figure 8: Wave scattering reduction factor to derive equivalent PGA.

Anderson et al, 2008 also point out that pseudo-static analyses treat the seismic coefficient as a constant horizontal static force applied to the soil mass whereas the peak earthquake load from a dynamic response analysis occurs for a very short time with the average dynamic force typically ranging from 30 to 70% of the peak value.

Taking into account the above considerations it would often be acceptable to design rigid walls for earthquake acceleration coefficients up to 20% less than recommended in the by NZSEE, 1980 and RRU, 1990 guidelines.

1.5 Castaic Power Station Application

The rigid wall elastic theory was originally developed by Wood, 1973 to provide design confirmation of the earthquake-induced soil pressures on a 29 m high soil retaining wall forming part of the main structure of the Castaic power-generating station in California. The Castaic power plant is a 1,250,000 kilowatt reversible-turbine hydro-electric facility located approxi·mately 75 km northwest of downtown Los Angeles and about 18 km from the San Andreas Fault. A typical cross-section of the power·house, which shows the extent of the soil retaining function of the upstream face of the structure, is reproduced in Figure 9.

Because of the rock foundation and the rigid nature of the structure, a good approximation for the earthquake pres·sures can be obtained by assuming rigid-wall behavior. The influence of the dynamics of the structure rocking on the foundation was investigated by Wood, 1973 but for the purpose of the following illustrative example the wall is assumed to be rigid.

Earthquake-induced pressures on the Castaic wall were computed by Wood, 1973 using the static finite element method, the normal mode finite element method and the exact analytical normal mode solution for the smooth rigid wall. In the finite element analyses both smooth and bonded wall contacts were used. In the application of the analytical solution the soil body was represented by an equivalent rectangle having an L/H = 1.67. The backfill soil was assumed to have a Young's modulus of 48 MPa, Poisson's ratio of 0.4 and a density of 1.92 t/m^3.

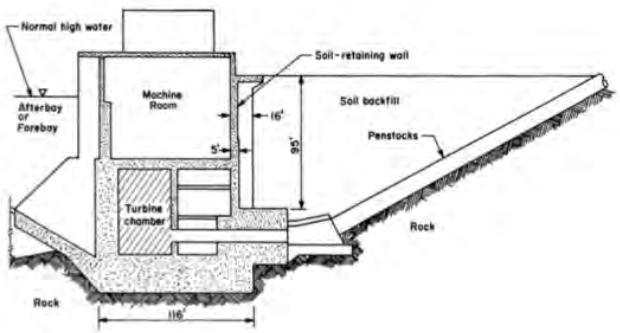

Figure 9: Typical section of Castaic power station. (Dimensions shown in feet. 95' = 29 m)

The static pressure distributions for a one-g horizontal body force and the significant static-one-g modal pressure contributions are plotted in Figure 10 for the analytical solutions (smooth rigid wall). The natural frequencies of the significant modes of vibration are given in Table 1.

Table 1: Natural frequencies analytical solution for L/H = 1.67.

Mode	Frequency, Hz	Period, sec
1,1	1.58	0.63
1,2	2.14	0.47
1,3	3.42	0.29

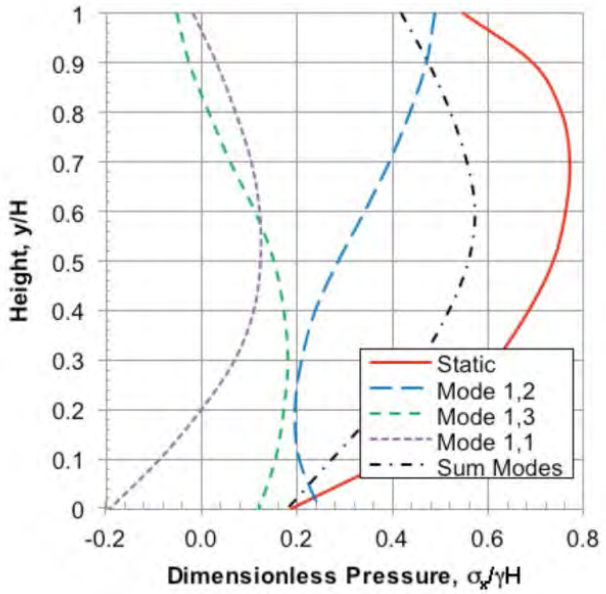

Figure 10: Modal and static response to static 1-g horizontal acceleration.

Maximum earthquake-induced pressure on the wall resulting from the recorded N-S and EW components of the 1940 El Centro earthquake were computed using time history and response spectrum methods assuming both 10% and 20% modal damping. The effects of the higher modes were included by using a "rigid" mode having a pressure distribution the difference between the static solution and the sum of the three modal contributions for both the smooth wall solutions. This "rigid" mode distribution was assumed to be subjected to the peak ground acceleration (PGA) of the input record (0.35 g for N-S component and 0.21 g for E-W component) in the modal analysis and the input acceleration in the time-history analysis. Spectra for the El Centro 1940 earthquake records are shown in Figure 11.

Plots of the pressure distributions from the time history analyses are compared in Figure 12 with the pressure distributions from the static horizontal body force

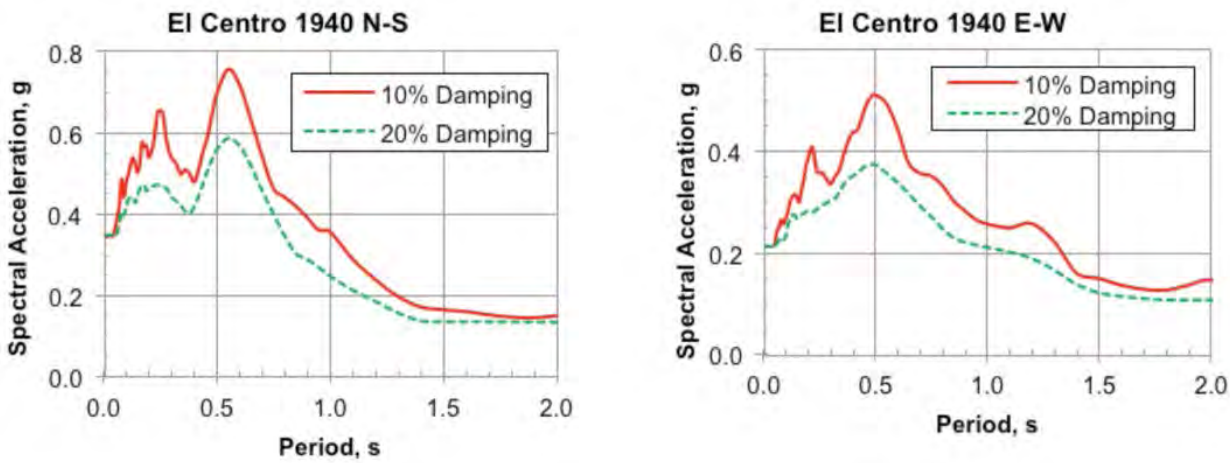

Figure 11: Response spectrum for El Centro, 1940 records

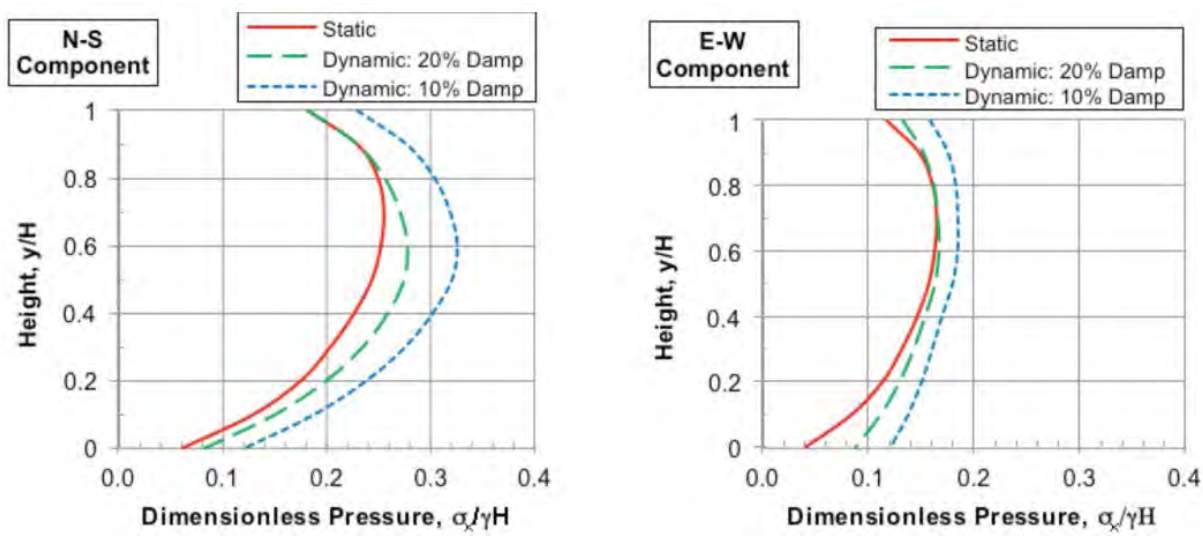

Figure 12: Dynamic time-history and static pressure distributions.

approximation (PGA accelerations). The static pressures agree reasonably closely with the dynamic pressures for 20% damping but the dynamic pressures are greater than the static pressures by typically 20% for 10% damping.

The maximum earthquake forces on the wall from the dynamic time-history, modal response spectrum and static horizontal body force methods are summarised in Table 2. The modal response spectrum force component have been added by taking the square root of the sum of the squares of the components (three modal and a rigid component). Ten percent damping was assumed in applying the modal response spectrum method.

Analysis Method	Dimensionless Force $F/\gamma H^2$	
	N-S Component	E-W Component
Static	0.21	0.14
Modal SRSS (10% damping)	0.23	0.14
Time-history. 10% damping	0.27	0.17
Time-history. 20% damping	0.23	0.15

Table 2: Wall forces from dynamic and static analyses

To assess the sensitivity of the wall force to the soil Young's modulus and damping, time-history analyses were carried out with the modulus both increased and reduced by a factor of two from the best estimate value, and for damping values ranging from 5% to 20%. Figure 13 shows the amplification of the peak dynamic force referenced to the static analysis force versus the damping value. The curves are based on the average amplification from the two earthquake input records (El Centro, 1940, N-S and E-W).

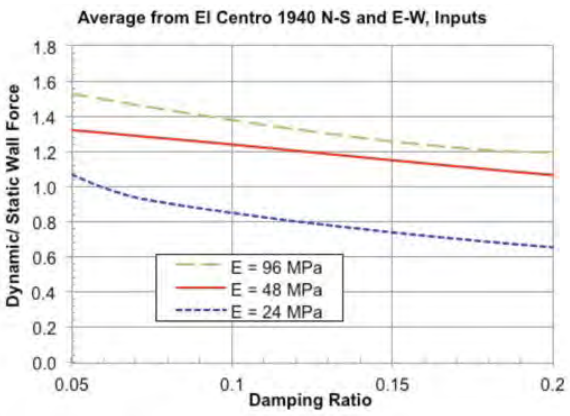

Figure 13: Dynamic force amplification versus damping for various modulus assumptions

For this particular example the dynamic force on the wall is quite sensitive to soil stiffness and damping assumptions with the dynamic amplification factor (dynamic/static force) varying from approximately 0.7 to 1.3 for a typical soil damping value in strong shaking of 15%. For the best estimate of the soil modulus the amplification factor at 15% damping was 1.15.

1.6 Design Recommendation

Many retaining walls are less than 10 m in height and rigid walls with heights less than 10 m will have periods of vibration of the backfill of less than 0.2 seconds. Reference to the spectra in Figure 11 indicates that significant amplification of dynamic forces will be unlikely for typical walls when the soil damping exceeds 10%. For typical rigid walls the static method will therefore give acceptable design forces.

Referring to Figure 3 and assuming moderate wave scattering effects a satisfactory design force acting on a rigid wall at a height of 0.5 H can be taken as a_{max} 0.8 γH^2 where a_{max} is the design PGA.

2. FLEXIBLE AND OUTWARD SLIDING WALLS

If under combined gravity and earthquake loads the top of the wall moves outwards (from either soil or structural deformations) by greater than 0.5% of the height of the wall a fully active pressure state will develop in the retained backfill. For this condition the M-O method provides a sufficiently accurate prediction of the earthquake pressures for most applications. The M-O equations are readily evaluated for typical wall and backfill geometries and for both cohesionless and cohesive soils.

2.1 M-O Active Pressure Coefficient

The M-O active earthquake pressure force P_{AE} on the back face (virtual or actual) of is given by:

$$P_{AE} = 0.5\, K_{AE}\, \gamma_s\, H^2 \qquad (6)$$

Where: K_{AE} is the active earthquake pressure coefficient, γ_s the unit weight of the soil and H the total vertical height of the soil on the back face (or virtual backface for a cantilever wall). K_{AE} is given by:

$$K_{AE} = \frac{\cos^2(\phi + \beta - \theta)}{\cos\theta \cos^2\beta \cos(\delta + \theta - \beta)\left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \theta - i)}{\cos(\delta + \theta - \beta)\cos(\beta + i)}}\right]^2} \qquad (7)$$

Where: ϕ is the peak soil friction angle; β the wall inclination (positive in a clockwise direction from the vertical); δ the mobilized interface friction angle assumed to act at the back of the wall; i the soil backslope angle (from horizontal); and θ the seismic inertia angle given by:

$$\theta = tan^{-1}\left[\frac{k_h}{1-k_v}\right]$$

kh is the horizontal acceleration coefficient and kv is the vertical acceleration coefficient. The angle parameters and the forces acting on the failure wedge behind the wall are shown in Figure 14.

Values of K_{AE} for soil friction angles of 30° and 35° and backslope angles varying from 0° to 25° are shown in Figure 15. Failure plane angles for the corresponding soil friction and backslope angles are shown in Figure 16.

2.2 Failure Plane Angle

At low failure angles the plane may pass through soils with higher strength parameters than the backfill soil. It is therefore important to calculate the failure plane angle for backfill soil and consider the influence of the surrounding soil outside the limits of the backfill zone. The failure plane angle α measured from the horizontal is given by:

$$\cot(\alpha - i) = -\tan(\phi + \delta + \beta - i) + \sec(\phi + \delta + \beta - i)\sqrt{\frac{\cos(\beta + \delta + \theta)\sin(\phi + \delta)}{\cos(\beta - i)\sin(\phi - \theta - i)}} \quad (8)$$

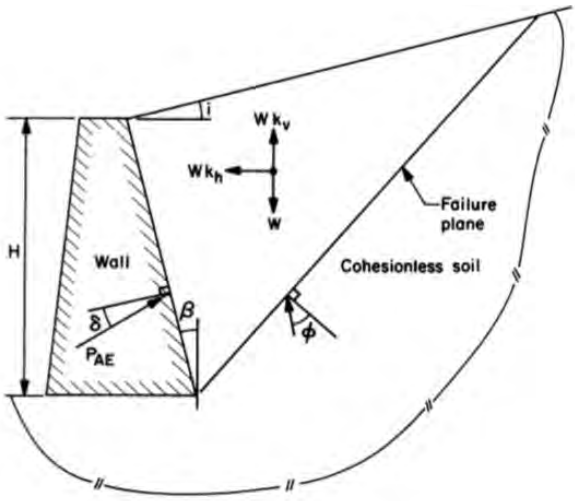

Figure 14: Definition diagram for Mononobe-Okabe analysis.

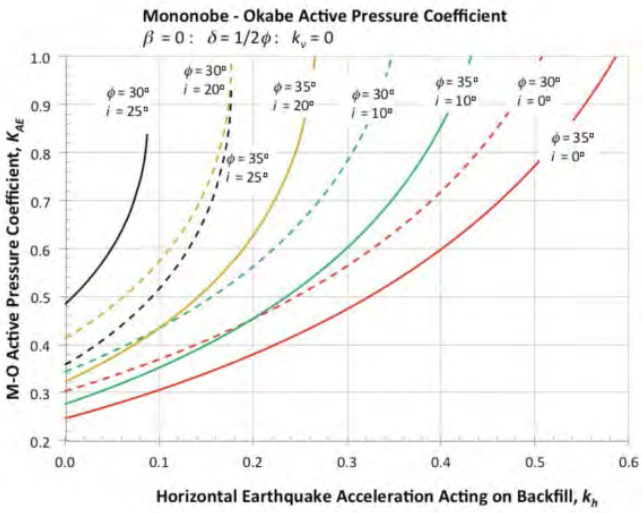

Figure 15: Mononobe-Okabe active pressure coefficient.

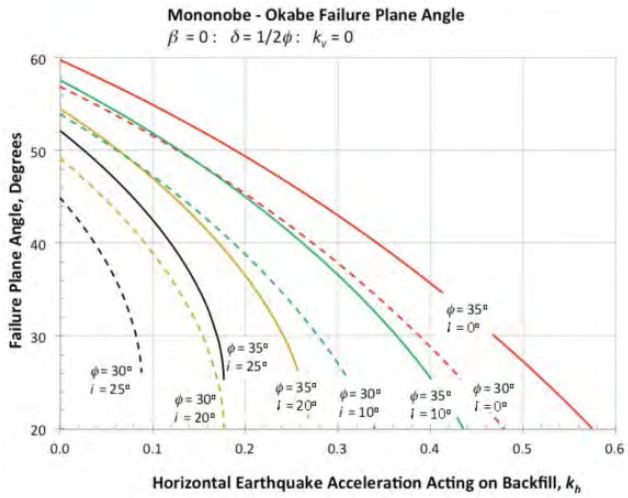

Figure 16: Monobe-Okabe failure plane angle for active pressure.

The value of K_{AE} for a selected failure plane angle and assuming the wall back face is vertical is given by:

$$K_{AE} = \frac{\cos(\alpha - \phi).(\tan(\alpha - \phi) + \tan\theta)}{\cos(\alpha - \phi - \delta).(\tan\alpha - \tan i)} \quad (9)$$

The equivalent equation for the case when the wall face has a slope angle can be found in Koseki et al, 1998.

Equation (9) should be used when the failure plane angle is predetermined by the boundary between the backfill and a surrounding soil with significantly greater shear strength than the backfill. This situation arises where the surrounding soil is weak rock or has significant cohesion.

Earthquake active pressure coefficients calculated using Equation (9) for a vertical wall, backfill friction angle = 30° and wall interface friction angle = 15° are plotted in Figure 17. Curves for k_h = 0.2 and 0.35 and several values of backfill slope angle, *i* are shown in the figure. These

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curves show that at low values of k_h and backfill slope angles, K_{AE} is not very sensitive to the failure plane angle. However, K_{AE} varies more significantly as both the backfill angle i, and the acceleration coefficient k_h increase. The peak K_{AE} values on each curve and the corresponding failure plane angle are the values obtained by evaluation of Equations (7) and (8). That is, for the case where there is no predetermined failure plane (similar shear strength for backfill and surrounding soil).

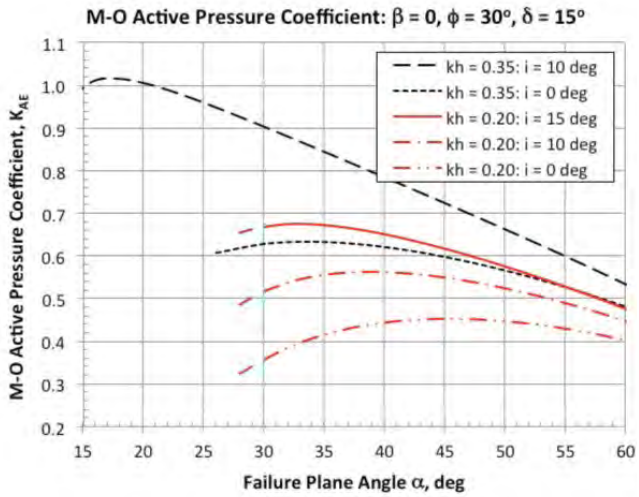

Figure 17. Variation of K_{AE} with failure plane angle (predetermined).

2.3 Failure on the Backfill Surface

When $\phi - \theta - i < 0$ Equation (7) does not have a solution because the term within the square root part of the denominator becomes negative. When subjected to horizontal earthquake acceleration the infinite slope shown in Figure 18 fails when $\theta = \phi - i$

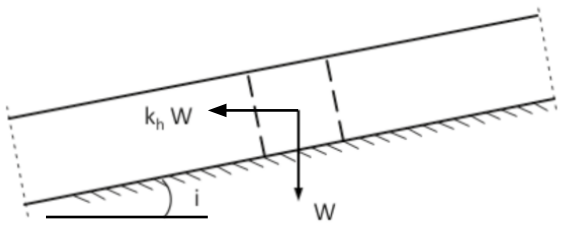

Figure 18. Infinite slope subjected to horizontal acceleration.

When the backfill slope i is $> \phi - \theta$ the slope is unstable. For $i = \phi - \theta$ the square root term in the denominator of the M-O K_{AE} equation is zero and K_{AE} reaches a maximum large value given by:

$$K_{AE} = \frac{\cos^2(\phi + \beta - \theta)}{\cos\theta \cos^2\beta \cos(\delta + \theta - \beta)} \quad (10)$$

Equation (10) is plotted in Figure 19 (for a vertical wall and $\delta = 0.5\,\phi$). For example, if $\phi = 35°$ and $i = 25°$, then $\theta = 10°$, $k_h = 0.176$ and for a wall-soil interface friction angle of 17.5° Figure 19 shows that at the point of insipient failure of the backfill slope the maximum value of $K_{AE} = 0.94$. At higher k_h values shallow failures will tend to form in the surface of the backfill. When this occurs it is unlikely that K_{AE} will increase significantly above its maximum value.

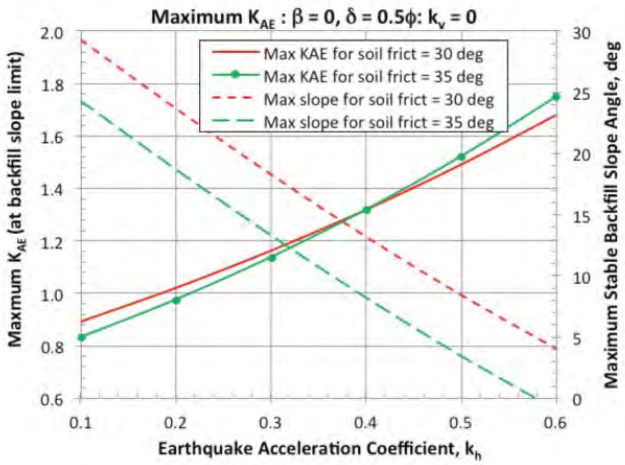

Figure 19. Maximum M-O pressure coefficients and backfill slope angles for $\phi = 30°$ and 35°.

Figure 19 indicates that the maximum K_{AE} values at the point of insipient backfill failure are not very sensitive to the soil friction angle and vary in an almost linear manner with the acceleration coefficient k_h. The variation of the maximum stable backfill angle with earthquake acceleration coefficient is also illustrated in the figure. The critical slope angle is readily calculated from $i = \phi - \tan^{-1} k_h$.

The maximum pressure coefficients vary from approximately 0.8 at $k_h = 0.1$ to 1.7 at $k_h = 0.6$. It may not be economical to design conventional walls for these high values. Tie-backs and the reduction of the acceleration coefficient resulting from outward movement should be considered for high accelerations and/or steep backfills.

2.4 Effects of Cohesion

Calculating M-O active pressure coefficients when the backfill has significant cohesion is straight forward. For a smooth vertical wall K_{AE} for a soil with both cohesion and friction is given by:

$$K_{AE} = \frac{k_h - \tan(\phi - \alpha)}{(\tan\alpha - \tan i)} + \frac{2c}{\gamma H}(\tan(\phi - \alpha) - \cot\alpha) \quad (11)$$

Where: c = the cohesion in the backfill and H is the height of the wall.

Equation (11) can be solved by spreadsheet iteration on the failure plane angle to give the maximum value K_{AE} for the particular wall geometry and backfill soil strength parameters.

The ratio of K_{AE} for a backfill soil with cohesion divided by K_{AE} for a soil with no cohesion is shown in Figure 20 for backfill friction angles of 30° and 35° and for two values of k_h. The wall height for this analysis was taken as 4 m. The results show that small amounts of cohesion reduce the K_{AE} values significantly with the reduction greater when the acceleration coefficient is relatively low. For medium height walls, backfill cohesion of 10 kPa reduces K_{AE} by a factor of 0.55 or greater. (The wall height is a parameter since the resistance from cohesion depends on the length of the failure plane.)

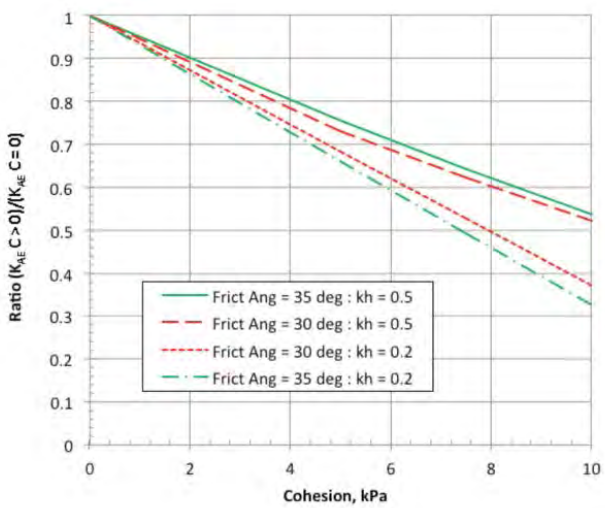

Figure 20. Reduction in K_{AE} from cohesion in backfill soil. (Wall height = 4 m.)

2.4 Application of Failure Plane Angle and Cohesion

Figure 21 illustrates the application of Equations (10) and (11) for a typical cantilever wall when the soil surrounding the backfill has significant cohesion. The analysis is for a 4 m high wall with a granular backfill having a friction angle of 30o and unit weight 20 kN/m^3 and subjected to a horizontal earthquake acceleration of k_h = 0.3. The first case analysed is when the failure plane is assumed to be on the interface between the backfill and surrounding soil that has an inclination angle of 60°. The second case is for the assumption of zero cohesion in the surrounding soil which is also assumed to have a friction angle ϕ = 30° and the third case is when the surrounding soil is assumed to have cohesion with c = 5 kPa and ϕ = 30°.

The results summarised in Table 2 show that if the surrounding soil has a small amount of cohesion the critical failure plane is likely to be on the backfill interface with the surrounding soil resulting in a pressure coefficient less than would be the case when the backfill and surrounding soil are both assumed to be cohesionless.

Analysis Case	Analysis Equation No	Cohesion in Surrounding Soil	Failure Plane Angle	K_{AE}
1	10	0	60° (interface)	0.51
2	7 & 8	0	42.60	0.57
3	11	5 kPa	46.50	0.41

Table 2: Example illustrating effects of cohesion in soil surrounding backfill

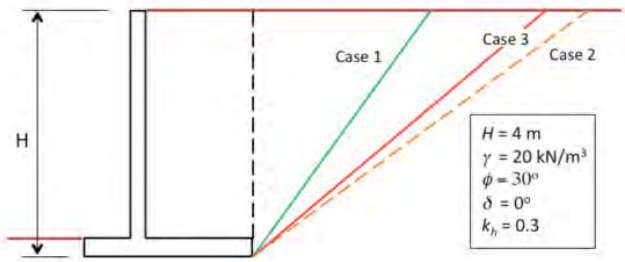

Figure 21: Example illustrating the effects of cohesion in the soil surrounding the backfill.

2.5 Comparison of M-O with other analysis methods

Comparisons of earthquake pressure coefficients and failure plane inclinations have been carried out for a number of typical concrete cantilever walls using the M-O equations and LimitState:GEO software. LimitState:GEO carries out limit analyses using a discontunity layout optimization technique (Smith and Cubrinovski, 2011).

The LimitState:GEO slip-line solution for one of these cantilever wall examples is shown in Figure 22. Details of the cantilever wall are shown in Figure 23. The backfill and surrounding soil were assumed to be cohesionless with a friction angle of 35° and the soil/wall interfaces were assumed to have a friction coefficient of tanϕ. The backfill slope was 15°. The analysis was for horizontal acceleration with the vertical acceleration assumed to be zero.

Figure 22: LimitState:GEO slip lines for cantilever wall example. (Wall details in Figure 23.)

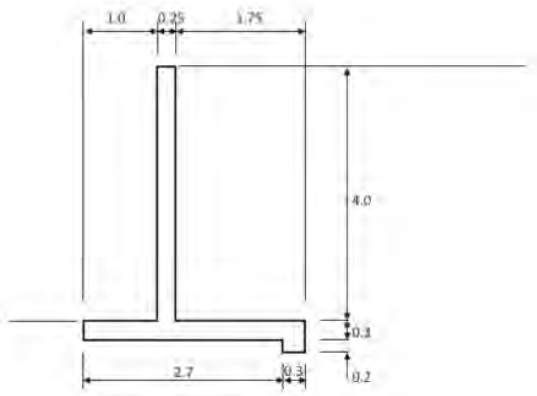

Figure 23: Wall details for example comparing M-O and LimitState:GEO solutions.

A comparison of the results from the two methods is given in Table 3.

Item	LimitState: Geo	M-O Analysis	Comment
Critical acceleration	0.27	0.28	M-O base friction = tanϕ
Failure plane angle	32.4°	32.5°	LimitState:GEO slip-line approximately linear
Bearing: capacity/demand	–	1.3	Indicates sliding rather than bearing failure

Table 3:. Comparison of M-O and LimitState:GEO results

The critical acceleration to initiate sliding in the M-O analysis is affected by the assumptions made regarding the base friction and passive resistance. In this example passive resistance was assumed to be effective on the depth of the foundation at the toe and the base friction coefficient was taken as tanϕ. These are the usual assumptions when a shear key is used on the footing. Bearing pressures and stem bending moments can be derived from both analysis methods. Again assumptions in the M-O analysis affect the results but reasonably good agreement was obtained for these outputs.

The application of limiting equilibrium log-spiral failure surface analysis equations to extend the M-O equations to unstable slopes and steeply sloping walls has been described by Leshchinsky et all, 2012. They compared their results and the results from research of others using similar limiting equilibrium analyses with the M-O results and concluded that for vertical walls all methods (M-O plus three different approaches) degenerate to the same planar failure surface. They suggest that the M-O is the simplest to apply for vertical walls. Even for moderately sloping walls the agreement between M-O and the other methods was sufficiently good for design purposes.

2.6 Outward Displacement

For high earthquake acceleration coefficients and walls with steep backfill slopes it will often be advantageous to design for permanent deformations arising from sliding of the wall and backfill wedge, soil foundation deformation or ductility in the wall structure. The design can then be based on the critical acceleration to initiate failure rather than the peak ground acceleration.

The outward movement of walls resulting from sliding, foundation bearing failures, or a ductile failure in the wall structure can be estimated using the *Newmark Sliding Block* theory. Outward movement can be conveniently estimated using the Jibson 2007 correlation equation. This was derived by statistical analysis of the displacements calculated from numerical sliding block computations using 2270 strong motion acceleration records. The Jibson equation for permanent outward displacement, d, expressed in centimetres is given by:

$$\log(d) = -0.271 + \log\left[\left(1-\frac{a_c}{a_{max}}\right)^{2.335}\left(\frac{a_c}{a_{max}}\right)^{-1.478}\right] + 0.424M_w \pm 0.454 \qquad (12)$$

Where: a_c is the critical acceleration to initiate sliding failure; a_{max} is the PGA in the acceleration record, and M_w the earthquake moment magnitude. The last term in the equation is the standard deviation of the model.

Evaluation of Equation (12) requires the calculation of the a_c / a_{max} ratio, selection of an appropriate earthquake magnitude and deciding an appropriate level for the probability of exceedance of the calculated displacement based on the standard deviation (0.454) given in Equation 12. The statistical probability factor in the equation can be conveniently evaluated using the NORMINV (probability, mean, standard dev) function in Excel.

Displacement vesus a_c / a_{max} from evaluation of Equation (12) for M_w = 7.0 and 16% probability of exceedance is compared with four other displacement correlation equations in Figure 24.

Background information of the correlation equations shown in Figure 24 is summarised in Table 4.

Reference	Magnitude Range Considered	No of Accelerograms Investigated	Source to Site Distance Included as Parameter
Ambraseys and Menu, 1988	6.9±0.3	52	No
Ambraseys and Srbulov, 1995	5.0 to 7.7	? (76 events)	Yes
Anderson et al, 2008	4.5 to 7.6	1800	No
Bray et al, 2010	5.5 to 7.6	688	No
Jibson, 2007	5.3 to 7.6	2270	No

Table 4: Correlation equations for estimating sliding block displacements

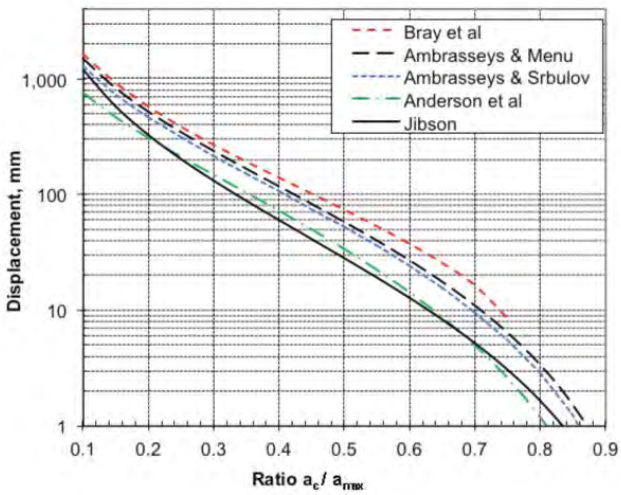

Figure 24: Outward displacements from correlation equations, Mw = 7.

Figure 24 shows that for a critical acceleration of 0.5 times the PGA the permanent outward movement of the sliding mass in a magnitude 7 earthquake is likely to be in the range of 30 to 70 mm (16% probability of exceedance). Many wall structures would not be seriously damaged by movements of this order.

The displacements calculated by the sliding block theory are strictly speaking at the centre of mass of the assumed rigid sliding block. Some rotation of a wall structure may occur when it fails by sliding and it is likely so that the displacements at the top of the wall could be up to 50% higher than estimated from the rigid siding block assumption.

3. CONCLUSIONS

Theoretical predictions based on the theory of elasticity for rigid walls subjected to static and dynamic cyclic loading have been verified by experimental investigations. Simple static solutions were found to adequate for design purposes. In many design applications there is unlikely to be dynamic amplification of the forces on the wall and wave scattering effects will reduce the forces predicted by the simplified theory. Forces approximately 20% less than given by the NZSEE and RRU recommendations should be satisfactory for most design applications.

For flexible walls and walls that slide outwards under combined gravity and earthquake-induced pressures the M-O method of analysis is a satisfactory design approach. The location of the failure plane should be investigated to determine whether it is located within the imported backfill material. Where the failure plane passes through soil outside the confines of the imported backfill adjustments should be made to the pressure coefficients to allow for potentially increased soil shear strength. Soil cohesion can result in very significant reductions to the M-O pressure coefficients.

For high horizontal accelerations and when there is significant slope of the backfill it will be difficult to design conventional walls to resist the predicted soil pressures. In these circumstances designing the wall to deform outwards either by sliding on the soil foundation or ductile deformations within the structure may be the best approach. Newmark Sliding Block theory can be applied to predict displacements when the failure acceleration is less than the design peak ground acceleration.

4. ACKNOWLEDGEMENTS

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GEOMECHANICS AWARD WINNER

DETERMINATION OF SITE PERIOD FOR NZS1170.5:2004

Tam Larkin[1] and Chris Van Houtte[2]

SUMMARY

The fundamental site period, *T*, is a key parameter for site classification in NZS 1170.5:2004. Many sites in New Zealand will fall into site classes C and D, where the boundary between the site classes is $T = 0.6$ seconds. NZS 1170.5 offers several methods of determining site classification. The intent of this paper is to expand on NZS 1170.5 and guide practising engineers towards more accurate and efficient methods for determining site period. We review methods to calculate the shear-wave velocity, then give specific examples for calculating the site period for five types of soil profile (uniform layer, shear-wave velocity increasing as a power of depth, shear modulus increasing linearly with depth, two-layer profile and three-layer profile). We find that NZS 1170.5 clause 3.1.3.7 for calculating site period at layered sites is unconservative and inconsistent with two other well-accepted methods for calculating site period. We consider the most accurate and efficient method of calculating site period for layered sites is to represent the profile as a lumped mass system, then calculate the fundamental frequency from the eigenvalues of the system. The successive application of the two-layer closed form solution is also considered an acceptable method.

INTRODUCTION

The New Zealand earthquake loadings Standard, NZS 1170.5:2004 [1], contains response spectra for structural design. The seismic loading on a specific structure will depend on, amongst other factors, the type of foundation soils where the structure is sited. Sites are categorised into five classes, A to E. The sites classes range from "rock sites" (Class A and B) to very soft soil sites, Class E.

The intent of the code is to classify the site according to its broad vibration properties as represented by the low amplitude fundamental site period, *T*. Empirical data and theoretical studies show conclusively that near surface materials have a significant impact on the surface motion; both the amplitude of the motion and the frequency content. The site period is determined by the geometry and nature of the geologic units present at the site. The site period used in NZS 1170.5 is independent of the strength of earthquake design motion, because the low amplitude period assumes very low strain response to represent the strain-independent soil properties.

Figure 1 shows the spectral shape factors for each of the site classes. There are significant differences in the shape factor between the various classes, illustrating the importance of assessing the site class. Class A, strong rock, and Class B, rock, were not found to be significantly different in the hazard study carried out for NZS 1170.5 and hence they are grouped together. It is clearly important to distinguish between Class C, Class D and Class E. The maximum shape factor (at short periods) is the same in the case of Class D and Class E but the plateau extends to 0.6 seconds in the case of Class D and 1.0 seconds in the case of Class E. Class C is important since many sites in New Zealand will fall in this category. For sites to fall into this class, the site period needs to be less than 0.6 seconds or to have depths of soil not in excess of those listed in Table 3.2 (page 14 of the Standard), which is reproduced here in **Table 1**.

NZS 1170.5 clause 3.1.3.1 specifies a hierarchy of methods to assess the site class. The stated hierarchy in order of preference is:

1. From the site period based on four times the travel time of shear waves from the underlying rock to the ground surface.
2=. From borelogs, including measurement of geotechnical properties.
2=. From a method known as Nakamura ratios.
2=. From recorded earthquake motions.
5. From borehole descriptors but with no measurement of geotechnical properties.
6. From surface geology and estimates of the depth to rock.

Most (if not all) sites are heterogeneous, i.e. they contain a number of soil layers of differing properties. Each layer will influence the site period. For these cases, there is another clause in NZS 1170.5 (clause 3.1.3.7) for determining the site period for layered sites. This clause states that the natural period of the site may be estimated by summing the contributions to the natural period of each layer. The contribution of each layer is defined by multiplying 0.6 seconds (the boundary between class C and class D) by the ratio of the layer's thickness to the maximum soil depths in Table 3.2. It appears the intent of this clause is for determination of site period without calculating the shear wave velocity.

Benefits in true cost and safety are likely to arise from using accurate methods to assess the site period and this is the motivation for this work. The intent of this article is to advise

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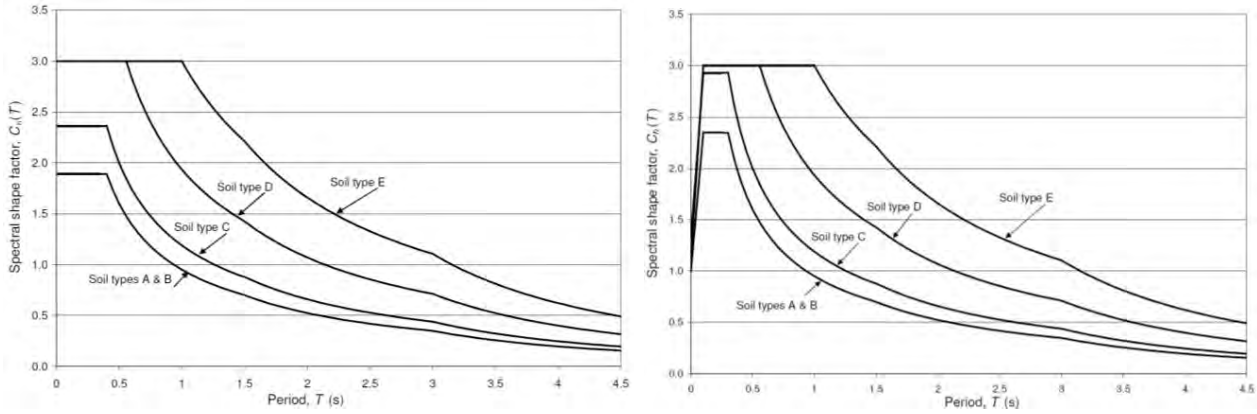

Figure 1: NZS 1170.5 spectral shape factors for general cases (left) and for modal response and numerical integration time-history methods (right).

Table 1. Reproduction of NZS 1170.5 "Table 3.2 – maximum depth limits for site subsoil class C".

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

the earthquake engineering community on the methods to assess site class, with particular focus on the specified hierarchy of methods detailed in NZS 1170.5. We begin with a background on several available methods to calculate shear-wave velocity (V_S) at a site, as this is a key parameter for accurate determination of site period. We subsequently present five example soil profiles (uniform layer, V_S increasing as a power of depth, shear modulus linearly increasing with depth, a two-layer profile and a three-layer profile) and discuss methods for calculating site period in each case. For the two-layer and three-layer cases, we compare NZS1170.5 clause 3.1.3.7 with conventional methods for determining site period.

BACKGROUND: DETERMINING SHEAR-WAVE VELOCITY

In an engineering context, the most important mode of seismic response is that associated with shear waves. These waves cause horizontal motion, and hence horizontal shear deformation of the foundation soil. Most of the damage to infrastructure originates from foundation response to shear waves. Of particular relevance to the topic of site period is the velocity of propagation of these shear waves, V_s, through the soil. A linear elastic analysis based on dynamic equilibrium shows that this velocity is related to the shear modulus through equation (1):

$$V_S = \sqrt{\frac{G_{\max}}{\rho}} \quad \text{m/s}, \tag{1}$$

where G_{max} is the tangential shear modulus at zero shear strain (MN/m^2) and ρ is the mass density (kg/m^3). Frequent use is made of this relationship in geotechnical earthquake engineering.

In situ measurements

The shear wave velocity of a soil may be measured in the laboratory or the field. In a general sense the "best" method is *in situ* measurement since the effective stress is correct and there is no sample disturbance. While in principle the methods are simple there are many complicating factors. What is presented here is a general overview. There are four main methods of *in situ* measurement:

(i) Downhole measurement, where the shear waves (SH) are created at the surface and the travel time to various positions at depth in a borehole are measured. A bi-directional source is useful to enable the shear waves to be readily separated from the compression waves. A relatively recent development is the use of a seismic cone penetrometer to carry out shear wave velocity measurements. A conventional cone penetration (CPT) device is used that has an accelerometer or geophone incorporated in the cone. In this case there is no need for a borehole, the CPT data is interrupted at the depth required and a shear wave velocity test carried out. Some site investigation companies now have such a device and this method represents an effective method of assessing the shear wave velocity profile at a site. As with all CPT work a calibration borehole is recommended.

(ii) Crosshole measurement, where shear waves are propagated between adjacent boreholes and the travel time measured. These tests usually invoke vertical particle displacement, i.e. SV waves. These tests are often used for investigations beneath existing foundations or for the purpose of machine foundations.

(iii) Geophysical refraction and reflection surveys, where a shear source generates shear-waves, and the subsequent arrival times of shear-waves are detected by a line of horizontal geophones. Difficulties arise in generating a large enough shear source to be clearly recorded by the geophones.

(iv) SASW / MASW – Spectral analysis of surface waves, or multi-channel analysis of surface waves, are geophysical methods which utilise the dispersive properties of surface waves (typically Rayleigh waves generated by a sledgehammer hitting a plate) to calculate dispersion

curves (phase velocity vs frequency plots). A 1D or 2D shear-wave velocity profile is obtained by inverting (i.e. back-calculating from) the dispersion curve. This technique was used very infrequently in New Zealand prior to the 2010-2011 Canterbury earthquake sequence but is becoming more popular. The data processing does require some specialist knowledge and therefore these methods should only be conducted by experienced personnel.

Of the four methods, downhole measurements are only representative of shear-wave velocity at a single point, while the other three methods represent an average value over a 2D line, which may be more beneficial at many sites. However, if the sites are more complex with 2D or 3D variations, the refraction, reflection and surface wave methods become very difficult. It is also considered best practice to use multiple methods at the same site, to validate the results and quantify uncertainties.

Laboratory measurements

Laboratory methods usually employ what are known as bender elements. These are wafers of a piezo-ceramic material about 6 mm long that generate a small electrical current on flexing. A bender element is installed in the top and bottom of a triaxial specimen, one element being the source the other being the pick-up. A small current is supplied to the source that causes flexure of the bender element with the consequent production of shear waves. These waves travel through the specimen and create flexure of the pick-up with consequent generation of a small current. By using an oscilloscope the travel time between the source and pick-up is measured.

Other laboratory devices, such as a Torsional Resonant Column, are used to measure the shear modulus. Such laboratory studies are usually very detailed and seek data that allows evaluation of the shear modulus as a function of shear strain. Laboratory tests suffer from specimen disturbance but are used when there will be changes in the effective stress at a site since this will produce a change in the shear wave velocity.

Empirical correlations

The most reliable method of assessing V_S is from site specific *in situ* measurement. If this data is not available, then empirical methods may be used to furnish an estimate for V_S or G_{max} based on *in situ* test results e.g. $(N_1)_{60}$ or q_c values, where $(N_1)_{60}$ is the normalised SPT value at 60% energy efficiency and q_c is the cone resistance from a CPT test. The following are suggestions from the literature for an initial estimate of G_{max} or V_S. It needs to be kept in mind that almost none of the data from which the empirical relationships were estimated were derived from New Zealand soils. When using these relationships it is important to consult the reference to understand the geological/geotechnical setting of the data.

Cohesive soil deposits

As an approximate method, the shear modulus is correlated with the undrained shear strength i.e. the ratio of G_{max} / s_u. From the limited data available for a residual New Zealand soil, Meyer (1999) [2] suggested an appropriate value of G_{max} / s_u is approximately 500. V_S may then be calculated from:

$$V_S = \sqrt{\frac{500 \cdot s_u}{\rho}} \quad \text{m / s}, \qquad (2)$$

Table 2. Values of G_{max} / s_u from Weiler (1988).

Plasticity Index	Overconsolidation Ratio (OCR) 1	2	5
15-20	1100	900	600
20-25	700	600	500
35-45	450	380	300

Based on values of s_u obtained using triaxial compression, Values of G_{max} / s_u as a function of over consolidation ratio and plasticity index have been suggested by Weiler (1988) [3], shown in **Table 2**.

Based on a wide ranging series of field tests, Mayne and Rix (1993) [4] have suggested the following relationship:

$$G_{\max} = 406 \cdot q_c^{0.695} \cdot e^{-1.13} \qquad (3)$$

where e is the void ratio, and both G_{max} and the cone tip resistance, q_c, are in kPa.

Cohesionless soils

There are a number of correlations of shear wave velocity (or G_{max}) with either SPT or CPT values. Most of this data relates to sedimentary soils from overseas. There are some data for New Zealand pumice soil from triaxial testing. Richart *et al* (1970) give further relationships for G_{max} for silica sand as a function of confining stress and void ratio [5]. There is great uncertainty about whether overseas correlations involving q_c and N obtained on quartz sands (all those below except (vi)) can be applied to New Zealand pumice soil. At this point these correlations should not be applied to New Zealand pumice soils.

(i) Based on CPT field tests in Italy on uncemented silica sands, Baldi *et al*. (1989) [6] propose the correlation shown in **Figure 2**.

(ii) Rix and Stokoe (1991) [7] have proposed the following relationship, where all variables are in kPa:

$$G_{\max} = 1634 \cdot q_c^{0.25} \cdot \left(\sigma_v^{'}\right)^{0.375} \qquad (4)$$

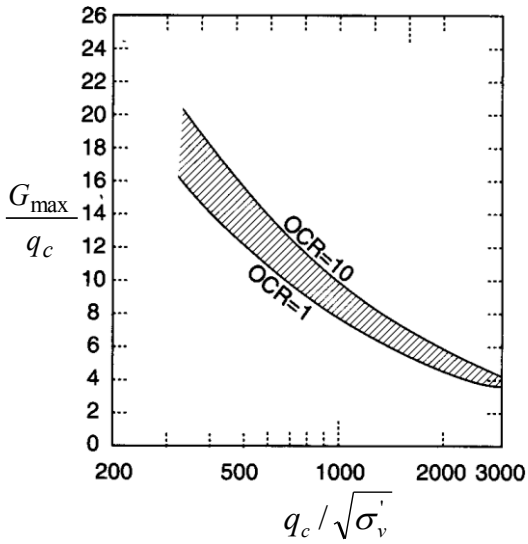

Figure 2: Shear modulus from CPT data (Baldi et al., 1989 [6]).

$$G_{\max} = 138{,}900 \cdot (N_1)_{60}^{0.33} \cdot (\sigma_m')^{0.5} \quad , \qquad (5)$$

where the stress units are Pa and σ_m' is the mean effective stress, given by:

$$\sigma_m' = 0.33(\sigma_v' + 2\sigma_h') \quad . \qquad (6)$$

(iv) Sykora and Stokoe (1983) [9] propose:

$$V_S = 107 \cdot N_{60}^{0.27} \quad \text{m/s} \quad . \qquad (7)$$

Note that the N value is not normalised.

(v) Imai and Tonouchi (1982) [10] propose:

$$V_S = 107 \cdot N_{60}^{0.315} \quad \text{m/s} \quad . \qquad (8)$$

Note that the N value is not normalised.

(vi) Marks *et al*. (1998) [11] proposed a relationship for New Zealand pumice sand:

$$G_{\max} = (14 \cdot D_R^{0.3}) \cdot \left(\frac{p_0'}{p_a}\right)^{0.6} \quad \text{MPa} \, , \qquad (9)$$

where D_R is the relative density, p_0' is the effective confining pressure of a triaxial specimen and p_a is atmospheric pressure. The determination of relative density is problematic for pumice soils, the reference should be consulted before using this relationship.

Finally, a Japanese study (1978) [12] presents the results of a large number of *in situ* wave velocity measurements correlated with N values, soil type, depth and geological age. Interested readers will find information on the background to the study within the reference.

CALCULATION OF SITE PERIOD

The term site period refers to the fundamental period of vibration of a horizontal site of linear elastic material when responding to vertically propagating shear waves with horizontal particle motion, known as SH waves. In the context of NZS 1170.5, site period is a key parameter because it defines the boundary between class C and class D (T = 0.6 s). This threshold is critical, given that the design loads change significantly either side of the boundary. To mitigate the large increase in forces at T = 0.6 s, McVerry (2011) [13] proposed intermediate spectra between the existing class C and class D spectra, which are defined entirely in terms of site period. Robust calculations of site period can justify the use of these alternate spectra for calculating earthquake loadings.

For NZS 1170.5 the site is assumed to be one dimensional, i.e. the lateral boundaries are far removed and have no influence on the motion of the site. The properties needed for calculation of the site period are the mass density, ρ, the thickness of the layer, H, and the shear modulus, G_{max}, or shear wave velocity V_S.

There are an infinite number of modal frequencies and mode shapes, the fundamental mode being the lowest value of ω, i.e. ω_1 . The fundamental period, T, may be calculated from ω by

$$T = \frac{2\pi}{\omega_1} \quad \text{seconds} \quad . \qquad (10)$$

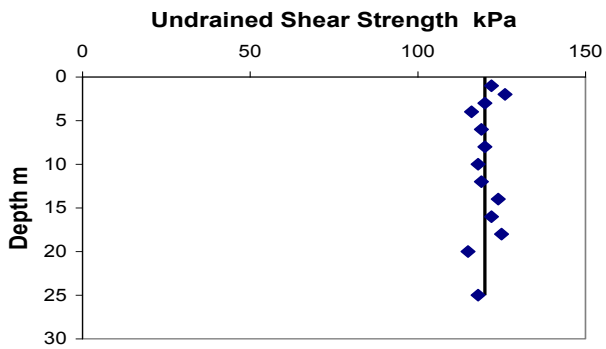

Figure 3: Variation of s_u with depth.

If the mass density is considered constant with depth (within an engineering approximation) then the only property needed to calculate T is the distribution of G_{max} or V_S with depth. For some simple distributions of V_S with depth, closed form solutions are available, while for more complex layered site numerical solutions prevail. For more information, Dobry *et al*. (1976) [14] provide a summary of the closed form methods.

To assist site period calculations for a variety of soil profile types, we now present five worked example calculations.

(i) Case 1: uniform layer

For this simple case, the shear wave velocity and mass density are constant with depth, and the lateral boundaries are assumed to be infinite in both planes. From the closed form solution, the first modal frequency is

$$\omega_1 = \frac{\pi V_s}{2H} \quad \text{rad/s} \quad , \qquad (11)$$

from which the fundamental period is

$$T = \frac{2\pi}{\omega} = \frac{4H}{V_s} \quad \text{sec} \qquad (12)$$

The first mode shape is $X(z) = \cos(\frac{\omega z}{V_S})$.

Note that equation (12) is only applicable for sites with constant mass density, ρ, and exactly represents the preferred method in the NZS 1170.5 clause 3.1.3.1 hierarchy for determining site period.

Example

Consider a uniform 25 m layer of saturated clay soil with OCR = 5, PI = 25, ρ = 1950 kg/m^3. A profile of the measured undrained shear strength, s_u, is shown in **Figure 3**. Weiler (1988) [3], gives G_{max} / s_u = 500 and G_{max} = 6 x 10^4 kPa (see Table 2). Applying equation (1):

$$V_S = \sqrt{\frac{1000 \times 6 \times 10^4}{1950}} = 175 \text{ m/s and}$$

$$T = \frac{4H}{V_S} = 0.57 \text{ sec} \quad .$$

Fundamental period = 0.57 s, therefore class C.

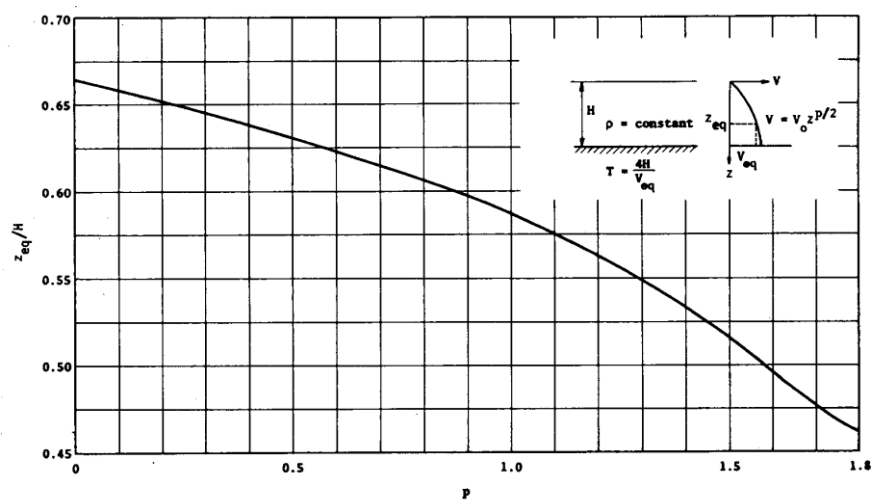

Figure 4: z_{eq} / H as a function of p, the closed form solution for equation (14), from Dobry et al. (1976) [14].

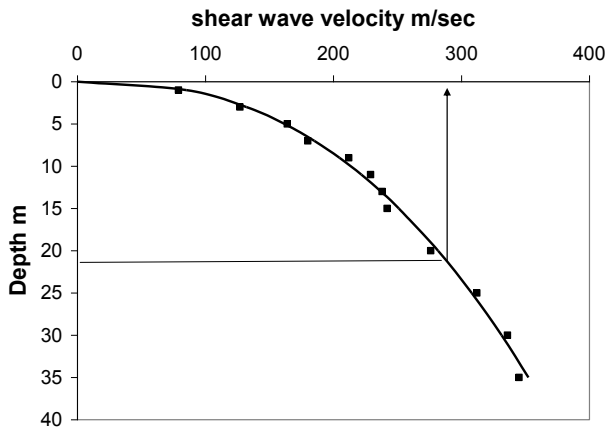

Figure 5: Example V_S profile for the case of velocity increasing as a power of depth. Solid black line is Eq (20) fitted by eye.

(ii) Case 2: velocity increasing as a power of depth

This case has been reported by Idriss and Seed (1968) [15] and Dobry *et al*. (1971) [16], and may be representative of uniform normally consolidated saturated clay deposits or uniform deposits of sand (water table at ground surface). The preferred method in the NZS 1170.5 clause 3.1.3.1 hierarchy can still be applied in this case, using the following closed form solution. The distribution of the shear-wave velocity is taken to be

$$V_S = V_{S0}\, z^{\frac{p}{2}} \quad . \tag{13}$$

Values of p are usually taken to be between 0.5 and 1. The fundamental period is given by

$$T = \frac{4\pi H^{(2-p/2)}}{(2-p)V_{S0}\, q} \quad \text{for} \quad 0 \le p < 2 \ , \tag{14}$$

where q is the first root of $J_n(q) = 0$, J_n is the Bessel function of order $n = (p-1)/(2-p)$. The solution for the period for this case may be found from the equation for a uniform layer

$$T = \frac{4H}{V_{S\,eq}} \tag{15}$$

where $V_{S\,eq}$ is the value of V_s at the "equivalent depth" z_{eq} . **Figure 4** is a graphic representation of the closed form solution, giving the value of z_{eq} / H as a function of p, which may then be used to solve for T.

Example:

Consider a 35 m layer of sand with the water table at the surface. **Figure 5** shows hypothetical results of SPT tests, converted to V_S e.g. by using equation (7). The results are fitted with equation (13) by eye, with p = 0.8 and V_{S0} = 85 m/s. Figure 4 with p = 0.8 gives z_{eq} / H = 0.608, thus z_{eq} = 21.3 m. Comparing with Figure 6, this gives $V_{S\,eq}$ = 290 m/s.

Entering in this value into equation (15) gives a fundamental period T = 0.48 seconds, therefore class C.

(iii) Case 3: Shear modulus increasing linearly with depth

This case for constant ρ has been presented by Ambraseys (1959) [17] for G_{max} increasing with depth, and by Urzua (1974) [18] for G_{max} decreasing with depth. For both of these cases, the site period can still be obtained using the preferred method in the NZS 1170.5 clause 3.1.3.1 hierarchy, using the closed form solution detailed here. If G_0 and G_H are the shear modulus at the surface and base of the layer respectively then the variable K is used where K is

$$K = \sqrt{\frac{G_{\max 0}}{G_{\max H}}} \quad , \tag{16}$$

and

$$\frac{G_{\max}}{G_{\max H}} = K^2 + \left(\frac{z(1-K^2)}{H}\right) \quad . \tag{17}$$

When K < 1 the modulus increases with depth, and when K > 1 the modulus decreases with depth. The fundamental period is given by

$$T = \frac{4\pi HK}{a_1(1-K^2)V_{S0}} \quad 0 \le K < 1 \ ,$$

$$T = \frac{4\pi HK}{a_1(K^2-1)V_{S0}} \quad K > 1 \ , \tag{18}$$

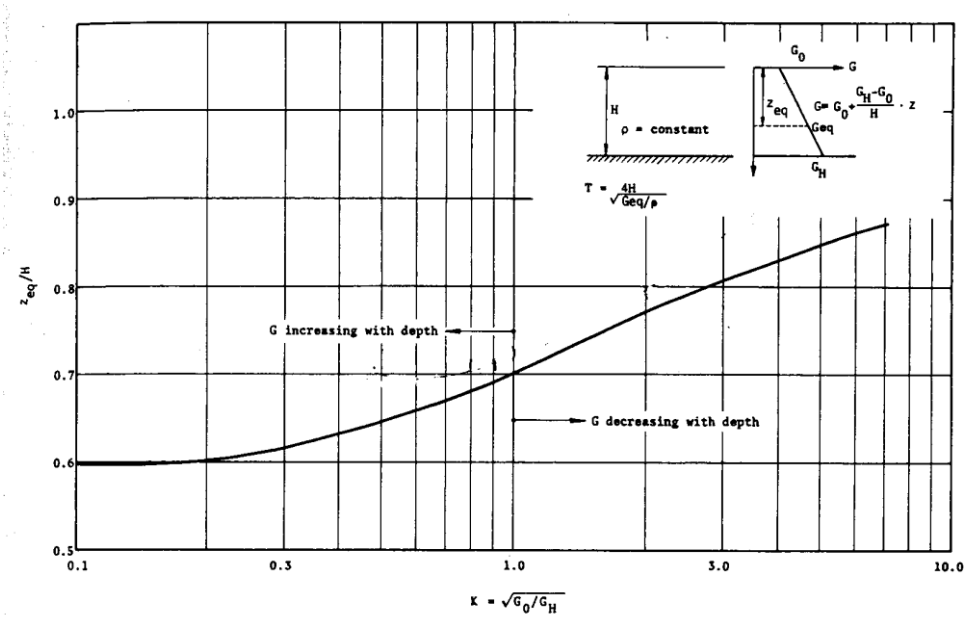

Figure 6: ***A graphical representation of the closed form solution to the shear modulus increasing linearly with depth, z_{eq} / H as a function of K, from Dobry et al. (1976) [14].***

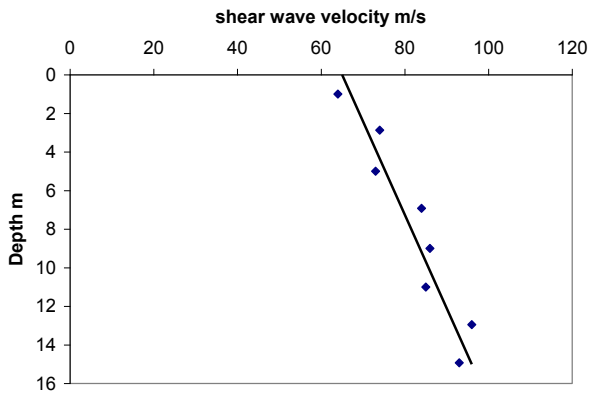

Figure 7: ***Example V_S profile for case of linearly increasing shear modulus.***

where V_{S0} is the wave velocity at the free surface and a_1 is the first root of

$$J_0(a_1)Y_1(Ka_1) - J_1(Ka_1)Y_0(a_1) = 0 \ , \qquad (19)$$

where $J_0(\,)$ and $J_1(\,)$ are Bessel functions of the first kind and Y_0 and Y_1 are Weber's Bessel functions. In this case also the period may be expressed using the equation for a uniform layer, $T = 4H / V_{S\,eq}$ where $V_{S\,eq}$ is the wave velocity at depth z_{eq} and the expression for z_{eq} for all K is

$$\frac{z_{eq}}{H} = \left(\frac{a_1}{H}\right)^2 (1-K^2) - \frac{K^2}{1-K^2} \ . \qquad (20)$$

Figure 6 is a graphical representation of the above closed form solution. This may be used to determine z_{eq} / H as a function of K. The value G at z_{eq}, G_{eq}, may then be found from equation (20) and used to evaluate $V_{s\,eq}$. The period is then calculated using $T = 4H / V_{s\,eq}$, equation (15), as in the previous case.

Example:

Consider a 15 m layer of soft normally consolidated clay with a mass density of 1,720 kg/m^3. A series of seismic cone tests have produced the measured shear wave velocity profile shown in **Figure 7**. The shear wave velocity at the ground surface, V_{S0} = 65 m/s. The shear wave velocity at the rock line, V_{SH} = 96 m/s.

Thus $G_0 = V_{S0}^2 \rho = 7.27$ MPa

$$G_H = V_{SH}^2 \rho = 15.85 \text{ MPa}$$

$$K = \left(\frac{G_0}{G_H}\right)^{0.5} = 0.68$$

$\frac{z_{eq}}{H} = 0.67$ from Figure 6; hence $z_{eq} = 10.05$ m

$$G_{zeq} = \left(G_0 + \frac{(G_H - G_0)z_{eq}}{H}\right) = 13.06 \text{ MPa}$$

$$V_{S\,eq} = \left(\frac{G_{zeq}}{\rho}\right)^{0.5} = 87.1 \text{ m/s}$$

$$T = \frac{4H}{V_{S\,eq}} = 0.69 \text{ s.}$$

Therefore class D.

(iv) Case 4: Two layer profile

The solution for calculating site period at a two layer profile is more complex, as continuity of shear stress and displacement at the interface of layers needs to be enforced. NZS 1170.5 specifies a different method for calculating site period for layered sites, by summing the contributions of each layer to

the overall site period. Here we compare two alternative methods with the method detailed in NZS 1170.5 clause 3.1.3.7, and show that the alternative methods give more precise results.

Closed form solution

Solutions for the two layer profile were presented by Madera (1971) [19], Chen (1971) [20] and Urzua (1974) [18]. The key variables in these solutions are:

$$\frac{\rho_A H_A}{\rho_B H_B} \quad ,$$

and the ratio of the fundamental periods of the lower and upper layers (T_B / T_A). **Figure 8** allows calculation of the period of the compound system as a function of the period of the upper layer.

Consider the two-layer site profile shown in **Figure 9**. For the clay layer, Weiler (1988) [3] gives G_{max} / s_u = 600, and therefore G_{max}= 4.2 x 10^7 Pa. This results in:

$$V_{S\,clay} = \left(\frac{G_{\max clay}}{\rho_{clay}}\right)^{0.5} = 152.7 \text{ m/s}$$

$$T_{clay} = \frac{4H_{clay}}{V_{S\,clay}} = 0.21 \text{ s} \quad \text{from equation (12).}$$

For the sand layer, use Sykora and Stokoe (1983) [9]. As the layer is approximately uniform, average the N values, thus $\overline{N}$ = 8.9.

$$V_{S\,sand} = 107\overline{N}^{0.27} = 193 \text{ m/s , from equation (7)}$$

$$T_{sand} = \frac{4H_{sand}}{V_{S\,sand}} = 0.25 \text{ s}$$

$$\frac{T_{sand}}{T_{clay}} = 1.19$$

$$\frac{\rho_{clay} H_{clay}}{\rho_{sand} H_{sand}} = 0.65$$

$$\frac{T}{T_{clay}} = 2.1 \text{ (from Figure 8)}$$

$$T = 2.1T_{clay} = 0.44 \text{ s .}$$

Therefore the fundamental period from this approach is **0.44 seconds**, which corresponds to a site class C.

Lumped mass solution

The second method we discuss is known as the lumped mass solution. In this approach, the soil profile may be idealised as a series of masses interconnected by shear springs. The mass is calculated to represent the surrounding soil and the stiffness of the shear spring is computed from the shear modulus. This is sometimes referred to as a 1D shear beam model. **Figure 10** shows the system. Generally the thickness of the sub-layers needs to be approximately 3 m or less for good accuracy. The dynamic equation of motion of the system under free vibration includes the mass matrix, *[M]* and the stiffness matrix *[K]* and may be written as

$$[M]\langle \ddot{x} \rangle + [K]\langle x \rangle = 0 \quad , \tag{21}$$

where $\langle \ddot{x} \rangle$ *and* $\langle x \rangle$ are the vectors of acceleration and displacement of each mass relative to the base. They are of dimension n, where n is the number of masses. Equation (21) may be transposed into the classic eigenvalue form.

$$[A] - \omega^2 [I] = 0 \quad , \tag{22}$$

where $[A] = [M]^{-1}[K]$, $[I]$ is the identity matrix and ω are the fundamental frequencies of the system. Since the system is closely coupled, the *[K]* matrix is tri-diagonal and symmetric and has the form:

$$K = \begin{matrix} k_1 + k_2 & -k_2 & 0 & 0 \\ -k_2 & k_2 + k_3 & -k_3 & 0 \\ 0 & -k_3 & k_3 + k_4 & -k_4 \\ 0 & 0 & -k_4 & k_5 \end{matrix} \tag{23}$$

The mass matrix is diagonal:

$$M = \begin{matrix} m_1 & 0 & 0 & 0 \\ 0 & m_2 & 0 & 0 \\ 0 & 0 & m_3 & 0 \\ 0 & 0 & 0 & m_4 \end{matrix} \tag{24}$$

The values of the fundamental frequencies can be easily solved using software e.g. SAP, MATHCAD or MATLAB, with the smallest eigenvalue (ω_1) of the [A] matrix corresponding to the fundamental period of the site, i.e. equation (16):

$$T = \frac{2\pi}{\omega_1} \quad .$$

The method is useful especially in the case of relatively thin soft layers of soil contained within a soil profile and where the profile contains layers where the shear wave velocity is a function of depth. Each soil layer is subdivided into sublayers, of thickness h_i, to represent what is actually a continuum. The magnitude of each mass is calculated to represent the soil over one half the sublayer on each side, i.e.

$$m_i = 0.5(\gamma_i h_i + \gamma_{i+1} h_{i+1}) \quad \text{kg} \tag{25}$$

where γ is the unit weight and h_i is the inter-mass distance. The lumped mass system represents a unit plan area since a one dimensional model is employed and thus the shear stiffness of the interconnecting spring is

$$k_i = \frac{(G_{\max})_i}{h_i} \quad \text{N/m} \quad , \tag{26}$$

Taking a finer subdivision of the profile can prove that the solution is converging.

An application of this method to the two-layer profile in Figure 10 is shown here. The mass and stiffness matrices may be formed by compiling the parameters **Table 3**. The soil profile has been discretised into sublayers, with both the sand and clay layers having four sublayers each. Note that the first sublayer is adjacent to the rock interface.

The [M] and [K] matrices are formed according to equations (30) and (31), although in this case both are 8 x 8 matrices. To obtain the eigenvalues, the [A] matrix is calculated by

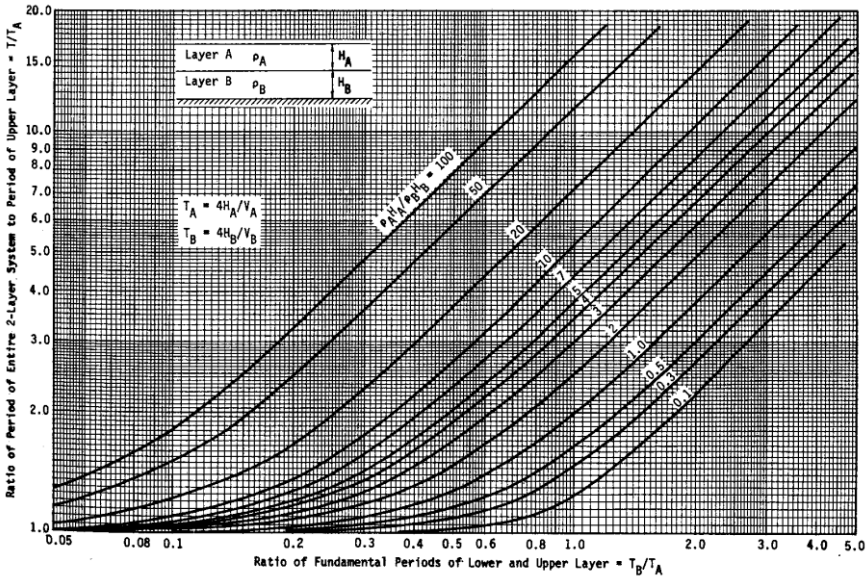

Figure 8: Closed form solution for a two-layer profile, from Dobry et al. (1976) [14].

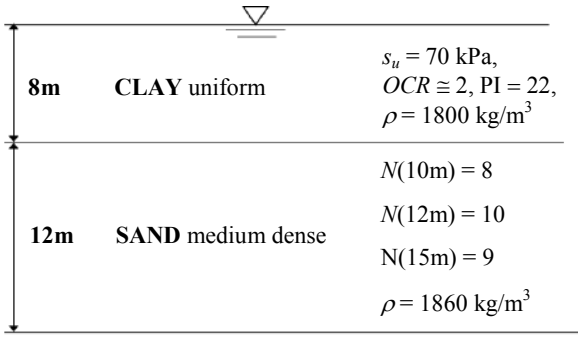

Figure 9: Example two-layer profile.

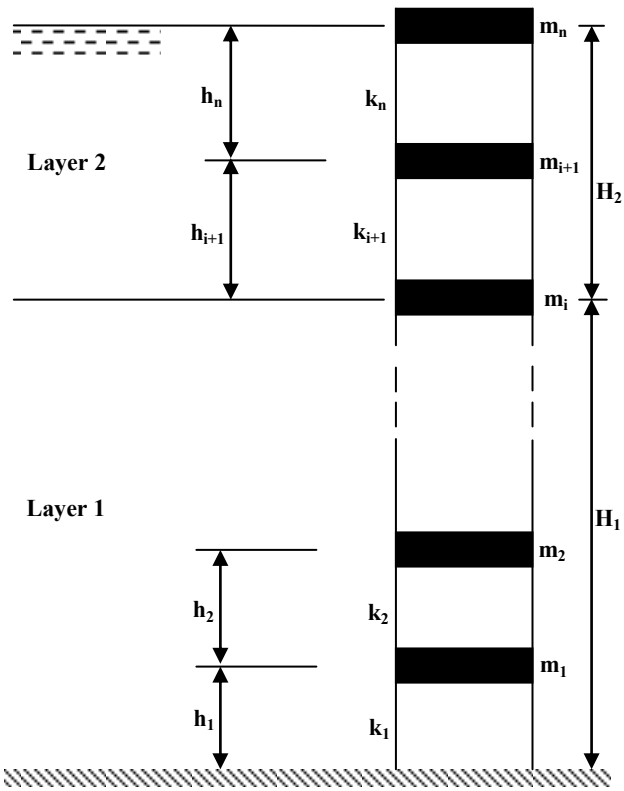

Figure 10: Example of a lumped mass representation of a soil layer system.

Table 3. Properties of the lumped mass system for the two-layer example.

Sublayer	h_i (m)	V_{Si} (m/s)	ρ_i (kg)	$G_{max\,i}$ (MPa)	k_i (MN/m)	m_i (kg)
1	3	193	1860	69.3	23.1	5580
2	3	193	1860	69.3	23.1	5580
3	3	193	1860	69.3	23.1	5580
4	3	193	1860	69.3	23.1	4590
5	2	152	1800	41.6	23.1	3600
6	2	152	1800	41.6	23.1	3600
7	2	152	1800	41.6	23.1	3600
8	2	152	1800	41.6	23.1	1800

$$[A] = [M]^{-1}[K] \tag{27}$$

and the eigenvalues (ω_1^2 to ω_8^2) can be computed using any applicable software. The fundamental frequency, ω_1, corresponds to the lowest eigenvalue, and in this case $\omega_1 = \sqrt{219.6} = 14.82$ rad/s, calculated using MATHCAD. The soil period, $T = 2\pi/\omega_1$ = **0.42 seconds**. This compares closely with the closed form solution (T = 0.44 sec).

NZS 1170.5 method

Section 3.1.3.7, in conjunction to Table 3.2, gives

$$t_{clay} = \frac{0.6\,H_{clay}}{40} = 0.12\,\text{s}$$

$$t_{sand} = \frac{0.6\,H_{sand}}{40} = 0.18\,\text{s}$$

$$T_{1170} = t_{clay} + t_{sand} = 0.30\,\text{s}$$

This method gives a fundamental soil period of ***T* = 0.30 seconds**. Comparing with the numerical and lumped mass solutions (0.44 and 0.42 seconds respectively), the NZS 1170.5 is unconservative with an error of approximately 30%.

(v) Case 6: Three layer profile

Most sites are comprised of a number of layers of soil with different shear wave velocity, mass density and thickness. The final case we analyse in this article is a three-layer soil profile, an example of which is shown in **Figure 11**. Here we present two alternative methods to calculate the site period and compare it with the NZS 1170.5 method.

Successive application of the two-layer solution

This method was developed by Dobry and Madera (described within [19]) and employs successive use of the two-layer closed form solution. The method assumes constant density for all layers and has been shown by Dobry *et al*. (1976) [14] to yield periods less than 10% in error, given this assumption. Where very large differences in mass density exist, there may be more deviation. The following steps are involved:

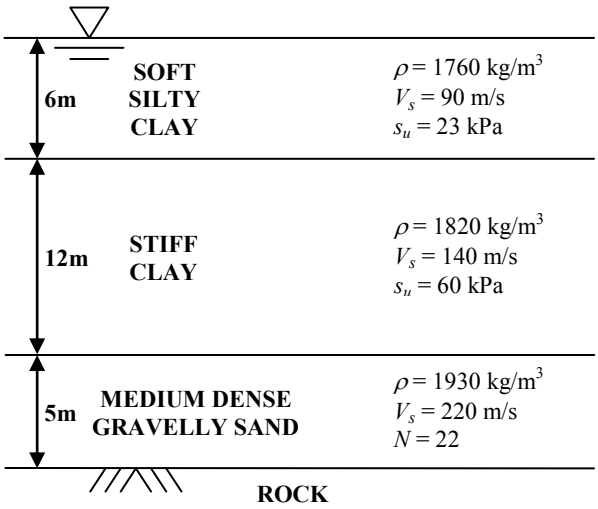

Figure 11: Example three-layer soil profile.

1. The top two layers are assumed to lie on rock and their period, $T_{1\text{-}2}$, is computed with the aid of Figure 8. For this, assume $\rho_A = \rho_B$.
2. The top two layers are replaced by a new top layer with $T_A = T_{1\text{-}2}$ obtained in 1 and $H_A = H_1 + H_2$.
3. The rock interface is assumed below layer 3 of the profile and the period, $T_{1\text{-}3}$, of the new system is estimated by Figure 8 using $T_A = T_{1\text{-}2}$, $T_B = 4H_3 / V_{s3}$ and $\rho_A H_A / \rho_B H_B = (H_1 + H_2) / H_3$.
4. The top three layers are replaced by a new top layer with $T_A = T_{1\text{-}3}$ obtained from 3 above and $H_A = H_1 + H_2 + H_3$.
5. The process is repeated until the last layer is considered. The estimated period of the profile is

$$T \approx T_{1-n}\ , \tag{28}$$

where n is the number of soil layers.

Table 4 shows the solution to the three-layer profile in Figure 11 using this method. The fundamental period the site, ***T* = 0.59 seconds**, therefore on the boundary between class C and class D.

Table 4. Solution to the three-layer example profile using the successive application of the two-layer solution.

Layers considered	H_A (m)	H_B (m)	H_A/H_B	V_A (m/s)	V_B (m/s)	T_A (s)	T_B (s)	T_A/T_B	T/T_A	T (s)
1 & 2	6	12	0.5	90	140	0.27	0.34	1.27	2.0	0.54
1 to 3	18	5	3.6	-	220	0.54	0.091	0.17	1.1	0.59

Table 5. Properties of the lumped mass system for the three-layer solution.

Sublayer	h_i (m)	V_{Si} (m/s)	ρ_i (kg)	$G_{max\,i}$ (MPa)	k_i (MN/m)	m_i (kg)
1	2.5	220	1930	93.4	37.4	4825
2	2.5	220	1930	93.4	37.4	5143
3	3	140	1820	35.7	11.9	5460
4	3	140	1820	35.7	11.9	5460
5	3	140	1820	35.7	11.9	5460
6	3	140	1820	35.7	11.9	5370
7	3	90	1760	14.3	4.8	5280
8	3	90	1760	14.3	4.8	2640

Lumped mass solution

This is the same method as applied in the previous case. Here, we divide the top and bottom layers into two sublayers, and the middle layer into four sublayers. The accuracy of the eigenvalues will depend on the number of sublayers. **Table 5** shows the properties of the lumped mass system. Forming [*M*] and [*K*] matrices as previously gives a smallest eigenvalue of $\omega_1 = \sqrt{116.1} = 10.77$ rad/sec. The soil period, $T = 2\pi/\omega_1 =$ **0.58 seconds**. This compares closely with the successive two-layer solution.

NZS 1170.5 approach

Section 3.1.3.7, in conjunction to Table 3.2, gives

$$t_1 = \frac{0.6H_1}{40} = 0.18 \text{ s}$$

$$t_2 = \frac{0.6H_2}{40} = 0.18 \text{ s}$$

$$t_3 = \frac{0.6H_3}{45} = 0.07 \text{ s}$$

$$T_{1170} = t_1 + t_2 + t_3 = 0.43 \text{ s.}$$

This method gives a fundamental soil period of ***T* = 0.43 seconds**, and the site would be classified as class C. Comparing with the successive two-layer and lumped mass solutions (0.59 and 0.58 seconds respectively), the NZS 1170.5 method is unconservative, with an error of roughly 27%.

(vi) Other cases

For closed form solutions for other example soil profiles (e.g. over consolidated crust overlying normally consolidated clay, shear modulus decreasing with depth), or alternative methods to the previous five cases, we refer the reader to Dobry *et al.* (1976) [14] and references therein.

ISSUES WITH TABLE 3.2 IN NZS 1170.5

As demonstrated in the previous section, NZS1170.5 clause 3.1.3.7 for evaluating of period at layered sites gives results that are inconsistent with other well-accepted calculation methods. For the two-layer and three-layer example soil profiles, the code method is shown to be unconservative by roughly 30%, as shown in **Table 6.**

Accepting the categories of soil and the representative strengths and *N* values from Table 3.2, another assessment of the depth of soil to limit the period to less than 0.6 seconds has been made. This has been done using previously identified empirical relationships from overseas data between strength and shear modulus [3], and N value and shear wave velocity, [9, 10]. **Figure 12a** compares a graphical representation of Table 3.2 with maximum depth limits for a site period of T = 0.6 s for a cohesive soil site with a PI of approximately 20 to 25 and an OCR of 1 and 5. **Figure 12b** is the same for cohensionless soils.

The maximum depths specified in Table 3.2 are inconsistent with the site period boundary of 0.6 seconds. For cohesive soils, the maximum depth specified by the curves derived from available correlations is approximately one half of that from NZS 1170.5. The effect is still evident for cohensionless soils, but not to the same extent. The overestimations in the Table 3.2 maximum soil depths mean that some sites (the sites in the shaded areas of Figures 12a and 12b) that should be classified as class D according to site period are instead classified as class C in NZS 1170.5 i.e. the code is unconservative. We believe that either the maximum depths in Table 3.2 should be amended, or removed altogether. Instead, we recommend that the lumped mass solution be adopted as the preferred method for determining site period for layered sites, as this method is simple, efficient, and easily adaptable to complex sites. Successive application of the two-layer solution is also considered an acceptable method to determine site period, despite being less efficient than the lumped mass solution.

Note that Figure 12b only applies to sites containing quartz sand. The curve should not be used for pumice sands since the correlations in the literature between SPT *N* value and shear wave velocity do not apply to volcanic sands. Pumice sands are a crushable material even under relatively moderate levels of stress with the result that this material produces very different behaviour during CPT testing compared with quartz sands.

Table 6. Comparison of methods to calculate site period for two-layer and three-layer examples.

Example profile	*Closed form solution (s)*	*Lumped mass solution (s)*	*NZS1170.5 clause 3.1.3.7 (s)*
Two-layer	0.44	0.42	0.30
Three-layer	0.59	0.58	0.43

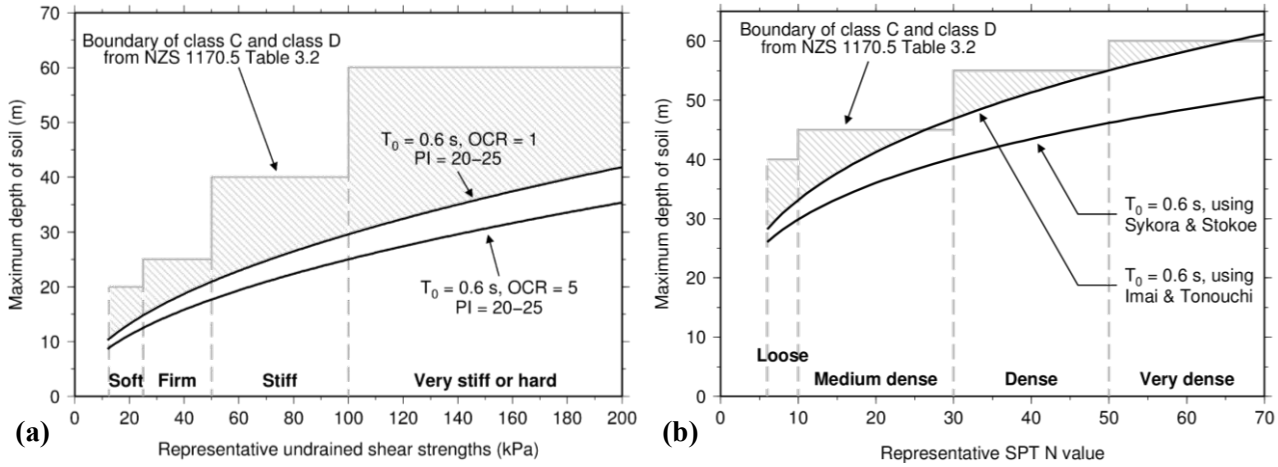

Figure 12: ***(a) Comparison of the maximum depth limits for site class C for cohesive soils in NZS1170.5 Table 3.2 (grey lines) with the depth to give T= 0.6 s, calculated using Weiler (1988) [3] (black curves). (b) Comparison for the maximum depth limits for site class C for cohesionless soils in NZS1170.5 Table 3.2 with the depth to give T= 0.6 s using the correlations of Sykora and Stokoe (1983) and Imai and Tonouchi (1982), equations (7) and (8) respectively. Shaded areas correspond to sites that are classified as C according to Table 3.2, but should be classified as class D according to site period. Note that correlations in (b) are only applicable for quartz sand and should not be applied to pumice sands.***

OTHER METHODS TO DETERMINE SITE CLASS

To this point, this article has discussed methods to determine the fundamental site period with and without shear-wave velocity measurements. These methods correspond to the first two methods from the hierarchy detailed in clause 3.1.3.1 of NZS 1170.5, and clause 3.1.3.7, which is specific to layered sites. However, the code identifies several other methods for determining site class, which we discuss briefly here.

Site period from Nakamura ratios

This technique was introduced by Nogoshi and Igarashi (1971) [21], and popularised by Nakamura (1989) [22], and consists of estimating the ratio between the Fourier amplitude spectra of the horizontal (H) to vertical (V) components of ambient vibrations. While used highly infrequently in New Zealand, this is an inexpensive, simple method of directly measuring the fundamental period at a given site. This method generally performs very well for horizontally layered soil profiles with large impedance contrasts i.e.

$$\frac{\rho_{rock} V_{S,rock}}{\rho_{soil} V_{S,soil}} \geq\sim 4 \quad , \qquad (29)$$

[23] and is generally able to detect surface topographic effects on the site period. However, the results become less certain when the impedance contrasts are more gradual, or when the slope of subsurface interfaces increases. When there is additional noise contamination e.g. from wind or harmonic industrial activity, the interpretation of the H/V ratios can be compromised. This method was the subject of an in-depth European study, SESAME (Site EffectS Assessment using AMbient Excitations). We refer the reader to a special issue of the Bulletin of Earthquake Engineering (2008), Volume 6, Issue 1, which covers the limitations of the method in detail, and gives guidelines for the application of the method.

Site period from earthquake recordings

Another method described in NZS 1170.5 clause 3.1.3.1 for calculating site period is via "recorded earthquake motions". The code is referring to a method known as horizontal-to-vertical spectral ratios (HVSR) from S-wave shaking (Lermo & Chavez-Garcia, 1993) [24]. This method is similar to Nakamura ratios, in that it involves taking the ratio between Fourier amplitude spectra of horizontal and vertical components, however the data is S-wave windows from recorded earthquake motions, rather than from microtremors. H/V curves are generally averaged for several events to give the final HVSR, and greater than 10 events is considered a reliable average.

As the method is from recorded data, it is generally only applicable in New Zealand for GeoNet recording sites (see http://magma.geonet.org.nz/resources/network/netmap.html). Note that NZS 1170.5 is interested in the low-strain fundamental period, therefore the chosen events for the HVSR should be free of nonlinear soil effects (e.g. with a PGA approximately less than 0.1g). This method is only valid where the recording instrument is free-field.

From boreholes with descriptors but without geotechnical measurements

This method is common in engineering practice. With no measured geotechnical properties, representative strength, modulus or shear-wave velocity values are assumed without consideration of the *in situ* conditions that can significantly affect the actual values. Presumably the assumed properties are used then in conjunction with Table 3.2 to determine the site class. Given the inconsistencies in Table 3.2 that we have outlined in this article, and the fact that the assumed geotechnical properties are unlikely to be representative of the true properties, this site classification method should be used with caution as it is unlikely to yield accurate results.

From surface geology and estimates of the depth to underlying rock

This method is described as the "least preferred method" to determine site class under NZS 1170.5. It is unclear how this method yields an estimate for site classification, however again we presume that Table 3.2 guides the selection of site class. Use of Table 3.2 requires geotechnical information and the depth to bedrock, neither of which are measured in this method, thus we also consider this the least-preferred method. This method is unlikely to give a reliable site classification estimate.

CONCLUSIONS

Under NZS1170.5:2004, the fundamental site period, T, is the key parameter to account for the influence of near-surface material on earthquake ground motion. Given that site period is closely related to the shear-wave velocity (V_S), this paper gives a background on methods of obtaining shear-wave velocity using *in situ* measurements, laboratory tests and existing empirical correlations. From there, we examine methods to calculate site period for various types of soil profiles according to the NZS 1170.5 clause 3.1.3.1 hierarchy for site classification. Examples for five types of sites are shown:

- Uniform layer;
- Shear-wave velocity increasing as a power of depth;
- Shear modulus increasing linearly with depth;
- Two-layer soil profile; and
- Three-layer soil profile.

For the two-layer and three-layer profiles, we offer two alternative methods for calculating site period, a closed form solution and a lumped mass solution. The NZS 1170.5 clause 3.1.3.7 is unconservative with respect to these two methods for both the two-layer and the three-layer profiles. We consider the lumped mass solution to be the most accurate and efficient method for calculating site period for layered sites. The Dobry and Madera method of successive application of the two-layer solution, is considered an acceptable, if less efficient method.

An issue is also identified with the bedrock depths in Table 3.2 of the code. The maximum bedrock depths for site class C are inconsistent with the site period boundary of 0.6 seconds. For cohesive soils, the maximum depths are unconservative by roughly a factor of two. For cohesionless soils, the maximum depths are overestimated by roughly 10 to 20%. These inconsistencies result in some sites being classified as class C, when according to site period, the site should be classified as class D. We recommend that Table 3.2 is either amended or removed in the next iteration of the Standard.

Further details are given on alternative methods in NZS 1170.5 to assess site classification, to supplement the information given in the code. Notes are made on their applicability to certain situations.

As a final thought, we suggest that calculation of site period is a task best suited for geotechnical engineers, since they have the greatest depth of technical knowledge with regard to the site characteristics.

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Management and documentation of geotechnical hazards in the Port Hills, Christchurch, following the Canterbury earthquakes

GEOMECHANICS AWARD PAPER 2

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ABSTRACT

This paper outlines the scope of work undertaken by the Port Hills Geotechnical Group (PHGG) to identify, document and assist with the management of geotechnical hazards in the Port Hills area of Christchurch following the 22 February 2011 earthquake. PHGG was a group of consultants formed during the state of emergency and subsequently contracted by Christchurch City Council (CCC). The Group consisted of geotechnical engineers and engineering geologists drawn from Aurecon, Geotech Consulting, GHD, Opus and URS, with support from Canterbury University, Bell Geoconsulting, SKM, Geoscience Consulting and Meridian Energy at different times.

PHGG began as a group dedicated to documenting instability in rock slopes (cliff collapse and boulder roll scenarios) and ground cracking to identify high risk areas and dwellings that should not be occupied as well as identifying areas suitable for protective works and liaising closely with CCC and the public. Following the Civil Defence phase, the role transitioned into working closely with GNS Science and CCC to assist with the development of life-risk models and risk zones while concurrently defining and supervising work packages for interim protective works and more permanent remedial works for key Council infrastructure and parkland.

The PHGG was a very successful team of dedicated professionals that developed excellent working relationships internally and worked collaboratively with its key stakeholders (CCC and GNS). A large part of the reason for this success was that the individuals and companies involved put the job first and focussed on achieving the best possible outcome for the city and its residents.

1 INTRODUCTION

The M_w6.2 Christchurch earthquake of 22 February 2011 caused deaths and injuries, severe damage to tens of thousands of homes, and the devastation of the city's central business district (CBD). In the Port Hills suburbs, five people were killed and over 200 houses were hit and damaged by earthquake-induced rockfall.

In this paper we outline the role of the Port Hills Geotechnical Group (PHGG) and the key lessons learned from its activities during both the Civil Defence Emergency phase that ended on 30 April 2011 and the Recovery Phase that followed. PHGG was a group of geotechnical engineers and engineering geologists drawn from Aurecon, Geotech Consulting, GHD, Opus and URS that formed during the state of emergency and was subsequently contracted by Christchurch City Council. PHGG had support from GNS Science, Canterbury University, Bell Geoconsulting, SKM, Geoscience Consulting, Golder Associates and Meridian Energy at different times.

Initially, the team was dedicated to documenting failures in rock (cliff collapse and boulder roll scenarios) and areas of extreme hill ground cracking to identify high risk areas and dwellings that should not be occupied. Initial work also focussed on identifying areas suitable for protective works and liaising closely with CCC and the public. Following the Civil Defence phase, the role transitioned into working closely with GNS Science and CCC to assist with the development of life-risk models and risk zones while concurrently scoping and supervising work packages for interim protective works and more permanent remedial works on key Council infrastructure and parkland.

The earlier M_w7.1, 4 September 2010 Darfield Earthquake was a larger magnitude event but was centred 35 km west of Christchurch. The ground accelerations in the Port Hills from the Darfield earthquake broadly conformed to 450yr 'design earthquake' (i.e. horizontal PGA of between 0.3 and 0.6g, vertical PGA of 0.3g) and caused very little damage in the Port Hills.

In contrast the M_w6.2 22 February 2011 earthquake had a shallow epicentre directly under the Port Hills and generated horizontal PGAs up to 2.1g with vertical PGAs as high as 2.2g. (Massey et al, 2012a). These are some of the highest earthquake accelerations that have ever been recorded.

During the PHGG operation period there were hundreds of felt aftershocks, many of which caused further damage. The largest of these were the M_w6.2 13 June 2011 and M_w 6 23 December 2011 earthquakes. Managing risks with such high levels of ongoing seismic activity introduced significant issues and challenges.

2 GEOLOGICAL SETTING

The Port Hills rockfall project area is the north-facing slopes of Banks Peninsula approximately between Westmoreland in the west and Godley Head in the east, plus the slopes facing Lyttelton Harbour as far east as Purau (Figure 1). This area was significantly affected by earthquake-induced rockfalls and ground cracking on 22 February 2011 (Hancox et al., 2011) and further rockfalls were caused by aftershocks, most notably on 16 April and 13 June 2011.

The Port Hills suburbs are sited on the northern slopes of the eroded, extinct Lyttelton basalt volcano. The rocks forming the ridges, slopes and sea cliffs of the Port Hills belong to the Lyttelton Volcanics Group rocks of late Tertiary (Miocene) age, and are about 10–12 million years old (Forsyth et al., 2008). These volcanic rocks comprise layers of hard, jointed, lava flows cross-cut by numerous intruded dykes, and interbedded with breccia (scoria), agglomerate (coarse angular gravel), compact sandy ash beds and ancient buried soils. The volcanic rocks are mantled by loess soils derived from wind-blown sand and silt, typically about a 1 m thick and locally more than 5 m thick.

The north-facing Port Hills suburbs are incised by a series of valleys that drain northwards. The lava flows of the volcano extend below the gravels, marine/estuarine silts and sands that underlie the valley floors. Valley sides may be covered with talus or scree aprons from previous slope failure or rockfall intermixed with or overlain by loess colluvium.

Many natural slopes around Lyttelton Harbour and in the valleys draining northward are formed in strong, interbedded lava flows and stand at steep angles. Cliffs formed on many coastal slopes (such as those around Godley Head, Lyttelton harbour and the outer coast) extend inland into the suburbs of Sumner and Redcliffs as remnants of sea-cut cliffs that become increasingly older with distance inland. These steep (~75–85°) cliffs are typically 15 to 30 m high and locally up to ~80 m high.

3 CIVIL DEFENCE EMERGENCY PHASE

The 22 February 2011 earthquake was centred in the Heathcote Valley on the north edge of Banks Peninsula and within the Port Hills suburbs (Figure 1). A peak ground acceleration of 2.2g vertical was recorded at Heathcote School, close to the epicentre, and widespread damage

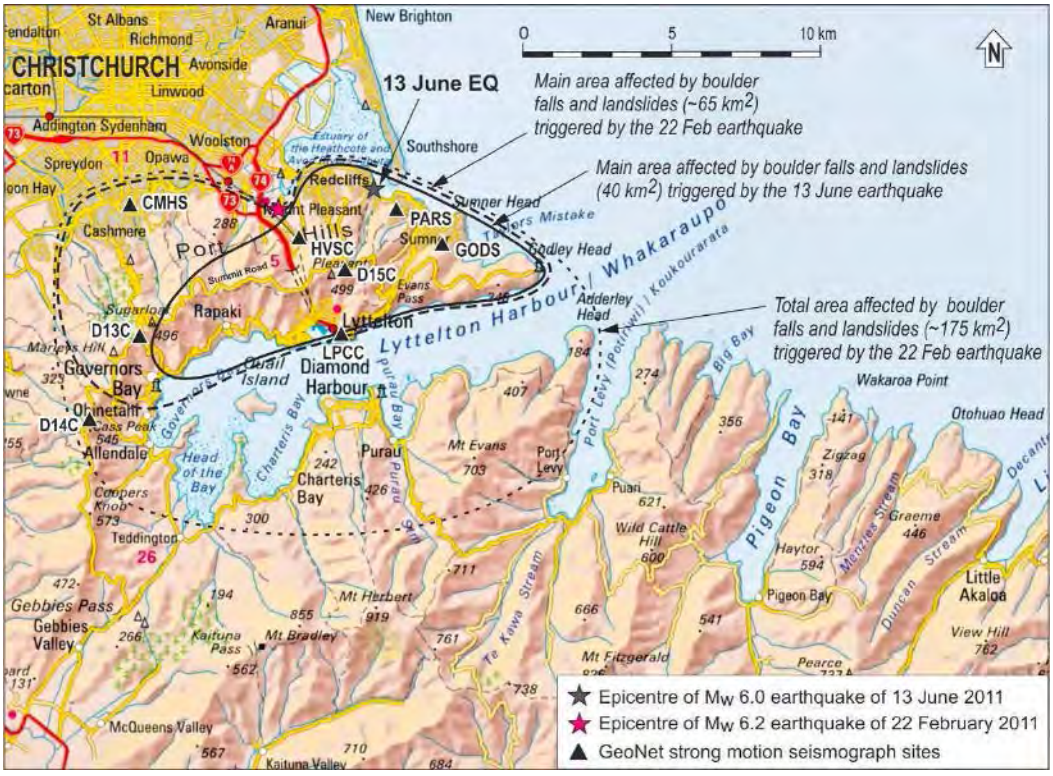

Figure 1. Areas affected by cliff collapse and boulder roll in February 2011 and June 2011 earthquakes. From Massey et al (2012), after Hancox et al (2011).

occurred within the city, particularly the CBD (where two major buildings collapsed), the eastern suburbs (liquefaction, lateral spreading, shaking damage) and the Port Hills (rockfall and shaking damage). As described by McLean and others (2012), a full emergency management structure was in place within two hours, with emergency operations command established in the Christchurch Art Gallery which had sustained only minor damage. The Minister of Civil Defence declared the situation a state of national emergency on 23 February. The state of national emergency stayed in force until 30 April 2011.

3.1 Immediate Response

PHGG began as a group of consultants, university staff and students, supported by GNS Science, who responded to the earthquake by immediately heading into areas of the Port Hills suburbs that they, as individuals or companies, knew to have the potential for stability problems that could affect dwellings or key infrastructure.

One of the first field assessments undertaken was an aerial inspection by GNS Science to identify areas affected by landsliding and to obtain a feel for the scale of geotechnical issues affecting the hill suburbs. Some of the photographs we have included in this review were taken by GNS Science on that flight on 23 February 2011.

CCC quickly recognised the benefits to be gained from having these individuals work together, and organised daily meetings with a designated Chairman (Mark Yetton) to ensure a coordinated approach and feedback of information to a central location. At that stage GNS Science were providing GIS support and collating the daily information to define hazard sources and types, and risk areas.

Within the first week the Port Hills suburbs were subdivided into nine sectors, each managed by one consultant team, to enable field inspections to be better focussed and managed effectively rather than having people operating in a random fashion as they chose. Each team managed

itself and reported back at the daily meetings, where they also shared their thoughts about the observed conditions. It quickly became apparent that different sectors had markedly different issues dominating within them, as summarised in Table 1.

Table 1: Port Hills Sectors and Sector Managers, Civil Defence Phase

Sector	Area	Team	Main Issues
1	Sumner and East	URS/SKM	Cliff collapse, boulder roll
2	Clifton Hill	Aurecon	Cliff collapse, mass movement, boulder roll
3	Redcliffs	Geotech Cons	Cliff collapse
4	Mt Pleasant	University/BGL	Retaining walls, boulder roll, cracking
5	Heathcote Valley	Opus	Boulder roll
6	Lyttelton	University/BGL	Boulder roll, retaining walls
7	Avoca/Huntsbury	GHD	Boulder roll, ground cracking
8	Inner Harbour	Aurecon	Boulder roll
9	Cashmere	University/GHD	Ground cracking, boulder roll

The daily meetings were initially also attended by GNS Science, Urban Search and Rescue (USAR) and Earthquake Commission (EQC) representatives, CCC and Environment Canterbury (Ecan) hazards analysts and GIS specialists. GNS Science handed responsibility for the data management over to CCC after about two weeks of assistance and reduced their participation to field staff. CCC provided a liaison person from about that time and this greatly improved links into the CD emergency management.

In the initial stages, USAR provided daily support to the field teams with advice on procedures for ensuring public safety. One key issue faced by the field teams was the management of Health and Safety, in particular the further instability caused by ongoing aftershocks. Initially, each team managed itself in accordance with company H&S procedures, but adapted for the aftershocks. Later, the team developed a single H&S Plan based on the individual corporate plans and requirements.

3.2 What we found

The four main types of ground damage and associated geotechnical risks observed by the field teams from PHGG and GNS Science are outlined below. Strictly speaking, all can be classified as some form of landsliding but we adopted informal terms that are more generally descriptive of the ground damage and which were more easily communicated to emergency managers and others who are not geotechnical specialists.

3.2.1 Cliff Collapse

There were four major collapses of cliffs (Redcliffs, Peacocks Gallop, Richmond Hill, Whitewash Head) as a result of the 22 February earthquake, and 40 houses were severely damaged. The collapse of cliffs created two main forms of damage and two main geotechnical risks associated with the subsequent aftershocks. These were

(i) cliff top instability with associated ground loss or ground cracking behind the cliff top, in many cases affecting houses

(ii) cliff base inundation of land and buildings (Figures 3 and 4) beneath high cliffs or fallen boulders that in some cases hit houses.

Figure 3. Cliff collapse at Redcliffs, 22 February 2011

Figure 4. Cliff collapse at Deans Head (Peacocks Gallop), 22 February 2011

3.2.2 Rockfall and Boulder Roll

Rocks were dislodged from outcrops on the slopes above lifelines and some of the residential suburbs of eastern Christchurch (eg. Sumner, Redcliffs) and the inner harbour area (Lyttelton, Rapaki, Governors Bay). Rocks fell onto roads (Figure 5) and/or rolled and bounced down hitting homes in residential areas and, in some cases, entering or passing right through dwellings (eg. Figures 6 to 8).

3.2.3 Ground Cracking and Subsidence

Extensive areas of ground cracking were initially observed in different general locations including

(i) the toe slope interface between steeper slopes and flat ground at the base of the slopes, thought at the time to be indicative of different ground responses to shaking

(ii) on loess-covered slopes as cracking roughly following the contour. Some of these areas were initially thought to be potential landslides

(iii) ridge crest cracking where strong amplification of shaking had occurred

(iv) cracking due to movement of retaining walls and/or filled areas, some of which subsided or collapsed (Figure 9).

Figure 5. Rockfall onto Sumner Road, 22 February 2011

Figure 6. Rock inside home, Morgans valley

Figure 7. House penetrated by rockfall, Morgans Valley, on 22 February 2011

Figure 8. House penetrated by rockfall, Rapaki, 22 February 2011

3.2.4 Mass Movement

Areas with both ground cracking and other contiguous deformation such as toe bulging were recognised as discrete mass movement features. In the short term after 22 February 2011, the main locations assessed as being affected by mass movement were Clifton Terrace (Figure 10), Egnot Heights and Vernon Terrace. Some of these areas moved more than a few hundred mm and in some cases ongoing movement appeared to be occurring.

Mass movement was also judged to be the most likely explanation for ground damage observed on Richmond Hill, and at Maffeys Road where the topography suggested possible reactivation of an old landslide.

Survey monitoring was established in each of these areas within a short period after the earthquake with regular resurveys initiated. For these area, PHGG identified houses and services that could potentially be affected by large scale or rapid movements and provided input to emergency evacuation and management plans for each area.

Figure 9. Retaining wall collapse caused by 22 Feb earthquake

Figure 10. Example of ground damage classed as mass movement, Kinsey Terrace, Clifton

3.3 Red Placards

From the first few hours after the earthquake PHGG teams responded to alerts by Civil Defence and structural engineers regarding houses or infrastructure in possible danger from falling rocks or ground movement and also checked known potentially dangerous settings in their sector. The main focus of the work in the Port Hills was to respond to life safety issues affecting lifelines, critical roads and residences.

Due to the extreme ground shaking, many houses suffered significant structural damage. As geotechnical engineers and engineering geologists, PHGG were not qualified to assess structural

damage, hence recommending red placards only on the basis of geotechnical hazards but recommending to USAR that structural assessments be carried out in some cases.

PHGG personnel would inspect the location, confirm and gather the appropriate details, apply their judgement with respect to risk and make a recommendation to USAR regarding placement of a red placard. Where houses were involved PHGG personnel liaised with the structural teams who had the designated powers to issue Red Placards requiring immediate evacuation.

PHGG consultants in each sector developed their own records of properties with dangerous geotechnical issues and were constantly updating and expanding these after further inspections as aftershocks occurred and/or new damage became apparent.

The need for red placards was reviewed at every opportunity, and these ultimately evolved after the emergency phase into S124 notices under the Building Act. When the quantified risk zones for cliff collapse and boulder roll became available following the GNS studies, there was a good correlation between quantified risk estimates and the red placards and S124 notices that had initially been issued based on judgement.

3.4 Documentation of Geotechnical Observations

3.4.1 Field Classification

Field assessment of the stability of individual rocks or rock outcrops was commenced in March 2011 after consultation with EQC and considered a number of criteria, including rock size, shape (potential to roll), evidence for recent movement, and property or lifelines at risk. In this simple classification system, unstable rocks were classified as High, Medium or Low risk on the basis of the following criteria (as shown in Table 2):

1. detached (either as loose boulders or in broken outcrop, as shown by fresh cracking above or below the individual rock or within a rock mass)?
2. ability to roll from their present position (due to their shape) ?
3. pose a direct threat to properties or lifelines?
4. currently unstable (can be wobbled or moved by hand)?

Table 2. Field stability classification for individual rocks and boulders

Criterion	Risk Classification		
	High	**Medium**	**Low**
Loose/detached	X	X	X
Able to roll	X	X	X
Threat to lifeline or property	X	X	
Currently unstable	X		

3.4.2 Data collection and management

In the immediate aftermath of the 22 February earthquake, all field teams were collecting data in any way that they could and providing it to a central location where GNS Science were collating it into a GIS database and using this to prepare preliminary hazard maps. GNS handed this role over to CCC/PHGG after about two weeks.

PHGG developed a standardised spreadsheet for use as a fieldsheet to ensure that all field teams collected the same data and recorded it in the same format. The field data could be re-entered into Excel and transferred to the database. This proved cumbersome, difficult to check and had significant potential for errors.

In recognition of the difficulties with the paper-to-GIS system, PHGG team member Aurecon proposed a real-time GIS-based data collection system that would allow the data to be entered directly in the field using an iPad for live update of the database and would also allow the data to be viewed. Over a period of time the system was refined to provide a number of layers that

could be accessed to show all available or user-selected information at any scale. This system did not become operational until mid-2011.

In terms of data collection and management, the iPad system, also duplicated on the Aurecon server where it was managed, was undoubtedly a huge step forward and was fully embraced by the PHGG consultants.

3.4.3 Field trials

To obtain a better understanding of three dimensional boulder roll patterns in Port Hills terrain PHGG personnel, accompanied by CCC and CERA representatives, undertook field trials to observe the way in which dislodged rocks behaved when rolling down slope. The trials were undertaken in remote, closed park areas on Godley Head and involved dislodging rocks loosened by the earthquakes and observing how they behaved during downslope travel.

Rocks dislodged from very steep outcrops or onto steep slopes (steeper than about 35°) gained momentum as they travelled and ran out for hundreds of metres before stopping. The majority of these rocks travelled more or less down the fall line but some gained sufficient momentum to depart from this line and travel largely independently of the topography until they slowed sufficiently to again be influenced by the terrain and turned to follow the fall line.

Based on these observations we were able to conclude that rolling boulders in Port Hills terrain could deviate by up to 30 degrees from the fall line, thus defining a cone of risk below any potential rockfall source. We also found that most rocks that gained sufficient momentum to continue to travel down slope broke up as they went thereby influencing their final travel distance.

Most of the slopes on which we conducted these trials were not vegetated but for one trial we were able to roll rocks into a gully that had been planted with flax and other native plants over a distance of about 100m. This demonstrated that vegetation was extremely effective at stopping rolling boulders.

3.5 Treatment Options

Almost immediately after the 22 February earthquake, specialised ropes-qualified contractors were brought into the Port Hills to help stabilise outcrops or detached boulders with the objective of protecting lifeline and key infrastructure. This work was extended to some slope above residential areas, partly to make bluffs safe for the field geologists to work below so that they could map and assess the slopes.

Where approved by CCC, unstable rocks were treated by

1. moving them into a stable position; or
2. breaking them into smaller pieces that could then be moved into a stable position; or
3. holding them in place (eg. with spot bolts, cables or mesh)

The work was managed as Work Packages scoped by PHGG and approved by CCC. PHGG developed interim design guidelines for mechanical fixes but there was a strong preference for stabilising or breaking rocks as these options avoided future maintenance requirements. The solution adopted at each treated site was decided in the field in consultation with the contractor who would undertake the work.

It was recognised that these solutions would not be suitable in all cases – for example for very large rocks or unstable bluffs more extensive works may be required to ensure their stability. There was no approval to undertake more major works such as anchoring, rock bolting, excavation or concreting, nor for the construction of protective works such as catch fences or other barriers.

The completed interim works were thoroughly tested by the 13 June aftershocks. Although many treated sites suffered some damage, no completed works failed in that event.

3.6 Community Liaison

One of the difficulties faced by PHGG was its role relative to that of EQC. As an insurer, operating under specific legislation, EQC was completely focussed on land and property damage whereas the priority for PHGG/CCC was the protection of life and infrastructure. While EQC did attend PHGG's daily meetings, their technical resource was initially focussed on the flat land, where much larger numbers of people were affected. During the CD Emergency phase, this left PHGG as the main point of contact for many issues that affected the Port Hills community.

3.6.1 Response to public enquiries

Throughout the CD Emergency Phase, CCC maintained a spreadsheet summarising requests for information received from Port Hills residents. Many of the requests were of a geotechnical nature and were passed to the sector teams for a response. Often this required a site visit and discussion with the owner or tenant (who may or may not have been resident at the time) with the result advised back to CCC to allow the spreadsheet to be updated and any formal response prepared.

3.6.2 Public Meetings

One of the major contributions of the PHGG team during the emergency phase was attendance at large public meetings organised by Civil Defence, large community meetings arranged by local community groups such as the Sumner Residents association and street level meetings involving smaller groups of residents. The geotechnical information shared at these meetings varied with the audience but was typically:

(i) Large public meetings – statements updating the public on the state of knowledge in their area. These were meetings managed by Civil Defence to provide information on a wide range of issues, with limited opportunity for questions and answers.

(ii) Community Meetings – usually involving several hundred people. PHGG provided: a briefing on geotechnical issues in the particular suburb, using visual aids where possible; interactive Q+A of a general nature; individual discussions in relation to individual properties or small groups of properties with common concerns.

(iii) Street meetings - briefing on geotechnical issues in the immediate area, using visual aids where possible; interactive Q+A of a general nature; individual discussions in relation to individual properties or small groups of properties with common concerns.

3.7 Key lessons from CD Emergency phase

The key lessons from the Civil Defence Emergency phase were

1. First response can be haphazard if there is no management strategy in place before such an event
2. Sectoring the affected area is extremely effective for focussing assessment teams in a systematic and more uniform way
3. Daily meetings allow the lessons and observations from each sector to be shared with others, and this better informs subsequent inspections (eg. by checking for similar issues).
4. A formal procedure on which to base the decision to recommend a red placard is essential to ensure a consistent approach and for documentation of the decisions. A simple flowchart was developed for later use.
5. Uniformity of data capture is critical to using the data effectively later. We did reasonably well but in hindsight we lost the opportunity for better quality ground cracking data and were disadvantaged by inconsistent recording of boulder sizes.

6. Having an electronic data collection a system in place and operational sooner would have saved much time that was spent entering data by hand, verifying data and producing paper maps for use in the field.
7. Health and safety management procedures that are appropriate for an emergency situation must be in place
8. Informal terms that are generally descriptive of the nature of ground damage, hazards and risks are more easily communicated to emergency managers and others who are not geotechnical specialists
9. Consistency and honesty with the public in meetings is ultimately repaid in good relationships and trust.

4 RECOVERY PHASE

Following the Civil Defence Emergency phase that ended on 30 April 2011, PHGG became part of the Port Hills Rockfall Recovery Project, with each of the main consultants contracted directly to CCC.

Many of our previous tasks continued with little change (for example, responding to public enquiries and attendance at public meetings and street scale meetings) the main difference being that these were now managed through normal CCC procedures and processes.

With this change at the end of the CD phase, PHGG reorganised itself to make best use of the technical abilities of its senior members. The changes included forming a leadership team, an internal 'engineering' team and transitioning the day to day sector management to the sector leaders. Regular meetings to coordinate activities, maintain awareness of what was happening in different areas and to ensure a reasonably standardised approach to tasks, documentation and reporting were continued, but the frequency was reduced and periodically reviewed.

At the same time the recognition of need for independent peer review led PHGG to recommend to CCC that they appoint an external reviewer to oversee and if necessary guide PHGG activities. Dr Fred Baynes was appointed as reviewer in June 2011 with a scope of work agreed between himself and Council.

Council subsequently appointed their own Geotechnical Advisor to liaise with PHGG, the reviewer and GNS Science, and to advise Council staff and management.

4.1 S124 notices

In July 2011, the CD Red Placards were replaced by Section 124 Notices, issued under the Building Act, that prohibited entry into or occupation of the affected dwellings. PHGG undertook a review of all 560 Red Placards on residential properties and recommended that 108 of these should not be reinstated as s124 Notices because the life-safety risk was judged to no longer remain for these dwellings. PHGG also provisionally identified 96 properties as 'retreat' properties that should be demolished and abandoned as too dangerous for safe residential occupation. All of these were within the rockfall risk areas shown on Figure 11, and were located on cliff edges or immediately under cliffs, with a small number in areas where there had been large numbers of boulders impacting homes.

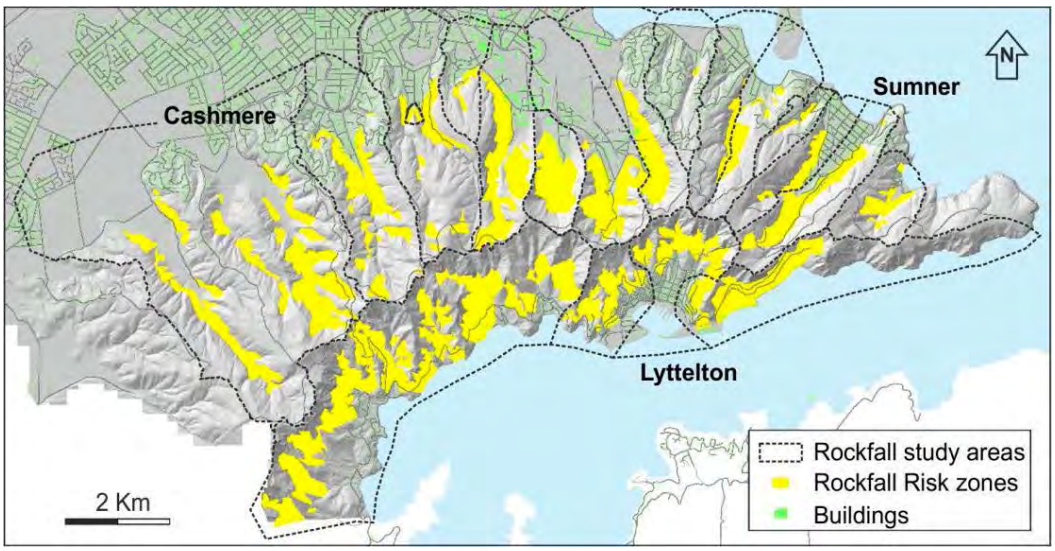

Figure 11: Rockfall risk zones. Courtesy Chris Massey, GNS Science.

4.1.1 Criteria

An s124 Notice could only be applied to a dwelling, and only where the life safety risk to occupants of the dwelling was judged to be so high that this restriction could be justified. As the majority of s124 Notices were placed well before the GNS life-safety model (released March 2012) had been developed, the s124 recommendations were based solely on individual site characteristics.

In boulder roll risk areas PHGG used field observations and judgement based on the following criteria:

1. Did rocks fall on this or an adjacent property?
2. Did rocks reach or pass the dwelling?
3. Was the dwelling hit by rocks?
4. Is the slope above the dwelling steep enough for rocks to roll down it?
5. Are there obvious sources for further rockfall?
6. Is there effective natural or man-made protection for the dwelling? This may be one or more of vegetation (e.g. shelter belts, plantations, dense scrub), another house(s), rock fences, bunds or topographic controls.

The protection was not deemed effective if it had been passed or penetrated by rockfall boulders (eg. if some rocks/boulders had passed right through a shelter belt or plantation it was not an effective barrier even if it had stopped other rocks).

For each dwelling assessed, a flow chart was completed on the basis of the joint opinion of two team members, one of whom was required to be a senior professional. Figure 12 shows the flow chart for boulder roll situations. Similar flow charts were developed for cliff top, cliff bottom, landslide and retaining wall situations.

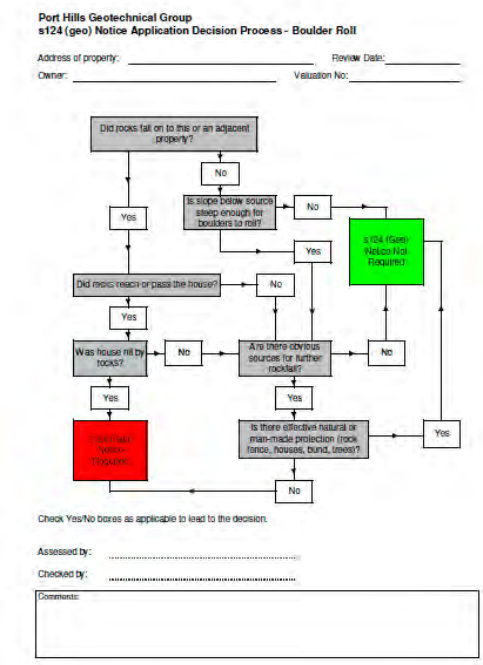

Figure 12. s124 Assessment form

Figure 13. Sample location for which s124 recommended

The breakdown of the various geotechnical issues that required the placement of an s124 notice on dwellings as of July 2011 is presented in Table 3. Risk from boulder roll was the most common reason for the s124 notices in all areas other than Sector 3, where cliff collapse is the dominant issue.

Initially, the s124 Notices expired after 60 days and had to be reassessed and replaced at that time. The Building Act was amended by an Order in Council to remove this requirement for a defined period (through until September 2013). At the time of writing, Council and the Department of Building and Housing are seeking to have this period extended.

Table 3. Main reasons for s124 Notices

Sector	Main reasons for s124 Notices (as at July 2011)					
	Boulder roll	Cliff Top	Cliff Bottom	Cracking	Access	**Total**
1	50	8	4			**62**
2	70	34	16			**120**
3	1	27	30			**58**
4	33	8	13	19	23	**96**
5	23					**23**
6	14			12	5	**31**
7	34					**34**
8	8					**8**
9	24					**24**
Totals	**257**	**77**	**63**	**31**	**28**	**456**
%	*56*	*17*	*14*	*7*	*6*	*100*

Note – 108 Red Placards were not replaced with s124 notices at this time. Most had been applied for cracking or dangerous access associated with retaining walls

4.1.2 Hazard Verification Reports

When it became apparent that some homeowners were defying the s124 notices and remaining resident in their dwellings, Council requested PHGG in late 2011 to develop a simple 'hazard verification report' that confirmed the reason for the dwelling having been assessed as hazardous, and that the dwelling was still considered to be at risk from a rockfall/boulder roll or cliff collapse event.

Council subsequently decided in December 2011 that by advising the owners who remained resident in their s124 dwelling that they were choosing to remain in a location with a heightened risk they had discharged their obligations under the Building Act. On this basis, Council would only physically enforce the notices at their discretion, based on advice from PHGG.

4.1.3 Temporary access plans (TAP's)

The Section 124 prohibited access notices only apply to buildings. In the interests of security and assessment for insurance assessment or repairs purposes, Council allows temporary access to the dwellings but people wishing to access their s124'd dwelling must complete an application form explaining the purpose, date and duration, and the proposed safety plan. Each application has been reviewed by the PHGG geotechnical lead for the sector to identify any properties with an unacceptable level of risk, to which access may be declined or restricted.

4.2 Life-risk Zoning

A very large and key part of the work undertaken to better understand and assess risks associated with natural hazards in the Port Hills has been the studies completed by GNS Science, with assistance from PHGG, to define and quantify life-safety risk zones. These studies are documented in a series of reports by GNS Science [see reference list for details] and have subsequently been used to aid Government, Council, insurers and individuals to make decisions in relation to their management of ongoing risk from those hazards.

The fallen boulder data upon which the GNS Science life-risk modelling was based was collected by GNS and PHGG. The different consultancies forming PHGG worked together with GNS Science, and agreed on the data to be gathered and on the definition of modelling parameters. The sector leaders were responsible for ensuring the integrity of the data. Without the fallen boulder data generated by the 22 February 2011 earthquake, and the subsequent further falls in various aftershocks, the reliable quantification of the risk zones would not have been possible. The fallen boulder database is now a unique and valuable asset that may have important uses in other risk studies, both in New Zealand and other countries.

4.2.1 Life-safety Risk Zones

The life safety risk zones developed by GNS Science in boulder roll areas are defined by a model developed on the basis of field data including boulder size, distribution across and down the slope, runout distances and angular relationships between source areas and runout distances. The model is based on the actual locations of boulders that fell and rolled downslope and hence includes the influence of topography, trees, fences and houses on some of the runout distances and trajectories. It was field calibrated by back tracking actual boulder paths and cross checked by rockfall modelling calibrated against slope surface materials and maximum runout distances. The modelling used the 95th percentile rock for the particular suburb and vegetation was excluded from the rockfall modelling as it cannot be considered a permanent feature of the slope. These factors combine to make the model somewhat conservative, which is normal practice for risk management.

For cliff collapse areas, the life safety risk zones were defined for both cliff top retreat and for the risk of debris inundation or flyrock at the base of cliffs.

The independent peer reviewer retained to give Council confidence that the services provided by the geotechnical consultants and GNS with respect to assessing life safety risk from rock fall and cliff collapse in the Port Hills were appropriate and reasonable.

Ground truthing

Ground truthing of the boulder roll and cliff collapse models was undertaken in early 2012 to field check the risk zones developed for the boulder roll and cliff collapse life safety risk models. Using a process agreed between GNS and PHGG, every property within the risk zones was individually assessed by pairs of experienced geo-professionals working with a standard form to ensure consistency. Individual sector leaders generally worked with a moderating geo-professional from outside the sector. The final life safety risk zones were adjusted for local topographic factors (ridges, gullies, flat areas) and source variations that could affect boulder numbers, roll paths and run out distances. Thus for each house the broad suburb scale GNS model was judged to either reasonably estimate, under-estimate or over-estimate the risk. For example, at a house within a local gully that could focus boulder roll, the risk might be assessed as greater than indicated by the GNS model.

GNS used this information to finalise the risk zones for the models and PHGG also used the field inspections to backcheck the need for s124 notices on the dwellings assessed.

4.3 Review of CERA zoning decisions

The Canterbury Earthquake Recovery Authority (CERA) is a government department formed in response to the February 2011 earthquake to help with the recovery. CERA progressively undertook zoning of the Port Hills between June 2011 and late 2012, with all dwellings ultimately being zoned Red or Green (in the interim, land was zoned white, meaning 'not yet assessed'). For properties zoned Red, the Government will make an offer to purchase the home and land; there is no Government offer for properties zoned Green. In the Port Hills, the zoning decisions were driven by life-safety risk and the Red zones were largely determined by CERA using the Year 5 risk models for boulder roll and cliff collapse risk. At the time of writing, the CERA zoning of the Port Hills is under review.

4.3.1 Pre-June 2012

The Canterbury Earthquake Recovery Authority (CERA) announced its first zoning decisions for the Port Hills on 22 June 2011, when the whole of the Port Hills and Banks Peninsula were zoned White (decision pending). By May 2012, through a series of decisions, most properties on broad gentle slopes or ridge tops had been determined to be safe for residential occupation and zoned Green.

PHGG had little involvement in these decisions, but the Green zone did include a number of properties with s124 Notices and PHGG reviewed these in May 2012, concluding that for 53 of the 77 properties reviewed there was no defensible basis for uplifting those notices.

4.3.2 After June 2012

On 29 June 2012, CERA announced the rezoning of 1107 residential properties on the Port Hills from White to Green, and that the Government's Red zone (buyout) offer would be extended to the owners of 285 severely at-risk or largely destroyed residential properties in the Port Hills. Of these properties, 191 related to cliff collapse and 94 to boulder roll. A further 158 properties that initially remained in the White zone due to mainly to boulder roll issues were subsequently rezoned Red in decisions announced in August and September 2012.

S124 notices were automatically uplifted from approximately 130 properties that were re-zoned to Green. PHGG reviewed the properties that lay close to the white/green boundary and identified twelve dwellings as 'should not have the s124 notice uplifted'. These notices were reinstated and, at CCC's request, the remainder were subsequently reviewed in August-September 2012 to confirm that the uplifts were appropriate. This review included 2D rockfall modelling that specifically considered the effects of vegetation on these slopes for the first time.

4.4 Remedial works

Physical works have been undertaken in a number of areas of the Port Hills since March 2011to remediate high risk rocks posing a hazard to houses, roads and key infrastructure, and in areas of park and conservation land. In many cases, due to the scale and complexity of the rock

outcrops these works are considered by CCC to have been interim hazard management rather than permanent remediation.

Although many treated sites suffered some damage in the 13 June aftershocks, no completed works failed in that event. However, that event loosened and dislodged many more rocks and highlighted the challenge faced in identifying and treating potential rockfall sources. As a result, work to protect homes was largely discontinued and the emphasis was placed on protection of Council assets and Parks from that time. Works at source to protect Evans Pass Road have been completed and the road reopened to all traffic (Engel 2013), and at the time of writing work is continuing on sections of the Summit Road.

4.5 Roading options studies

PHGG have carried out preliminary hazard assessments for a number of lifelines and key routes (non-NZTA) in the Port hills including Evans Pass, Summit Road (Evans Pass to Mt Pleasant Road), Port Hills Road, Dyers Pass, Lyttelton to Governors Bay (alternative to Tunnel route), Summit Road (Evans Pass to Scarborough), Mt Pleasant Road (Summit Road to Main Road) and Bridle Path Road.

Sumner Road, which runs between the Evans Pass/Summit Road intersection and Lyttelton, has been closed to all traffic since February 2011 due to debris on the road and the presence of extensive earthquake induced instability on the slopes and cliff faces above the road. In an options study reviewed by PHGG, reopening Sumner Road was compared with the upgrade of alternative routes over the Port Hills and the construction of a new road. No final decision has yet been made.

Two sections of Main Road, which links Sumner and Ferrymead, have been affected by cliff collapse. CCC commissioned engineering options reports for both the Moa Bone Cave and Peacocks Gallop areas that are currently protected by containers as an interim solution (see Figures 14 and 15).

Figure14. Containers protecting Main Road at Peacocks Gallop

Figure 15. Rockfall debris behind containers at Peacocks Gallop.

4.6 S124 reviews

Approximately 130 properties that were Green zoned as a result of the 29 June announcement were subject to automatic uplift of s124 notices. This was a policy decision and was not based on the criteria used to assess the need or otherwise for s124 notices.

PHGG's review of the automatically uplifted s124 Notices in the Green Zone was completed in October 2012 and included desktop review from which properties requiring field inspection and/or rockfall modelling for a robust decision were identified. This review provided Council with documentation of the basis for uplifting or reinstating those s124 Notices that were automatically uplifted. It was also used as a check on properties considered by PHGG to have been marginal calls for zoning decisions but have not previously had s124 Notices.

The outcomes from the review included a recommendation to reinstate 22 of the s124 Notices (including the 12 reinstated almost immediately after the 29 June announcement). For all

remaining properties reviewed, the uplift of the s124 notice was assessed to have been a reasonable decision.

4.7 October 2011 Rainfall Event

On 19 October 2011, the Port Hills were subject to the most significant rainfall event since the 22 February earthquake. This triggered inspections of the Port Hills areas by the sector teams to determine the effects on damaged land areas and potentially unstable rock source areas. The inspections showed widespread minor effects but no large scale changes due to the rain. Up to the time of writing the Port Hills have not been subject to the type of high intensity or prolonged heavy rainfall which can occur, particularly in the late winter period.

4.8 S124 Determinations

Property owners who disagreed with their s124 Notice were able to request a formal review (Determination) by the DBH (Department of Building and Housing, Ministry of Business, Innovation and Employment, MBIE).

At the time of writing, 12 Port Hills residents in boulder roll zones had requested a formal review of their s124 Notice by the DBH, but 3 had subsequently withdrawn.

4.9 Port Hills risk management zones (District Plan)

In late 2012, PHGG provided inputs to Christchurch City Council workshops working through a range of issues relating to future planning on the Port Hills. This work includes potential changes to the Christchurch City Plan and Banks Peninsula District Plan to strengthen the Council's existing consenting processes in the light of post-earthquake knowledge.

Changes to the District Plans relating to the management of land-use and subdivision activities in high risk areas may include zoning to prevent or control new developments and controls (resource consent requirements) for rockfall protection work (including rockfall protection structures) to keep the natural hazard risk to a level that is as low as reasonably achievable.

4.10 Rockfall protection structures

The design of rockfall protection is a complex task that requires the consideration of many different kinds of data (geological, geotechnical and topographical) from the site and source areas as well as trajectory analysis (path and bounce height) to estimate the kinetic energy that must be managed by the design.

PHGG has been involved in two levels of assessment of rockfall protection for Port Hills properties:

4.10.1 Suburb-scale rockfall protection structures (RPS)

PHGG and CERA held a combined workshop in July 2012 to discuss rockfall protection. The overall objectives of the workshop were to better define locations and dimensions for large-scale rockfall protection works suggested on the basis of preliminary 3D rockfall modelling undertaken for CERA and use this to develop cost estimates from which CERA would undertake cost-benefit analyses.

CERA subsequently concluded that area-wide (suburb-scale) protective works were not practicable for a number of reasons including uncertainty around timeliness and costs. The areas that had been considered for area-wide protective works were then zoned Red by CERA.

4.10.2 Private RPS

Following a decision by Councillor's on 6 December 2012, owners of Port Hills properties who had received a Red zone offer could apply to the Council for funding to erect a private rockfall protection structure. CCC developed, with input from PHGG, contractors, suppliers and other consultants, a *Technical Guideline* to provide design guidelines for rockfall mitigation in the Port Hills. The design objective is to reduce the life-safety risk to an acceptable level.

Approval is a two stage process. Firstly, applicants need to retain an engineer to develop a design for Council to approve, and for which building and/or resource consents may need to be issued. When the application iss received, approved consultants are retained to undertake the design review for Council. Once consented, the funding can be applied for. Funding is capped at Council's 50% share of the Red zone offer for the property.

At the time of writing, eight pre-application meetings had been held and construction was underway on one site.

4.11 Demolitions

Although no building consent is required for the demolition of houses in the Port Hills, Council has recommended that geotechnical advice is obtained prior to commencing demolitions in the Port Hills. PHGG sector leaders (using a standard form) identify any known geotechnical hazards on the basis of advice of proposed demolitions.

PHGG also created a list of properties that currently provide down-slope protection (from rockfall/boulder roll only) to other properties such that, if that property was demolished, it would be necessary to assess whether or not a new s124 would need to be placed on the property or properties currently protected by those upslope.

4.12 Lessons Learned in the Post-emergency Phase

Further lessons learned in the post-Civil Defence Emergency phase included:

1. Given our current knowledge of the future seismic hazard it is simply not possible or cost effective to identify and remediate all potential rockfall source sites over large areas, rural or park land. Protection, retreat and/or relocation of infrastructure away from these potential boulder roll sources are the only feasible solutions.
2. Further aftershocks may induce new instability in the damaged rock mass at locations that often defy prediction. In the large June 2011 aftershocks new rockfalls came from sites that had no obvious indications of damage from the previous earthquakes.
3. Potential rockfall source areas that have been built over (eg. Scarborough) did sustain shaking damage due to high PGA's but had few problems with boulder roll as roads, retaining walls and buildings constructed as part of the development prevented rockfalls or captured rocks before they gained momentum.
4. Properties that have been zoned Green or Red by CERA, primarily on the basis of application of life-risk models that necessarily 'average' the risk across a wide area, will not always align with s124 Notices placed using other site specific criteria
5. Communications to keep affected parties as fully informed as possible are crucial. Much of the angst in the community is due to the inability to provide all the information required to make individual decisions in the shortest possible time frames.

5 CONCLUSIONS

The damage caused on 22 February 2011 by a very strong earthquake located directly under the residential hill areas of Christchurch demanded an immediate emergency geotechnical response. This response came largely from locally-based engineering geologists and geotechnical engineers (working in conjunction with USAR engineers) from a range of companies and organisations, and from GNS Science.

The local consultants involved fortuitously aligned into a robust and efficient working group who became known as the Port Hills Geotechnical Group (PHGG). The PHGG was a very successful team of dedicated professionals that worked collaboratively with its key stakeholders (CCC and GNS). A large part of the reason for this success was that the individuals and companies involved focussed on achieving the best possible outcome for the city and its residents.

As soon as possible following the disaster it is essential to remove the risk to life by evacuating and/or restricting occupancy in high risk areas until lifelines and properties can be adequately assessed and/or protected.

The lessons learned during this emergency response, and the successful methods that were developed and applied to mitigate and manage the immediate risk, have potential application to other regions and towns. The organisation and preparation of appropriate management and documentation systems should not wait until a disaster that requires a geotechnical emergency response has already occurred.

Documentation in the early stages need not be complicated but systems need to be in place and be consistently applied for a sensible and defensible outcome. There was a very good correlation between high risk properties identified on the basis of site-specific factors and given Red Placards (or s124 Notices) in the immediate aftermath of the February 2011 and June 2011 aftershocks and the highest risk zones later defined in the GNS life-safety risk models.

At the time of writing (mid June 2011) many of the evacuated properties retain s124 notices and most will remain until significant mitigation (such as rock scaling or breaking, bolting, berms or rock fences) occurs to reduce the risk to acceptable levels.

It will almost certainly take longer than initially expected to recover from a major natural disaster. Consequently, consistent communications to keep affected parties as fully informed as possible are crucial during the emergency phase and must also continue through the recovery phase.

In the post emergency phase the recognition of previously unknown active faults directly under the city has led to a radical reassessment of the local seismicity model. The revised seismicity model, in conjunction with the unique dataset of the observed pattern and density of earthquake triggered rockfall, has led to the first extensive quantitative risk modelling undertaken for a New Zealand city.

The risk modelling has formed a defensible basis for long term decisions on the need for retreat from some areas of the hills and provided appropriate information that will be incorporated into future District Plan revisions.

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Assessment of Liquefaction-Induced Land Damage for Residential Christchurch

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ABSTRACT

Christchurch, New Zealand experienced four major earthquakes (M_w 5.9 to 7.1) since 4 September 2010 that triggered localized to widespread liquefaction. Liquefaction caused significant damage to residential foundations due to ground subsidence, ground failure, and lateral spreading. While liquefaction effects were expectedly severe in some suburbs, there was little to no damage in other areas, where more serious effects were expected based on existing liquefaction vulnerability criteria. These damage variations indicate that existing liquefaction vulnerability criteria do not capture fully the consequences of liquefaction. This paper first presents some general features of liquefaction-induced damage to land and dwellings in residential areas, and then examines the effectiveness of liquefaction vulnerability parameters in predicting/explaining the observed liquefaction-induced damage in residential areas of Christchurch. The Liquefaction Severity Number (*LSN*), a new parameter, is presented and discussed using results from 5,500 CPT and validated regional groundwater models.

Keywords: Foundations, Ground Failure, Liquefaction, Seismic Performance, Vulnerability

INTRODUCTION

Christchurch City and the Canterbury region of New Zealand (NZ) have been affected by a series of earthquakes and aftershocks with the four most significant earthquakes being: 4 September 2010 (Mw 7.1), 22 February 2011 (Mw 6.2), 13 June 2011 (Mw 6.0), and 23

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December 2011 (M_w 5.9). The liquefaction-triggering magnitude-weighted (Idriss & Boulanger 2008) peak ground accelerations (*PGA*) in the western and northern parts of Christchurch were highest in the Sept. 2010 earthquake, whereas in central, eastern, and southern Christchurch they were highest in the Feb. 2011 earthquake. The characteristics of the ground motions during the Canterbury earthquake sequence are described in Bradley et al. (2013).

The earthquake shaking from these events triggered localized to widespread minor to severe liquefaction in Canterbury. In some Christchurch suburbs, the damage caused by the liquefaction was severe as illustrated in Figure 1. The incidences of subsidence associated with voluminous liquefaction ejecta and other modes of ground deformation, such as lateral spreading, was a primary cause in the observed damage to residential dwelling foundations in the Canterbury region. While extensive triggering of liquefaction was observed, in some areas this triggering had little to no consequence to the built environment, where more serious effects might have been anticipated from conventional analyses based on estimated soil properties. These observed variations indicate that existing liquefaction vulnerability criteria were not able to capture fully the consequences of liquefaction.

Land is insured for natural disaster damage in New Zealand under the 1993 Earthquake Commission (EQC) Act. As a result, an extensive land damage assessment process was undertaken to characterize the extent of liquefaction-induced land damage and quantify losses. Initially, existing liquefaction vulnerability methodologies were utilized, and results were compared to the observed land damage datasets to facilitate the claims settlement process and to understand the reasons why some parts of Christchurch were affected more seriously by liquefaction. The results showed that existing liquefaction vulnerability procedures were not able to capture the observed damage data well. Consequently, a new liquefaction vulnerability parameter was developed.

There is extensive literature on the liquefaction phenomenon and liquefaction triggering evaluation procedures, but there is substantially less work on vulnerability indicators that address the consequences of liquefaction for residential dwellings on shallow foundations.

This paper provides a review of existing liquefaction vulnerability methods that evaluate the likelihood of consequential liquefaction at the ground surface, and compares the results obtained for these methodologies with the liquefaction-induced land damage observations obtained after major events in the Canterbury earthquake sequence.

Figure 1. Observed liquefaction-induced land damage and dwelling foundation damage due to Christchurch earthquakes: (a) Extensive liquefaction in low-lying Christchurch suburbs (23/02/2011); (b) Suburban Christchurch street covered with liquefaction ejecta (23/02/2011); (c) Pavement completely buried by liquefaction ejecta and ponded water after liquefaction (24/02/2011); (d) Surface water flowing over liquefaction ejecta with collapsed concrete block wall (22/02/2011); (e) Liquefaction ejecta next to brick house that subsided (25/05/2011); (f) Uplift of concrete floor inside house with liquefaction ejecta and water mark around base of walls (01/03/2011).

A new liquefaction vulnerability parameter, Liquefaction Severity Number (LSN), is presented and compared to the observed liquefaction-induced land damage datasets. The development, observations, and calculations that provide support for LSN are described in greater detail in a series of reports prepared for the EQC, which are summarized in Tonkin &Taylor (2013). Only some of the salient aspects of this work are discussed in this paper.

DAMAGE MAPPING AND FIELD INVESTIGATIONS

Following the four major Christchurch earthquake events, a qualitative survey of land damage and dwelling foundation damage was undertaken as part of the coordinated response by agencies of the NZ government. Liquefaction-induced land damage mapping of residential properties (based on criteria defined in the left column of Figure 2) was carried out immediately after the September 2010, February 2011, and June 2011 earthquakes to assess the extent and severity of the surface effects of liquefaction. The land damage mapping was carried out by a small team of senior geotechnical engineers who cross-checked observations to ensure broad consistency across their assessments.

In addition to the land damage mapping, a more detailed land damage inspection program was undertaken on each of some 65,000 insured residential properties by a team of approximately 400 engineers for insurance claim damage assessment purposes. Visually observed damage to the foundations of homes was recorded based on the criteria described in the right column of Figure 2. These detailed property inspections were undertaken where land damage claims were lodged with EQC between October 2010 and December 2012. Generally all properties with land damage were inspected at least once during this period and many with a reassessment after subsequent events. In all, some 100,000 inspections were made. Not all properties with land damage after September 2010 had a detailed land damage inspection completed before the February 2011 event. After February 2011, properties were generally inspected once. Thus, properties inspected before June 2011 would not have included additional damage from the June 2011 and December 2011 events. Similarly, inspections before December 2011 would not have picked up damage from the December 2011 event.

An example of the land damage distribution from the February 2011 event is shown in Figure 3. The foundation damage to dwellings has been compiled into a database and the worst overall severity from the seven assessed damage categories (defined in Figure 2) was then plotted on a map. If more than one detailed inspection was undertaken for the property, then the inspection with the worst foundation damage was plotted on the map. The distribution of dwelling foundation damage as accumulated from all four earthquakes is shown in Figure 4. The white areas on the maps of Figs. 3 and 4 are land parcels (in non-residential areas) where observations of land or foundation damage were not undertaken.

Land Damage Categories	
Category	Criteria / Description
Blue	No observed ground cracking or ejected liquefied material
Green	Minor ground cracking but no observed ejected liquefied material
Light Orange	No lateral spreading but minor to moderate quantities of ejected material
Dark Orange	No lateral spreading but large quantities of ejected material
Red	Moderate to major lateral spreading; ejected material often observed
Dark Red	Severe lateral spreading; ejected material often observed

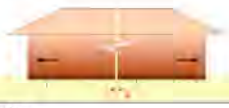
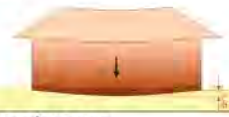
Dwelling Foundation Damage Categories			
Type of Damage	Minor	Moderate	Major
Stretching 	0 to 5mm	5 to 30mm	>30mm
Hogging 	0 to 20mm	20 to 50mm	>50mm
Dishing 	0 to 20mm	20 to 50mm	>50mm
Racking/Twisting 	0 to 10mm	10 to 30mm	>30mm
Tilting 	0 to 20mm	20 to 50mm	>50mm
Abrupt Differential Movement 	0 to 10mm	10 to 20mm	>20mm
Global Settlement 	0 to 50mm	50 to 100mm	>100mm

Figure 2. Liquefaction-induced land damage and dwelling foundation damage inspection criteria. For the mapping of the September 2010 earthquake, a single 'orange' category was used, but this was split into the light and dark orange categories for mapping of the February 2011 and subsequent earthquakes.

Most of the ejected liquefied material was generally removed and major cracks filled (but not repaired) between each of the events. The qualitative land damage mapping therefore, generally recorded the incremental effects of each earthquake. However, there are likely some effects from previous events that influenced the land damage observed after later events, such as the effects of unrepaired cracks on the integrity of the non-liquefied crust.

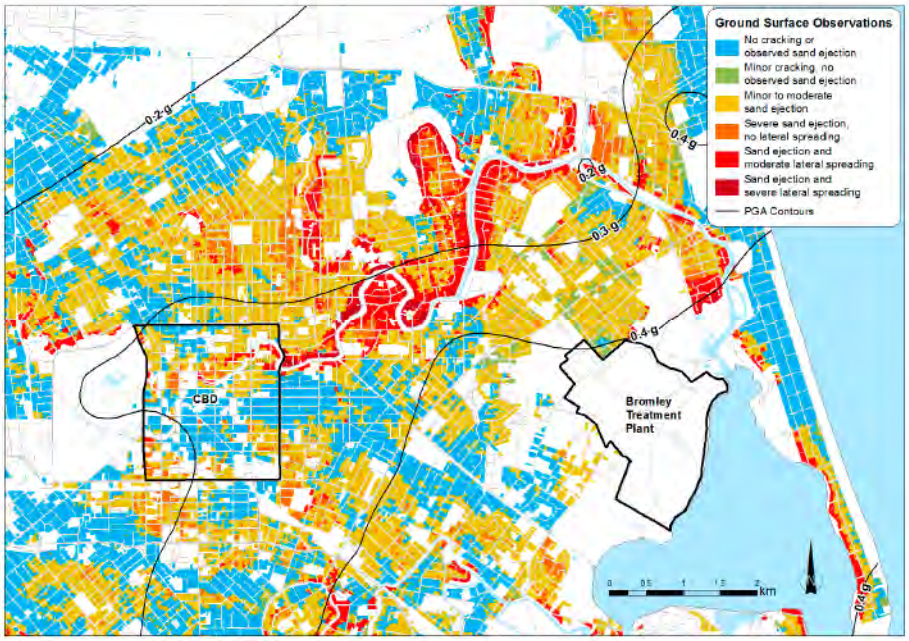


Figure 3. Liquefaction-induced land damage observations across Christchurch after the February 2011 earthquake, with the February 2011 magnitude-weighted *PGA* contours overlaid based on Bradley & Hughes (2012).

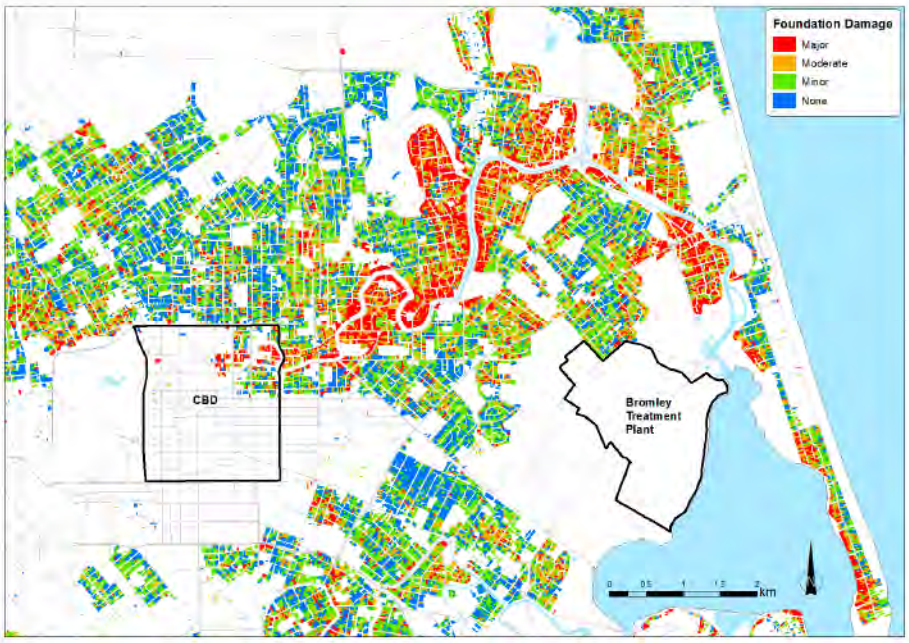


Figure 4. Dwelling foundation damage observations in Christchurch after the December 2011 earthquake. Foundation damage is cumulative due to earthquakes from September 2010 to December 2011.

The residential foundation deformation dataset was based on assessments of the cumulative observed foundation deformation from all previous events prior to the inspection date. Comparison of Figures 3 and 4 show that areas with a high density of major foundation damage coincide generally with areas of more severe observed liquefaction-induced land damage (i.e., the areas where there were large quantities of sand ejected or severe lateral spreading). By contrast, areas with a low density of foundation damage generally coincide with areas with none or minor observed land damage. This shows, not surprisingly, that land damage resulting from liquefaction and residential dwelling foundation deformation damage are strongly correlated.

As a result of the damage from the Canterbury earthquake series, the NZ government classified residential land in Canterbury into various zones and categories (Rogers et al. 2013). The residential Red Zone is land identified by the Canterbury Earthquake Recovery Authority (CERA) where the repair and rebuild process is not practical because the required land repair and improvement works would be difficult to implement, prolonged, and disruptive for landowners. Owners with insured properties in the residential Red Zone were able to sell their properties to the NZ government to manage the withdrawal process (Rogers et al. 2013). The criteria used for defining areas of residential Red Zone were:

- Significant and extensive area wide land damage
- The uncertainty of success of engineering solutions in terms of design, commencement, and implementation given the on-going seismicity.
- Repair and rebuild would not be cost effective because they would be highly disruptive and protracted for landowners, and
- The health or wellbeing of the residents was at risk from remaining in the area for prolonged periods.

The balance of the inspected residential land, which was not classified as residential Red Zone, was further categorized by the Ministry of Building, Innovation & Employment (MBIE) into three technical categories (TC) to assist with the rebuilding of homes on the flat land. This categorization of land was performed on the basis of the mapped land damage for the September 2010 and February 2011 earthquakes after normalization by each event's respective magnitude-weighted *PGA* contours (Bradley & Hughes 2012). The location of the technical categories land areas (which are shown on Figure 5) differentiate the levels of specific geotechnical investigation and foundation design options that are required to address the potential liquefaction issues described in Table 1.

Table 1. MBIE Technical Categories for Rebuilding Christchurch (MBIE. 2012)

Technical category	Description
TC1	Liquefaction damage is unlikely in future large earthquakes. Standard residential foundation assessment and construction is appropriate.
TC2	Liquefaction damage is possible in future large earthquakes. Standard enhanced foundation repair and rebuild options in accordance with MBIE Guidance are suitable to mitigate against this possibility.
TC3	Liquefaction damage is possible in future large earthquakes. Individual engineering assessment is required to select the appropriate foundation repair or rebuild option.

As of March 2013, the land damage mapping was supplemented by an extensive geotechnical site investigation program that included approximately 7,500 CPT, 1,000 boreholes with SPTs, geophysical testing, and piezometers. The number of investigations will continue to increase as the rebuilding of Christchurch progresses. Subsurface data are available through the CERA geotechnical database: https://canterburygeotechnicaldatabase.projectorbit.com. The CPT soundings in conjunction with conventional liquefaction triggering methods have been used as the primary tools to assess the depth of the critical layer for liquefaction triggering and to derive parameters representing liquefaction vulnerability. The CPT locations in Christchurch of all CPT greater than 5m depth are shown in Figure 5. The spatial distribution of geotechnical investigation data (including CPT) are concentrated in the TC3 areas where ground investigations are required for foundation design purposes (MBIE, 2012).

Figure 6 shows the location of piezometers used to develop a model of the median depth to groundwater surface across the Christchurch area (van Ballegooy et al., 2013). Offsets were then made to the median groundwater surface to construct inferred groundwater surfaces at the time of each earthquake based on measured deviations from the median depth of existing monitoring well locations (Tonkin & Taylor, 2013). The groundwater model was used to estimate the hydrostatic pore water pressures that were used in the liquefaction triggering analyses at each CPT site to calculate various liquefaction vulnerability parameters.

AVAILABLE LIQUEFACTION VULNERABILITY INDICATORS

Ishihara (1985) published observations on the protective effect of an upper layer of non-liquefied material against the effects of liquefaction at the ground surface. He plotted material observations of ground failure for sites using the thickness of the underlying liquefied (H_2) and the thickness of the overlying non-liquefied surface layer (H_1), often referred to as 'the

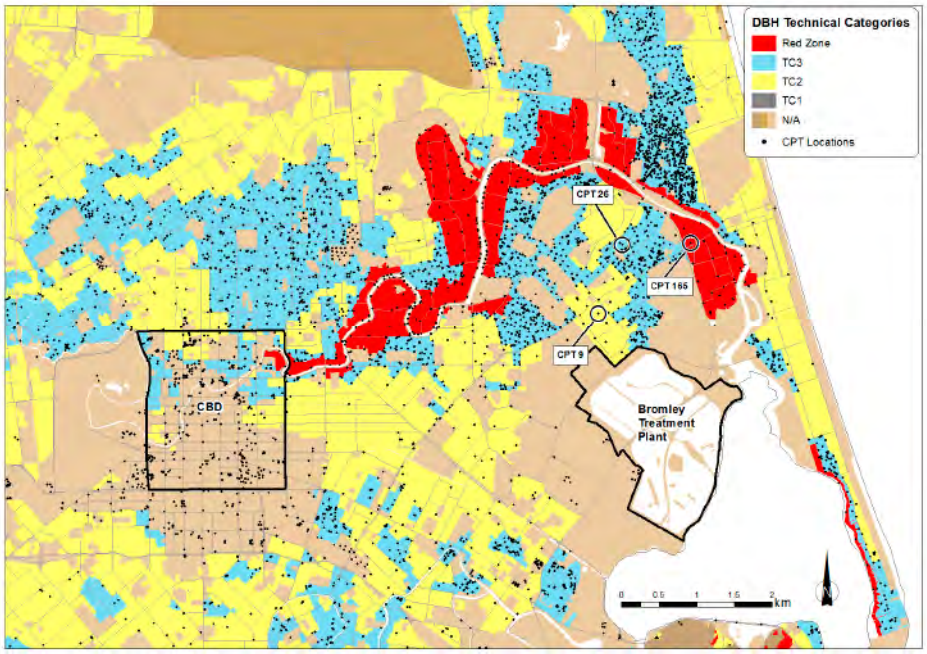


Figure 5. Distribution of CPT used in this study along with areas for the MBIE technical categories TC1, TC2, and TC3 and the CERA residential Red Zone ("no re-build zone").

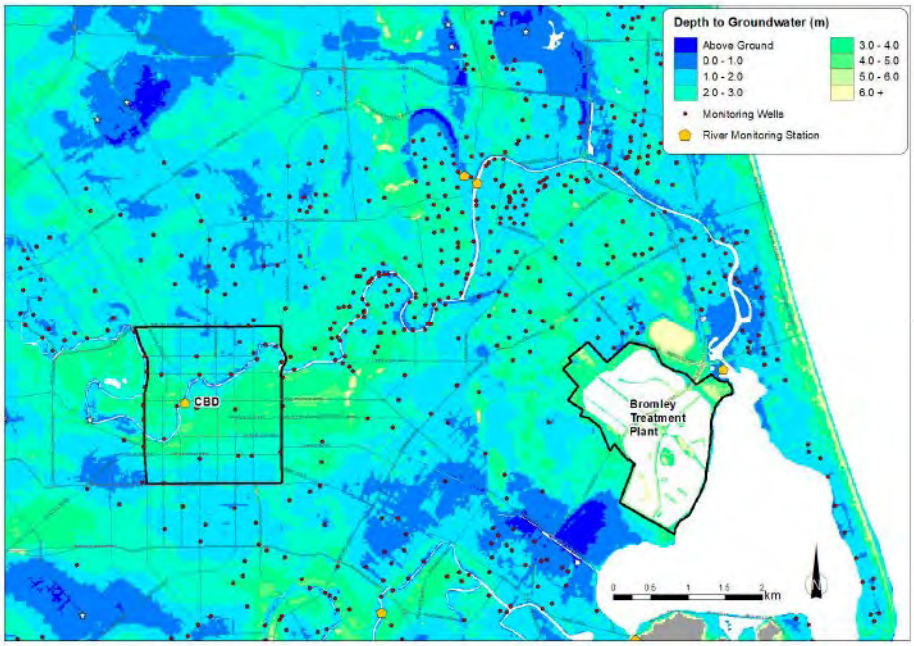


Figure 6. Estimated depth to the groundwater surface immediately prior to the February 2011 earthquake with the layout of piezometers used to construct the groundwater surface.

crust' (Figure 7). Ishihara's work was based on observations from two earthquakes with limited ranges of ground accelerations. Boundary curves were defined that separated those sites which had manifestations of liquefaction at the ground surface from those sites that did not. The Ishihara (1985) liquefaction-induced ground damage plot is only applicable at level ground sites with no free-face. It has often been used in engineering practice with judgment to assess the likelihood of liquefaction-induced ground damage and the potential consequences of liquefaction occurring at depth at a level site.

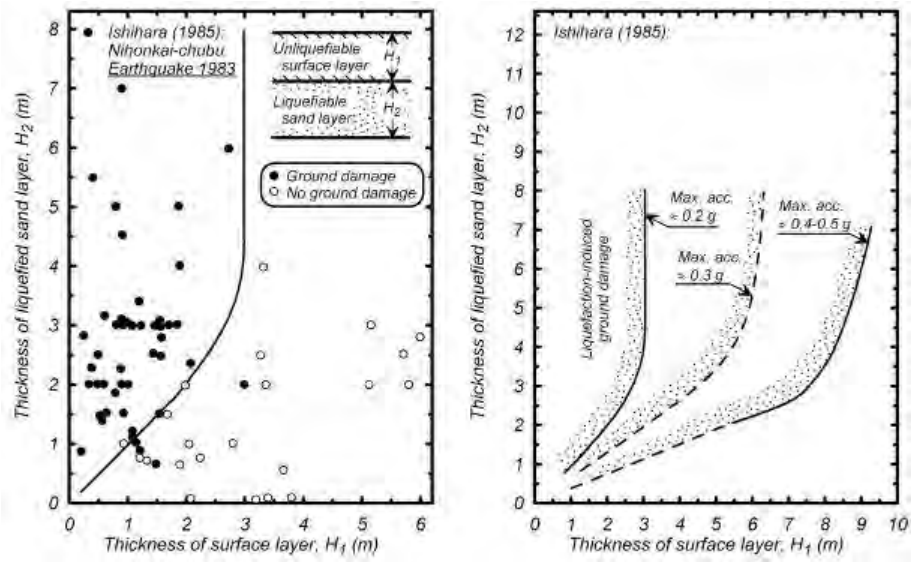


Figure 7. Combinations of non-liquefied surface layer thickness, H_1, liquefied layer thickness, H_2, and peak ground surface acceleration that distinguish between liquefaction-induced ground damage and the absence of such damage (Ishihara, 1985; *figures reprinted from Idriss & Boulanger, 2008*).

Youd & Garris (1995) added data to the Ishihara (1985) plot and showed that it captured their dataset from 13 additional earthquakes. Their conclusions were that the Ishihara bounds for sites not susceptible to liquefaction-induced ground damage appear to be valid but that these bounds were not always reliable for predicting ground surface disruption for sites prone to liquefaction. Both studies indicated that the crust typically had a critical thickness beyond which surface manifestations of liquefaction were unlikely regardless of the thickness of underlying liquefied material (H_2). These studies did not directly measure damage to structures, but instead considered only whether evidence of liquefaction was observed at the

ground surface. The conclusion drawn was that an upper crust of non-liquefiable material has a beneficial effect in mitigating the damaging effects of liquefaction at the ground surface.

The vulnerability of sites to liquefaction was also considered by Iwasaki (1982) and subsequently by Juang (2005a and 2005b). Iwasaki's Liquefaction Potential Index (*LPI*) is a measure of the vulnerability of sites to liquefaction effects. As defined in further detail in the next section, *LPI* is the summation of liquefaction severity in each soil layer, which in turn is a function of the Factor of Safety for liquefaction triggering (*FoS*), weighted by a depth factor that decreases linearly from 10 to 0 over the top 20 m. The resulting *LPI* varies between 0 and 100 (representing negligible to high vulnerability to liquefaction-induced ground damage). The *LPI* uses a liquefaction triggering methodology, which incorporates the soil density and soil profile (which is inferred from the CPT in this study), depth to groundwater, and shaking severity represented by the Cyclic Stress Ratio (*CSR*) (Idriss & Boulanger 2008). It addresses a multi-variate problem in terms of a single parameter.

There is little additional work that relates the quality or performance of the non-liquefied crust in relation to damage of structures and foundations. Work by Cascone & Bouckovalas (1998) and Bouckovalas & Dakoulas (2007) considers the capability of a crust of undrained fine-grained soils overlying liquefied material to support loads from shallow strip or pad foundations. A critical layer thickness is defined where the theoretical foundation bearing capacity failure surface occurs completely within the crust of non-liquefied material. Observations in Christchurch indicate that the crust materials do not typically exhibit an un-drained response, because the crust is composed primarily of silty sand. Thus, the results of these studies are not directly applicable in this case.

All relevant published assessments of liquefaction vulnerability rely on identifying which layers of soil are likely to liquefy within a soil profile under the specified level of earthquake shaking. Three current CPT-based triggering methods have been considered in this study, all of which have evolved from the Seed & Idriss (1971) simplified method that compares Cyclic Stress Ratio (CSR) to Cyclic Resistance Ratio (CRR) to provide a factor of safety (FoS). The methods considered are 1) Robertson & Wride (1998) as described in Youd et al. (2001), 2) Seed et al. (2003) as presented in Moss et al. (2006), and 3) Idriss & Boulanger (2008).

The results of the LPI and LSN analyses presented in this paper are insensitive to the selection of the CPT-based method used in this application. Results using the Idriss & Boulanger (2008) are presented in this paper. The Idriss & Boulanger (2008) method requires a fines content for each layer. As thousands of CPT soundings required analysis, an

automated procedure was utilized wherein the apparent fines content calculated in accordance with Robertson & Wride (1998) was utilized to estimate the fines content of the soils. Additional work is warranted to investigate fully the effects of this assumption.

VULNERABILITY PARAMETERS

For each CPT, the following three liquefaction vulnerability parameters were calculated:

1. The thickness of the non-liquefied crust at the ground surface (H_1) and the cumulative thickness of liquefied layers (H_2) in accordance with Ishihara (1985) over the top 10 m of each deposit where CPT were advanced to a depth of 10 m.
2. Liquefaction Potential Index (*LPI*) calculated in accordance with Iwasaki (1982):

$$LPI = \int_0^{20} \mathrm{F_1 W}(z)\,\mathrm{dz} \qquad (1)$$

   where $W(z) = 10 - 0.5z$, $F_1 = 1 - FoS$ for $FoS < 1.0$, $F_1 = 0$ for $FoS \geq 1.0$, and z is the depth below the ground surface in meters.

3. Liquefaction Severity Number (*LSN*), a new parameter developed in this study to evaluate liquefaction-induced land damage, is defined as:

$$LSN = 1000 \int \frac{\varepsilon_v}{z}\,\mathrm{dz} \qquad (2)$$

   where ε_v is the calculated post-liquefaction volumetric reconsolidation strain entered as a decimal, and z is the depth below the ground surface in meters for depths greater than 0.0. In practice, *LSN* is calculated as the summation of the post-liquefaction volumetric reconsolidation strains, each calculated for an underlying soil layer divided by the depth to the midpoint of that layer.

In this study, the Idriss & Boulanger (2008) liquefaction triggering evaluation procedure was used to define the thickness of the liquefiable layer (H2) and for calculating FoS, and the Zhang et al. (2002) procedure was used to calculate the post-liquefaction volumetric strain.

Iwasaki's *LPI* represents an early attempt to develop an index for assessing the vulnerability of land subjected to liquefaction. Its value is between 0 (representing no liquefaction vulnerability) and 100 (representing extreme liquefaction vulnerability). *LPI* provides a straightforward method for assessing the vulnerability of sites, with published ranges of values indicating the severity of liquefaction. Sites with an *LPI* of more than 5 have a high liquefaction risk, and sites with *LPI* greater than 15 indicate very high risk (Iwasaki, 1982). Toprak and Holtzer (2003) indicated similar *LPI* values based on observations from

the 1989 Loma Prieta earthquake. Potentially liquefiable layers only contribute to the *LPI* when their calculated *FoS* falls below 1.0. As *FoS* decreases, it provides a higher contribution of the calculated *LPI*. While the *LPI* is a useful parameter that captures important aspects of liquefaction vulnerability, this study identified some limitations of *LPI,* which are discussed later.

In the current study, an alternative indicator of liquefaction vulnerability, the Liquefaction Severity Number (*LSN*), was developed and employed. The value of *LSN* is theoretically between 0 (representing no liquefaction vulnerability) to very large number (representing extreme liquefaction vulnerability). The hyperbolic depth weighting function ($1/z$) in Equation (2) can yield a very large value for *LSN* at shallow depth. While this aspect is important for the general application of *LSN* and is being investigated, it does not appear to have practical implications for the use of *LSN* in this study. Very large *LSN* values can only be calculated when the groundwater table is very close to the ground surface and soil layers immediately below the ground surface liquefy. The groundwater levels shown in Figure 6 are predominantly at depths that do not generate very large *LSN*. In this study, *LSN* values are typically between 0 and 100.

LSN is an extension of the *LPI* philosophy. It attempts to quantify the effects of liquefaction and consequent land damage using volumetric strains (adopted in conventional settlement calculations, e.g., Zhang et al. (2002) which is a CPT-based method extending the work of Ishihara & Yoshimine (1992)) in conjunction with depth weighting by a hyperbolic function ($1/z$) rather than a linear reduction. The hyperbolic function gives much greater weight to liquefaction at shallow depths (as compared to that suggested by *LPI*), and considers shallow liquefaction to be the key contributor in the overall damage to land and relatively light residential buildings supported on shallow foundations. This inference was supported by general observations during the liquefaction-induced land damage mapping, particularly the observation that ejection of liquefied material and loss of crust integrity tended to result in significant differential settlements, with various forms of severe ground distortion, cracking, and fissuring.

There are four important differences between the proposed *LSN* parameter and the existing *LPI* parameter:

1. Because *LSN* is based on the volumetric strains used in settlement calculations (e.g., using the CPT-based Zhang et al. (2002) method in this study), *LSN* values are continuously calculated even for *FoS* of greater than one. Thus, *LSN* values start to

increase as excess pore water pressures rise when *FoS* < 2.0, and include continuous smooth transition when *FoS* < 1.0 as implied from the Zhang et al (2002) volumetric strain functions. Conversely, *LPI* accounts for the effects of layers only with *FoS* < 1.0. It will be seen later that *LSN* starts to increase at lower accelerations than *LPI*, because *LSN* reflects the weakening effect of soil layers where *FoS* is approaching one, but is not yet below one.

2. The maximum damage contribution of any soil layer within the deposit is limited by the initial relative density of the soil as represented by CPT tip resistance. This is implied by the Zhang et al (2002) and Ishihara & Yoshimine (1992) volumetric strain relationships used in the settlement calculations. In these relationships a limiting volumetric strain is eventually reached, which is a function of the soil's relative density and not a function of the seismic demand. Conversely, the *LPI* parameter continues to increase with increasing *PGA* because it is a direct function of *FoS,* which continues to decrease as the seismic demand increases.

3. Liquefying layers with a lower relative density are expected to develop larger strains which in turn will result in larger damage at the ground surface as compared to the effects of liquefaction from a layer with higher relative density. With *LSN* the calculated strain value is used as a damage index that includes the effects of strength loss and the potential for soil ejecta rather than as an index purely for settlement calculation. By contrast, for a calculated *FoS*, *LPI* provides the same value irrespective of the relative density of the soil. This approach erroneously indicates that the consequences of liquefaction are not related to the relative density of the liquefied soils for a given *FoS*. Because *LPI* does not explicitly address the relationship between relative density and *FoS*, it should be less successful in differentiating between the damage potentials of sites with different densities of soil.

4. As discussed previously, *LSN* places greater importance on the thickness of the non-liquefied crust when the groundwater table is close to the ground surface through the use of the hyperbolic depth weighting function. *LSN* suggests that shallow liquefaction is significantly more damaging for land and surface structures than deep liquefaction relative to the contribution of shallow and deep layers in *LPI*.

The available CPT results (around 5,500 suitable soundings from the total of 7,500 tests) have been analyzed using the seismic demand model developed by Bradley & Hughes (2012) for the 4 September 2010, 22 February 2011, 13 June 2011, and 23 December 2011

earthquakes based on the strong motion records recorded by the NZ GeoNet (www.geonet.org.nz) and the pre-earthquake groundwater levels defined in the regional groundwater model described by van Ballegooy et al. (2013) and Tonkin & Taylor (2013). *LSN* and *LPI* values were calculated for each CPT site using the different triggering methods outlined earlier combined with deformation calculations from Zhang et al. (2002, 2004). Both the Ishihara & Yoshimine (1992) and Zhang et al. (2002) volumetric densification methods were assessed in this study. However, only the results of the Zhang et al. (2002) method are presented in this paper. This method provided for better differentiation of trends in the observed damage.

The results for the February 2011 earthquake were initially used to prepare a plot similar to that of Ishihara (1985), using an equivalent two layer system, where H_1 is the crust thickness and H_2 is the cumulative thickness of liquefied layers (CTL) in the top 10 m. The plot shown in Figure 8 was developed using data from the upper 10 m of the CPT soundings for all CPT that were advanced more than 10 m below the ground surface. The Ishihara (1985) methodology was difficult to apply, because the crust thickness H_1 and the thickness of the underlying liquefying soil layer, H_2, are difficult to define in a soil profile where there is often more than one liquefying soil layer among non-liquefying layers. By assuming the cumulative thickness of liquefying layers = H_2, the data indicated that almost all H_1 crust thicknesses were typically lower than the required crust thickness to protect against surface liquefaction effects defined by Ishihara (1985). Therefore, sites with both surface manifestations of liquefaction and no surface manifestations of liquefaction plot on top of each other or to the left of the threshold curves shown in Figure 8 with no clear dividing line for the Christchurch damage datasets. Thus, in this application, this plot was not useful.

LPI AND *LSN* PARAMETERS CALCULATED FOR REPRESENTATIVE CPT

Three CPT from eastern Christchurch (whose locations are shown in Figure 5) have been presented for illustrative purposes (see Figure 9). Each CPT is fairly representative of the CPT soundings found in each of the MBIE/CERA TC2, TC3, and residential Red Zones in this area of Christchurch. These representative CPT results have been assessed using the seismic demand from the 4 September 2010, 22 February 2011, and 13 June 2011 earthquakes using the corresponding groundwater depth immediately prior to each event. The calculated *FoS* against liquefaction triggering for each CPT profile is shown for the three events in Figure 9.

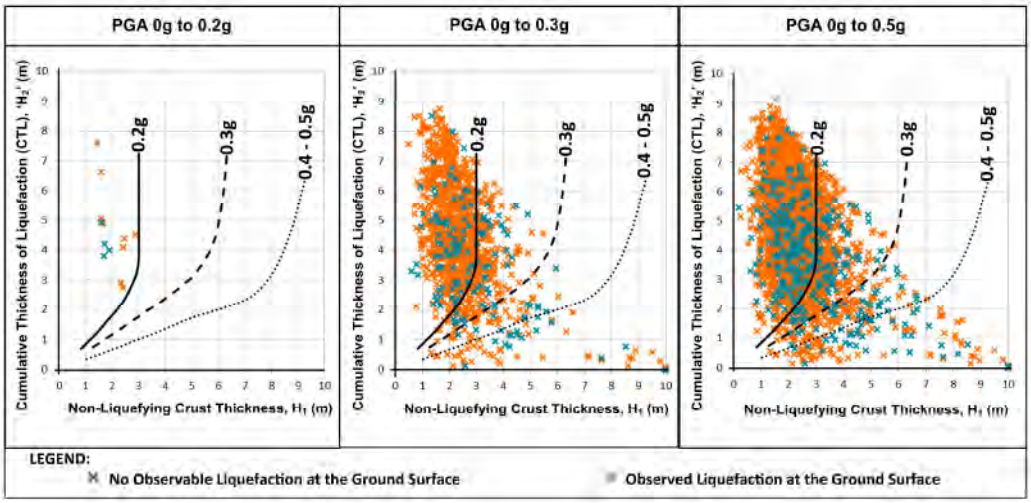


Figure 8. Christchurch earthquake data (adjusted to M_w7.5) superimposed on Ishihara (1985) plot of cumulative thickness of liquefying soil (CTL) in the upper 10 m of the CPT profile (assumed to be equal to H_2) vs. non-liquefying crust thickness (H_1) for the magnitude-weighted *PGA* experienced at each CPT site for the February 2011 earthquake as estimated by Bradley & Hughes (2012).

Figure 10 then shows the contribution of each layer to the distribution of the calculated *LPI* and *LSN* liquefaction vulnerability parameters. The plots in Figure 10 show the difference in contribution from shallower layers for *LSN* compared to *LPI*. Figures 9 and 10 show that the February 2011 earthquake event caused the lowest factors of safety, and the highest *LPI* and *LSN* values in the soil profile at all three CPT locations; whereas, the September 2010 event caused the lowest *LPI* and *LSN* values.

Table 2 summarizes the calculated *LPI* and *LSN* values for each event for the three CPT as well as the observed land damage at the respective sites. The calculated *LSN* and *LPI* values were highest in the 22 February 2011 earthquake (corresponding to the earthquake with the highest seismic demand) and lowest for the 4 September 2010 earthquake (corresponding to the earthquake with the lowest seismic demand). This is consistent with the magnitude-weighted *PGA* experienced during these events and generally fits the liquefaction mapping observations. The calculated *LPI* and *LSN* values are highest at the residential Red Zone site and lowest at the TC2 site, and show a broadly consistent correlation with liquefaction-induced land damage observations for the three sites.

The land damage observed near the residential Red Zone CPT after the June 2011 earthquake was greater than the land damage observed after the February 2011 earthquake. However, Figure 9 indicates that only the top 8 m of soil material likely liquefied during the

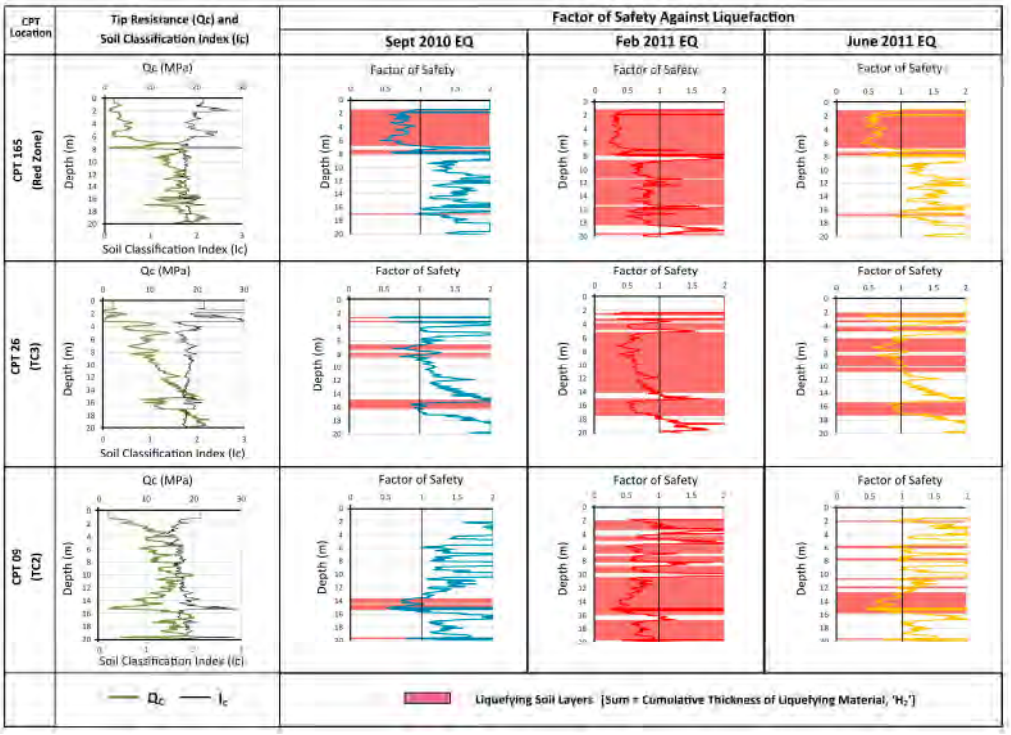


Figure 9. CPT results and liquefaction triggering *FoS* profiles for representative CPT in TC2, TC3 and residential Red Zone areas of eastern Christchurch.

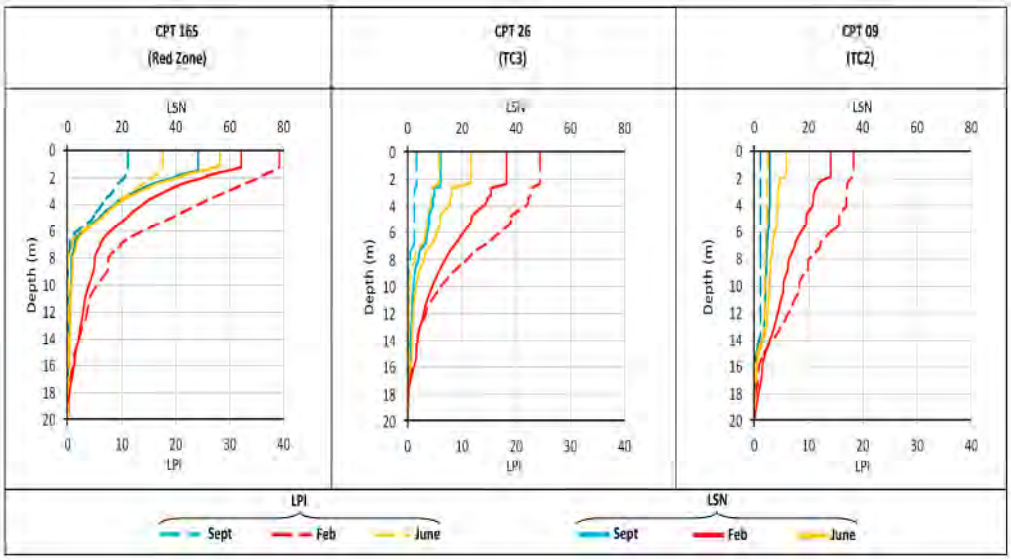


Figure 10. Relative contributions liquefiable layers to the calculation of *LPI* and *LSN* for TC2, TC3 and residential Red Zone CPT.

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September 2010 and June 2011 earthquakes, whereas the top 18 m liquefied during the February 2011 earthquake. Therefore, more land damage would be expected near this CPT for the February 2011 event compared to the other event, as indicated by both the *LPI* and *LSN* values shown in Table 2. One possible explanation is that the crust was damaged and compromised from the February event, resulting in weaknesses and preferred pathways that allowed more sand ejecta to reach the ground surface during the June 2011 earthquake, hence increasing the severity of the manifestations of liquefaction.

Table 2. Comparison of land and foundation damage with *LPI* and *LSN* values for TC2, TC3 and Residential Red Zone at the three representative CPT soundings.

CPT		**CPT_165**	**CPT_26**	**CPT_9**
Location		**Residential Red Zone**	**TC3**	**TC2**
Liquefaction -induced land damage category	04 Sept. 2010	Minor to moderate ejecta	No observable land damage	No observable land damage (based on aerial photos only)
	22 Feb. 2011	Minor to moderate ejecta	Minor to moderate ejecta	Moderate ejecta
	13 June 2011	Large quantities of ejecta	Minor cracking. No observed ejecta	Minor cracking. No observed ejecta
Foundation damage category	All events (typical damage)	Major damage	Minor to moderate damage	Minor to moderate damage
***PGA* (g) Magnitude weighted**	04 Sept. 2010	0.16	0.16	0.17
	22 Feb. 2011	0.36	0.32	0.37
	13 June 2011	0.19	0.20	0.22
LPI	04 Sept. 2010	10	1	1
	22 Feb. 2011	39	24	18
	13 June 2011	17	5	2
LSN	04 Sept. 2010	46	11	6
	22 Feb. 2011	62	35	28
	13 June 2011	55	22	11

Figure 10 shows that the *LSN* parameter for the residential Red Zone CPT is not spread too far apart for the seismic demand for the three earthquakes considered. The land damage around the residential Red Zone CPT is broadly similar for the three events (refer to Table 2). In contrast, Figure 10 shows that the *LPI* parameter for the residential Red Zone CPT is much

smaller for the September earthquake and much larger for the February earthquake. These differences in sensitivity of the *LSN* and *LPI* values are partially a result of the strain limiting 'plateauing' behavior of the *LSN* parameter with increasing *PGA* compared to the non-plateauing behavior of the *LPI* parameter with increasing *PGA*. The other reason the *LSN* has a lower calculated range for the three different earthquakes compared to the *LPI* range is because the depth weighting function for the *LSN* parameter places more emphasis on the upper liquefying layers. Therefore, the additional 10 m of the liquefied soil layers in the February event, which is shown in Figure 9, has little additional contribution compared to its effect on the calculated value of the *LPI* parameter. These trends are less obvious for the TC2 and TC3 CPT locations, because the February 2011 earthquake seismic demand triggered additional liquefaction in soil layers near the ground surface, increasing the calculated values of the both the *LPI* and *LSN* parameters.

The response of the calculated parameters *LPI* and *LSN* to variations in *PGA* is shown in Figure 11. It shows that there is a particular sensitivity in the *PGA* range of 0.15 g to 0.25 g for the selected residential Red Zone CPT, where layers within the CPT trace begin to drop below a *FoS* = 1.0. Below 0.1 g for *LSN*, and below 0.15 g for *LPI,* the calculated parameters are insensitive to the *PGA* for the residential Red Zone CPT. Similar trends are also observed for the TC2 and TC3 CPT, but the threshold *PGA* are higher for the TC3 CPT and higher again for the TC2 CPT. *LPI* continues to increase with increasing *PGA*, but the rate of increase in *LSN* steadily decreases with increasing *PGA*, because the contribution to *LSN* is strain-limited with respect to the initial relative density of the soil. As discussed previously, the use of volumetric strains in the *LSN* calculation causes *LSN* to start to increase at lower *PGA* values than *LPI*. Thus, *LSN* accounts for the effects of soil layers where the *FoS* is just above 1.0. The limiting effect of using volumetric strain results in diminishing slopes at high *PGA*, whereas *LPI* continues to increase with increasing *PGA*. The lower relative density of the soil deposits in the residential Red Zone leads to higher *LSN* values than the TC3 and TC2 zones at each *PGA* level. Lastly, the additional weighting of shallow liquefiable soils leads to higher *LSN* values in the residential Red Zone CPT relative to the TC2 and TC3 CPT for the September 2010 and June 2011 events. These observations are as a result of the four identified differences in the *LPI* and *LSN* liquefaction vulnerability parameter features discussed previously.

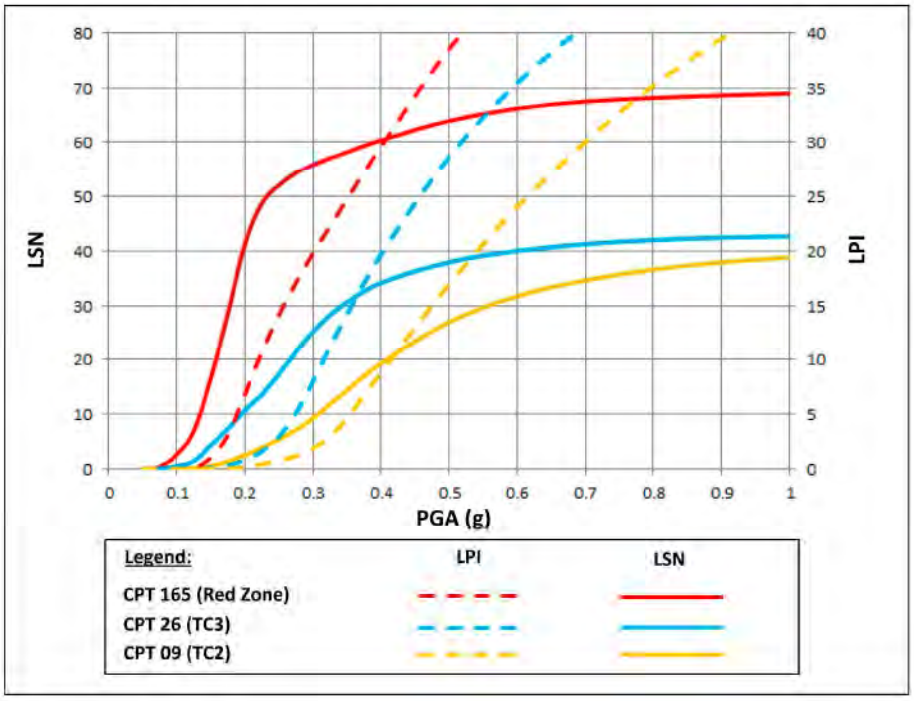

Figure 11. Sensitivity of *LSN* and *LPI* to magnitude weighted *PGA* for residential Red Zone (major damage), TC3 (moderate damage) and TC2 (minor damage) CPT.

CORRELATION OF DAMAGE DATA WITH *LPI* AND *LSN*

The calculated *LPI* and *LSN* parameters have been interpolated between CPT investigation locations and correlated against the mapped land damage and dwelling damage. This typically generated datasets of between 30,000 and 40,000 points, depending on the earthquake event considered and the number of available measured/observed damage attributes. All land damage observations are available through the CERA Canterbury geotechnical database (https://canterburygeotechnicaldatabase.projectorbit.com). The land damage observations in the database were provided courtesy of the NZ Earthquake Commission.

Figure 12 shows a comparison of the observed land damage and observed residential dwelling foundation deformation data plotted against the calculated parameters *LPI* and *LSN* for the September 2010, February 2011, and June 2011 earthquakes as a series of box and whisker plots. The range in calculated values is denoted by the horizontal line showing the minimum value on the left and maximum value of the right. The bar gives an indication of the distribution in calculated values with the left hand and right hand ends of the bar representing

the 25th and 75th percentile values respectively. The median (50th percentile) value is denoted by the black vertical line in each bar. The criteria for the damage categories are defined in Figure 2, and the geospatial distribution of the damage categories are shown in Figures 3 and 4. Furthermore, as an example of an event specific plot, the spatial distribution of calculated *LSN* across Christchurch relating to the February 2011 event is presented in Figure 13, on which the magnitude weighted seismic demand contours for the February 2011 earthquake are overlaid. Figure 13 can be compared directly with the data shown in Figures 3 and 4, which are also related to the February 2011 event. The results in Figures 12 and 13 show that there are generally strong correlations with the *LPI* and *LSN* vulnerability parameters for the different damage categories and weaker correlations with crust thickness.

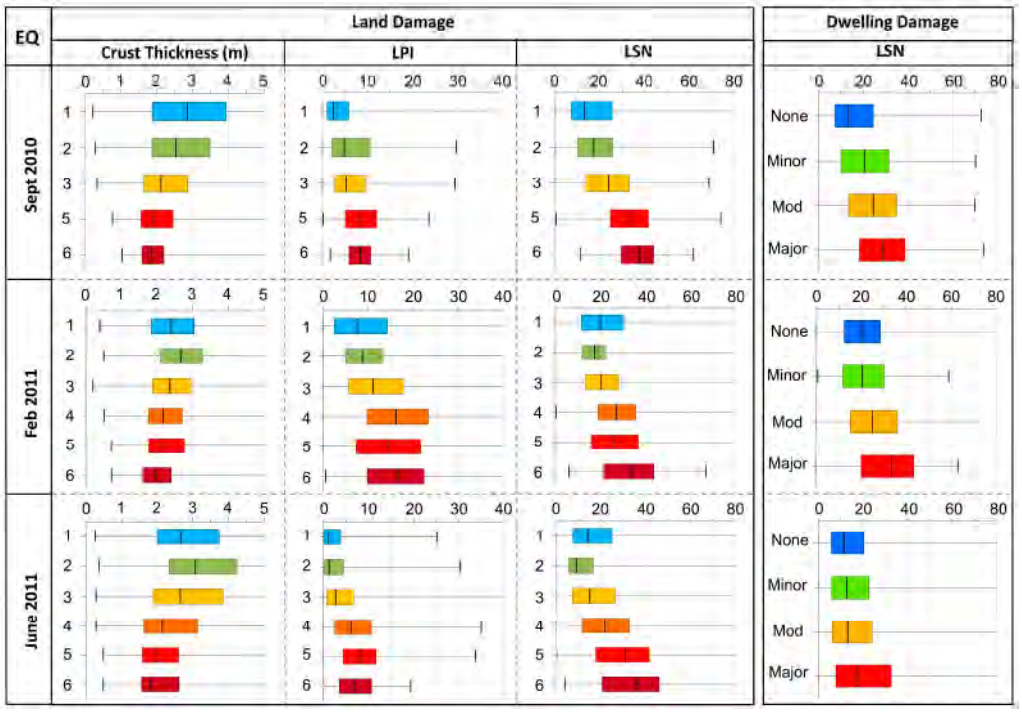

Figure 12. Box and whisker plots of *LPI* and *LSN* based on observed land damage and foundation deformation damage for the 4 September 2010, 22 February, and 13 June 2011 earthquake events. (*For the land damage categories, 1 = blue, 2 = green, 3 = light orange, 4 = dark orange, 5 = red and 6 = dark red - see Fig. 2).*

A statistical data analysis of the results indicates that both LPI and LSN correlate with measured damage to land and house foundations. However, the relationship between the LPI value and the observed damage differs for each event (as shown in Figure 12), which indicates that the LPI correlation with land damage and foundation damage is event-specific

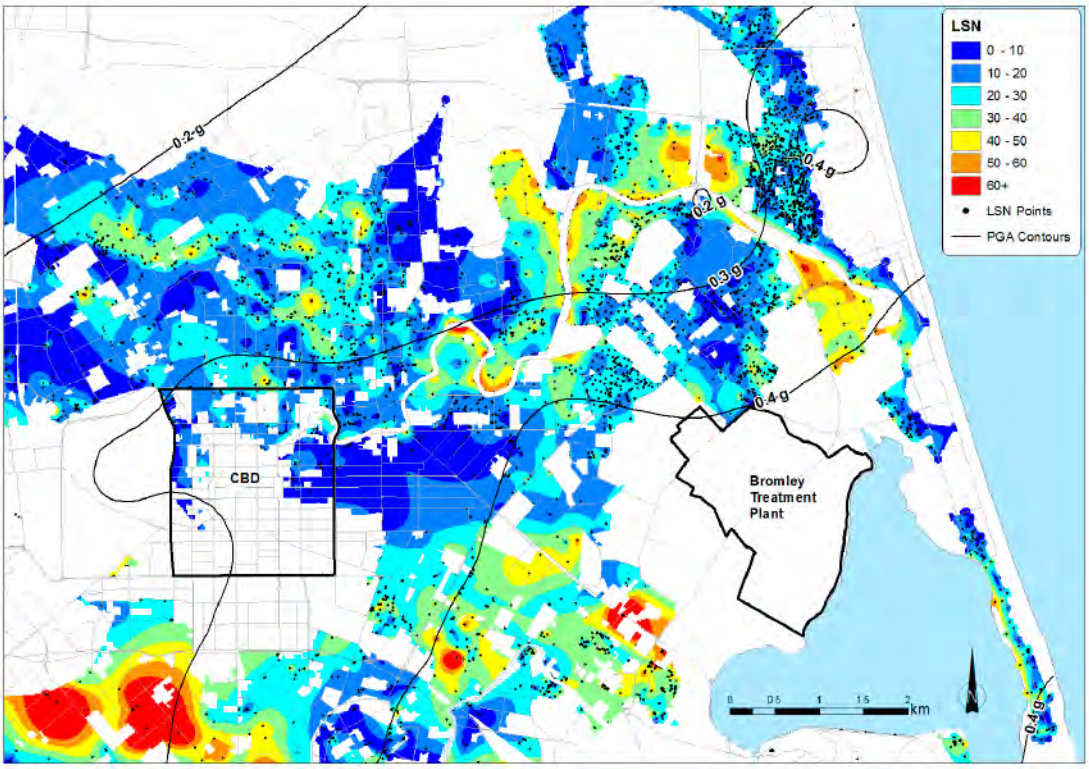

Figure 13. Distribution of Liquefaction Severity Number (*LSN*) across Christchurch for the February 2011 event with the magnitude-weighted *PGA* contours from Bradley & Hughes (2012) overlaid.

for this data set. Differences in the *LPI* values calculated for each of these three events do not track with differences observed in the liquefaction-induced land damage. Conversely, the *LSN* parameter appears to provide a more consistent fit to the mapped land damage over the three different events considered, where a significant range of seismic shaking levels are experienced.

Crust quality has been observed to be another important factor influencing whether liquefaction effects are likely to be consequential at a site. This factor has not been considered in either the existing *LPI* or the proposed *LSN* liquefaction vulnerability parameter, and may account for some of the observed variability in the dataset correlations shown in Figure 12. Additionally, in the areas where the soil has a minimal cumulative thickness of silty soils (e.g., with a soil behavior type index, $I_c > 2.4$) in the upper 10 m, the results are generally consistent with the observed land damage around the region. Conversely, the south-western area of Christchurch consistently had calculated *LPI* and *LSN* parameters that were higher than values which would have indicated the lower observed damage in these areas. The high

LSN and *LPI* values in the lower left corner of the map shown in Figure 13 when compared to the lower amount of observed land and dwelling damage shown in Figures 3 and 4 illustrates a shortcoming of both parameters. This area is characterized by soils that are different in composition (with greater thickness of soil with $I_c > 2.6$), which may be a reason why both the *LPI* and *LSN* liquefaction vulnerability models are not matching the observed land damage. Further work is required to evaluate and understand these discrepancies.

Lastly, the *LPI* and *LSN* parameters are not intended to be an indicator of the vulnerability to the lateral spreading hazard. Therefore, not all the areas with land damage have been identified by the *LPI* and *LSN* parameters because of a superimposed damage from lateral spreading.

CONCLUSIONS

Liquefaction-induced damage to land and residential dwelling foundations has been observed to correlate strongly with the severity of liquefaction effects at the ground surface. The liquefaction-induced damage resulted in differential settlement of the ground and loss of large volumes of foundation soils due to ejection of liquefaction material at the ground surface. The Ishihara (1985) criteria based on the crust thickness could not differentiate between CPT sites where ejection of liquefied material was and was not observed in the residential areas of Christchurch. Moreover, the Ishihara criteria were difficult to apply consistently, because many of the sites in Christchurch have inter-bedded or inter-fingered liquefying and non-liquefying layers.

LPI correlates well with the observed liquefaction-induced damage to land and house foundations. However, the relationship between the *LPI* value and the observed damage differs for each event, which indicates that the *LPI* correlation with liquefaction-induced land damage and dwelling foundation damage is event-specific and produces inconsistent correlations for the three events. Additionally, *LPI* values greater than 5 were calculated at sites where no liquefaction-induced land damage and dwelling damage were observed, which is inconsistent with values corresponding to high liquefaction risk recommended by Iwasaki (1982).

LSN places greater weighting on the contribution of the shallow liquefiable soils to the ground surface damage compared to those layers that liquefy at greater depths relative to *LPI*. In addition, *LSN* is able to capture the non-linear relationship between the strength of shaking

and ground surface damage from liquefaction, beyond a specific level depending on the initial density state of the soil. For the range of seismic demands imposed by the Canterbury earthquake series on the residential areas of Christchurch, *LSN* provides a more consistent correlation with the datasets of observed damage than *LPI*, with a repeatable relationship between the *LSN* value and the severity of observed damage. Additional work is warranted to refine the *LSN* parameter and evaluate its effectiveness in capturing liquefaction-induced land damage in other seismic regions.

ACKNOWLEDGEMENTS

This work would not have been possible without the data obtained by the NZ hazard monitoring system, GeoNet and the extensive remote sensing and ground investigations of land and dwelling damage, sponsored by the NZ Government through its agencies the NZ Earthquake Commission, the NZ Ministry of Civil Defence and Emergency Management and Land Information. The majority of the site investigation data in the geotechnical database have been provided courtesy of the NZ Earthquake Commission and Christchurch City Council. The contributions from the University of Canterbury in developing the seismic models for the various major earthquakes are gratefully acknowledged. We acknowledge the road database in Figures 3, 4, 5, 6 and 13 is supplied by Terralink International Ltd. Rivers, lagoons and coastlines used for those figures licensed under Creative Commons Attribution 3.0 New Zealand and sourced from LINZ. Crown Copyright Reserved. The US National Science Foundation provided research grants that allowed the U.S. authors to contribute to this work.

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POSTER COMPETITION WINNER

Can Chemical Alteration Reduce Long Term Productivity in Deep Geothermal Reservoirs?

C. Baker and I. Duncan

UNIVERSITY OF CANTERBURY
Te Whare Wānanga o Waitaha
CHRISTCHURCH NEW ZEALAND

Geomechanical Issues Associated with the Production of Deep Geothermal Reservoirs

1. Chemistry in Geothermal Energy

The amount of energy produced by a geothermal energy field depends largely on the temperature and fluid flow through the reservoir at depth. Fluid flow is governed by the permeability of the reservoir rock. Chemical alteration arises from an under or over saturation of minerals in pore fluid. This can remove or deposit rock within a reservoir. Such behavior is called dissolution and precipitation and can cause blocks in a reservoir to slip, dilate and contract. See below for a schematic illustration of this process.

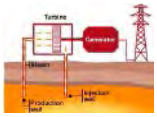
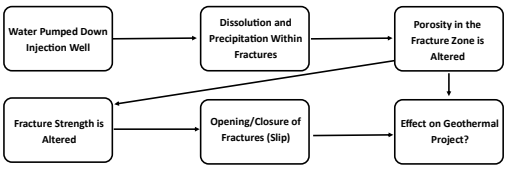


Figure 1. Geothermal Energy Production Process.



2. Project Overview

Project Scope

To investigate the need to consider chemical alteration in geothermal feasibility studies and operation.

Methodology

- Model fractures on a micro scale with PFC3D (discrete element modelling software, DEM) to explicitly account for changes in rock microstructure (dissolution).
- Derive fracture properties (strength and dilation) for various levels of chemical alteration.
- Model a reservoir as a series of moveable blocks using 3DEC (DEM software) with time dependent fracture properties applied.
- Investigate the evolution of reservoir stability and identify any production issues.

Site Details

Reservoir data has been taken from a commonly studied French geothermal field in Soultz-sous-Forêts, France. A 1km thick band of Two-mica Granite rock, at 4km depth below the surface, is the region of interest. Chemical alteration data has been taken from a geochemistry study (using FRACHEM software) conducted on data from the Soultz-Sous-Forêts site.

3. Chemical Alteration on a Micro Scale

The granite rock is represented at the micro scale as a series of balls with appropriate bond strengths and stiffness to replicate rock strength at site (Figure 3). A fracture is then introduced and numerical shear box tests are conducted to determine fracture strength and dilatancy.

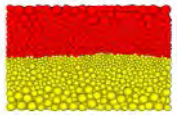


Figure 3. Fractured Granite Specimen Before Shear Box Test (PFC3D image).

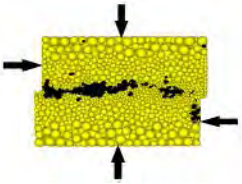


Figure 4. Fractured Granite Specimen During Shearing with Confining Stress (PFC3D image).

- Specimens are sheared at a range of confining stresses to produce a failure envelope for the fracture (Figure 4).
- Normal displacements are recorded during shearing to investigate dilation angle evolution.
- A portion of the balls are removed and tests are repeated to replicate dissolution.

Fracture Strength with Chemical Alteration

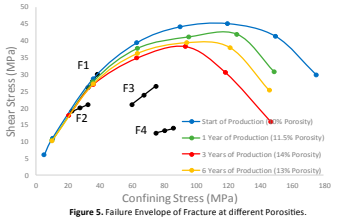


Figure 5. Failure Envelope of Fracture at different Porosities.

Two failure mechanisms apparent:

- **Shear Failure** (confinement < 80MPa)
- **Compaction** (confinement > 80MPa)

Modelled fracture strength was then used as an input for reservoir modelling.

PFC modelling suggested the following implications for reservoir stability:

- Dissolution caused an earlier onset of shear failure.
- Significant strength reduction in compaction failure due to dissolution.
- Seismicity induced by both shear and compaction failures.

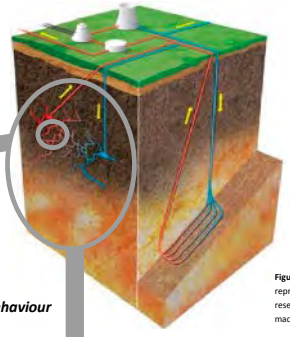


Figure 2. 3D reservoir image (left) is representative of the Soultz-Sous-Forêts reservoir and illustrates the micro and macro scale used.

4. Chemical Alteration on a Macro Scale

The fractures on a macro scale are affected by the distribution of in-situ stresses in the block and the pressures introduced by fluid pumped into the borehole. The behavior of the fractures was modelled by creating a numerical model using 3DEC Itasca software.

Modelling Steps

- The reservoir was modelled as a fractured block (Figure 7).
- Intact rock properties from site were assigned to the reservoir block.
- In-situ and boundary stresses representative of the conditions on site were applied
- Reduced fracture strengths during production were applied according to PFC3D dissolution results.
- A hydrostatic pore pressure field was applied to the fractures.
- Production overpressure was applied to the centre of the block (GPK-2 borehole).

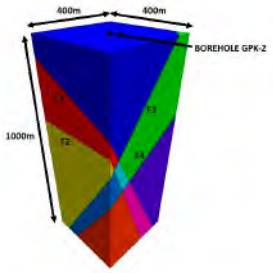


Figure 7. Numerical Reservoir Model with Fractures.

Outputs

A number of outputs were monitored in order to determine the behaviour of the reservoir. These outputs were:

- Normal and shear stresses on fractures.
- Shear slip along fracture surfaces.
- Induced seismicity from fault failure (Figure 8).

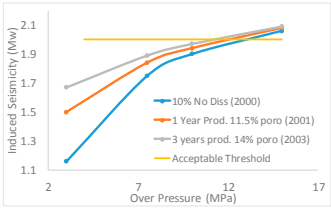


Figure 8. Induced seismicity for various porosity and over-pressure values.

5. Key Results

- Existence of two failure mechanisms for faults at 4km-5km depth. Faults dipping West display shear (dilative) failure and faults dipping SE (subject to σ_H) undergo compaction (contractive) failure.
- Chemical alteration caused a strength reduction in both failure mechanisms, but caused a significant strength reduction in compaction failures. The onset of compaction occurred at lower confining stresses than shear failure.
- Fault-slip modelled on 3DEC induced seismicity was comparable to that recorded at site.
- Dissolution over 3 years of production induced seismicity exceeding tolerance (M2.0).
- To remain within acceptable seismicity, the production flow rate needed to be reduced by 34%.
- Chemical alteration resulted in significant production issues, and should be taken into consideration.

Process of geothermal image retrieved from: http://nothingnerdy.wikispaces.com/GEOTHERMAL+ENERGY
Geothermal Field image retrieved from: http://geothermalinstallation.wordpress.com/

Acknowledgements: The authors would like to thank Cedric Lambert and Roger Nokes for their input and guidance towards this research.

Effect of near-fault ground motion on bridges including soil-foundation-structure interaction

Authors: Darshan Pradhan & Timur Sibaev
Supervisor: Dr. Tam Joseph Larkin

CIVIL AND ENVIRONMENTAL ENGINEERING

THE UNIVERSITY OF AUCKLAND
FACULTY OF ENGINEERING

POSTER COMPETITION RUNNER-UP

Introduction

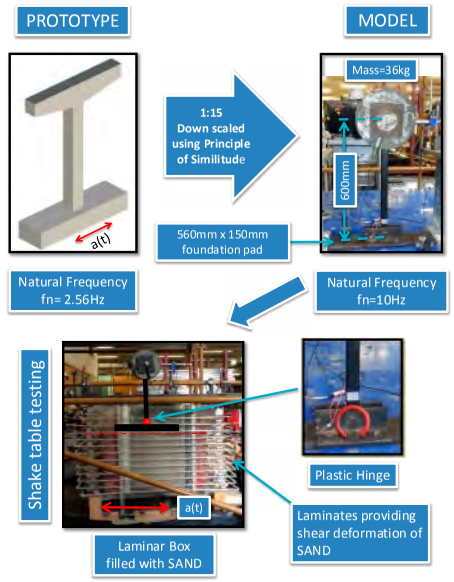
Bridges on shallow foundations are often designed without considering soil-foundation-structure interaction (SFSI). Therefore, plastic hinge is assumed to form at the bridge pier. Forward directivity pulses in near-fault ground motions (GMs) require most of the seismic energy to be dissipated in a few large cycles of motion. This causes significant ductility demand of structure. It is also likely to induce soil plastic deformation and uplift of shallow foundation.

Objectives

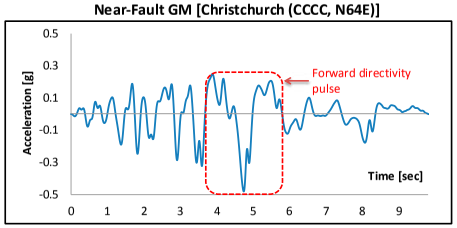
This study investigates SFSI for a bridge pier model subjected to near-fault GMs, in particular:

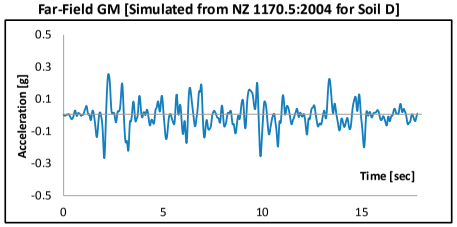
- Top displacement due to plastic hinge rotation and footing rotation,
- Accelerations and displacements of the model on soil with varying stiffness (loose and medium dense (MD) sand),
- Pressure distribution at the foundation-soil interface.

Methodology



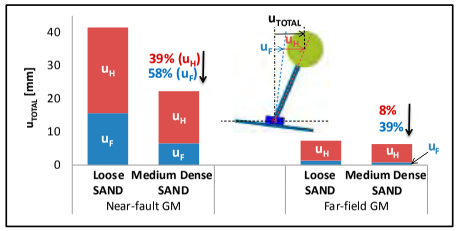
Ground Motions

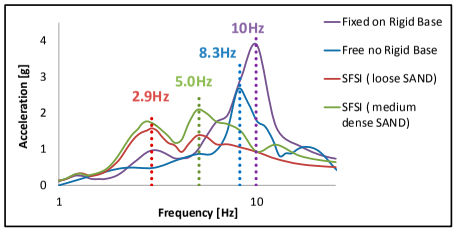




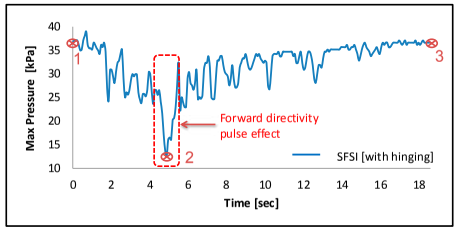
Results of Shake Table Testing

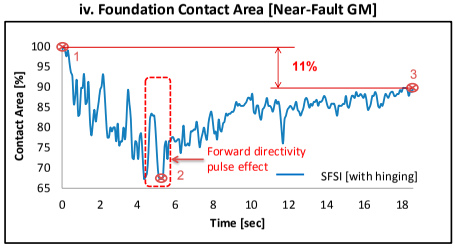
Varying sand density

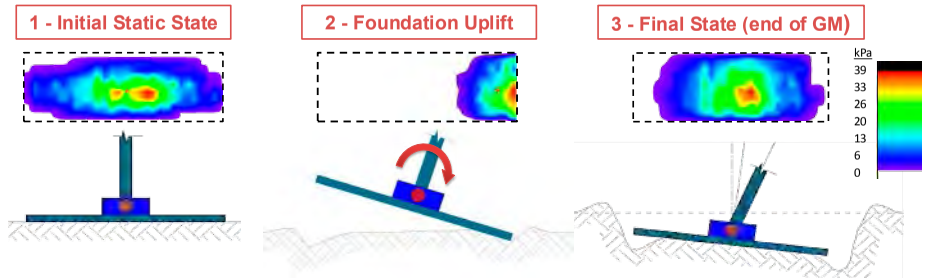




SFSI : Pressure Distribution







Discussion

Varying sand density

- The graph [i] shows that the top total displacement of the model is relatively high for Near-fault GM compared that to a far-field GM. Top displacement due to foundation rotation is decreased significantly with an increase of sand density.
- The graph [ii] illustrates the reduction in maximum spectrum acceleration and dominant frequency for the induced vibration with consideration of SFSI (10Hz → 2.9Hz). Increase in sand density has increased both maximum spectrum acceleration and dominant frequency (2.9Hz → 5Hz).

SFSI : Pressure Map

- Graphs [iii & iv] show that strong forward directivity pulse has caused reduction on maximum pressure and foundation contact area [2], which suggests that foundation uplift has occurred.

Conclusions

Sand density

Increase in sand density results in:

- Increase in dominant frequency and maximum spectrum acceleration of the induced vibration.
- Decrease in total deformation at the top of the model.
- Reduction in top displacement which is dominated by reduction of deformation due to footing rotation.

Near-fault GM

- Causes reduction of foundation contact area, large top displacement and foundation uplift.

Acknowledgements

The authors would like to thank Yuanzhi Chen for her invaluable input and guidance.

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Recommendations for post-disaster geotechnical response for hilly terrain: Lessons learned from the Christchurch Earthquake Sequence

ABSTRACT

Analysis of the geotechnical response to coseismic slope failure during the Canterbury Eearthquake Sequence (CES) indicates that it was well executed but would have been improved with pre-planning. We present a series of recommendations to the government agencies and geotechnical professionals that will allow us to prepare ourselves for responding to future geotechnical disasters. These include preparing ourselves for rapid post-disaster deployment through training, group organisation, and definition of communication protocols. We also recommend developing tools for geotechnical professionals to use during post-disaster response for collecting, analysing and communicating data and life-safety assessments.

INTRODUCTION

The 22nd February 2011 earthquake was the first event in the CES to cause widespread co-seismic rockfall, cliff collapse and loess failure. Rapid deployment of geotechnical professionals was required to undertake life-safety risk assessments and inform on hazard management. This disaster required a large scale assessment of damage with a high demand for geotechnical expertise. A large number of geotechnical practitioners from Canterbury and elsewhere in New Zealand self-mobilised to undertake life-safety assessments.

Due to the occurrence of liquefaction during the 4th September earthquake, the geotechnical response in the flat areas of Christchurch had an established framework within which to operate. Processes for coordinating the response in the hilly terrain, however, developed post-earthquake because there was no pre-existing framework for large-scale slope assessment during a disaster. This led to coordination and execution challenges.

This technical note will present the data collection methodology and key results that capture the successes and challenges we have identified from this experience. We will then provide a series of recommendations that will help the geotechnical community better prepare to respond to future disasters affecting hilly terrain.

METHODOLOGY

The data collection comprised two separate aspects: review of international earthquakes and response systems, and interviews with practitioners involved

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Katherine is an Engineering Geologist with Beca Ltd in Christchurch. Katherine recently graduated with a MSc (Hons) in Engineering Geology. Her research was focussed on post-earthquake risk assessment of earthquake induced landslides, and the management and execution of emergency geotechnical response. Katherine has worked primarily in the Otago and Canterbury regions, and has several years of experience in the civil engineering industry at Opus prior to her time at Beca.

Marlène Villeneuve

Marlène is a Senior Lecturer at the University of Canterbury in the Engineering Geology Programme. She immigrated to Christchurch on 5th September 2010, but decided to stay despite the rocky start. She previously worked in tunnel design and construction in Switzerland, the USA and Australia, having obtained her PhD in tunnelling at Queen's University in Canada. She currently works in rock mechanics applied to tunnelling, geothermal, landslides and seismic amplification with a particular focus on lab testing and numerical modelling.

Thomas Wilson

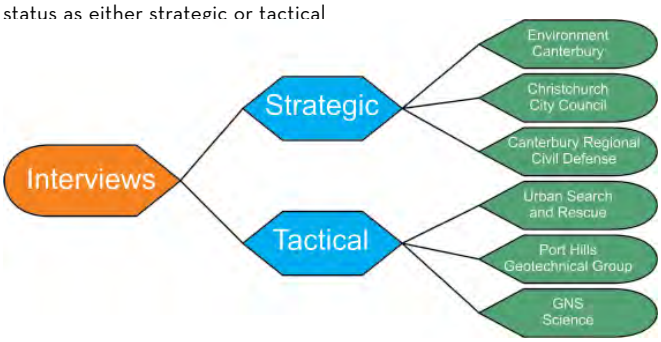
Tom is a Senior Lecturer at the University of Canterbury in the Hazard and Disaster Management Programme. He specialises in natural hazard and risk assessment, focusing on critical infrastructure, primary industries and isolated communities. He also has research and teaching interests in disaster risk management, particularly the role of science to increase disasters resilience.

with the Christchurch Port Hills geotechnical response. We used the data from case studies to assess what our successes were and provide best practice guidance for our recommendations.

We reviewed the response to the 2008 Wenchuan earthquake, China, the 1999 ChiChi earthquake, Taiwan (Goltz et al. 2001; Loh and Tsay 2001; Lin 2006) and the 1994 Northridge earthquake, California (Norton et al. 1994; USGS 1996). The lessons learned from these events show that there are a number of key things that can be done to optimise geotechnical response to disasters. They particularly highlight the value of implementing pre-event planning to guide post-earthquake response. After the ChiChi and Northridge earthquakes, the response to earthquake-induced landslides was encompassed in the government disaster response framework. This enables any response to be executed rapidly post-earthquake, with the roles of governing and tactical organisations clearly defined.

Local and national governmental agencies and many private organisations and institutions formed the geotechnical response to earthquake-induced slope failure in the Port Hills. Interview participants from these organisations were selected for their involvement in the emergency response in the Port Hills. The participants were broken into two groups (Figure 1): strategic level represents those organisations that lead the response, while the tactical level represents those organisations that conduct the response.

Figure 1: Schematic of organisations involved in interviews and their status as either strategic or tactical



Interviews were conducted in a semi-structured format, which involved developing questions guided by the research objectives, and followed a strict ethical approval process. The semi-structured method differs from a structured interview by allowing the interviewer to modify the pace and order of questions, or add further questions as the interview progresses to probe further response from participants (Gillham 2000; Qu and Dumay 2011).

Interpretation of interview results was conducted in several iterations:

1. Results from interview analysis were placed in a coherent timeline of the CES commencing 4th September 2010 until December 2011.
2. Challenges and successes identified were examined to consider the probable causes, which led to developments in the geotechnical response. The response to the CES was compared to the responses to historical international earthquakes to provide supporting evidence or contrast with comparable events.
3. A temporal model of the geotechnical response to the CES was developed to emphasise the evolution of tasks and requirements in the context of post-earthquake coordination of the geotechnical response. Phases within the model were developed to delineate stages within the geotechnical response to allow comparison with historical international earthquakes.

KEY SUCCESSES AND CHALLENGES FROM THE REPONSE

Coordination

The geotechnical earthquake response in the hilly terrain was a situation few geotechnical professionals had been involved in, and, being unforseen, was not integrated with the Civil Defence and Emergency Management (CDEM) response plan. Coordination evolved in the first days and weeks of the response, beginning with the geotechnical professionals self-mobilising and loosely coordinating with the different strategic level organisations involved (Figure 2). In the first week, the Port Hills Geotechnical Group (PHGG) developed and worked with strategic level organisations through channels that were established during the response (Figure 3). At the end of the state of emergency, Christchurch City Council established employment agreements with the PHGG and GNS Science to perform stability analyses in a more conventional fashion (Figure 4).

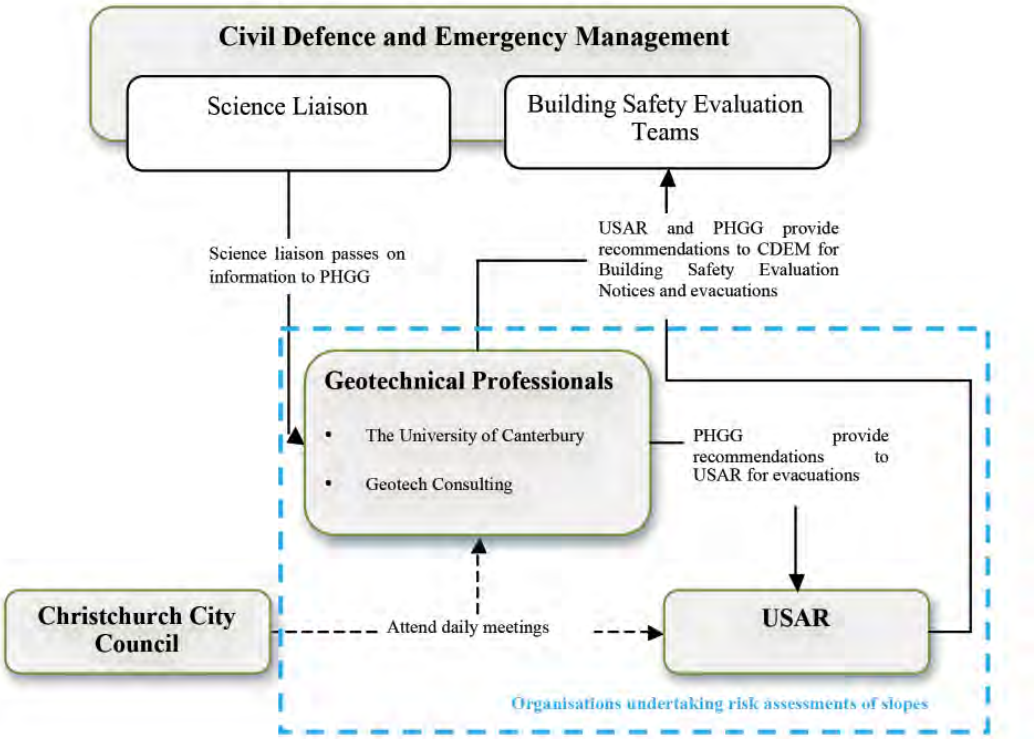


Figure 2: Interactions of organisations in response in Port Hills approximately 48 hours after 22nd February 2011 earthquake during the Canterbury Earthquake Sequence

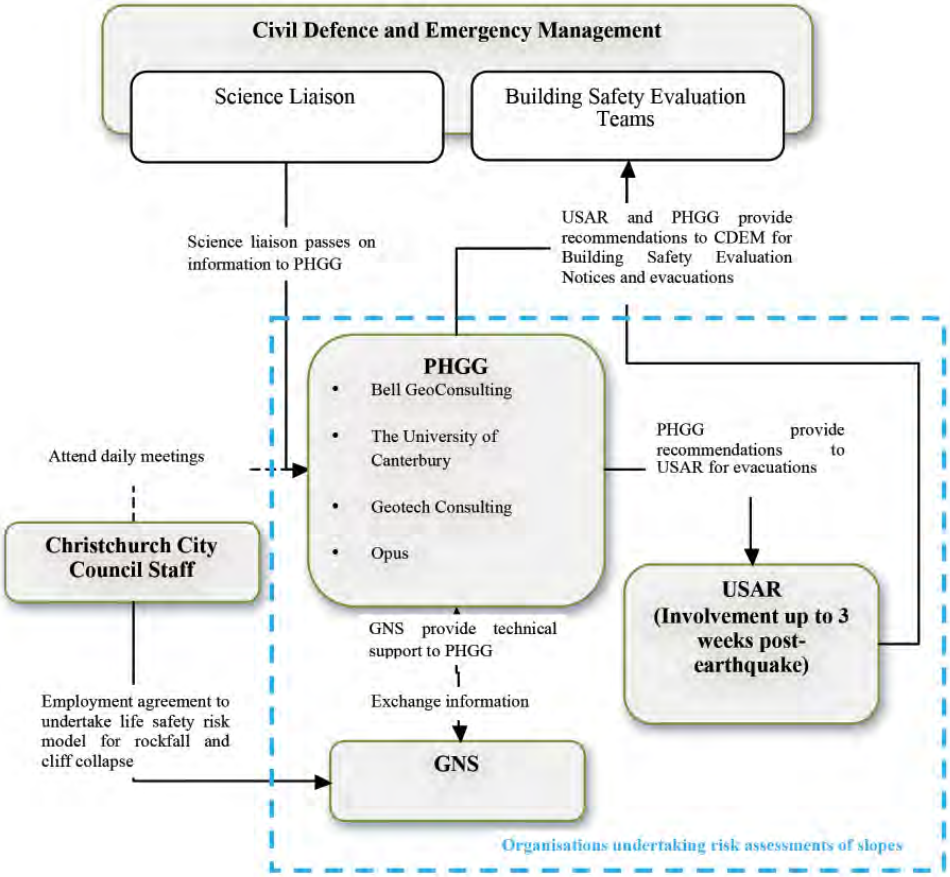


Figure 3: Interaction of organisations in the Port Hills response from approximately one week after the 22nd February 2011 earthquake

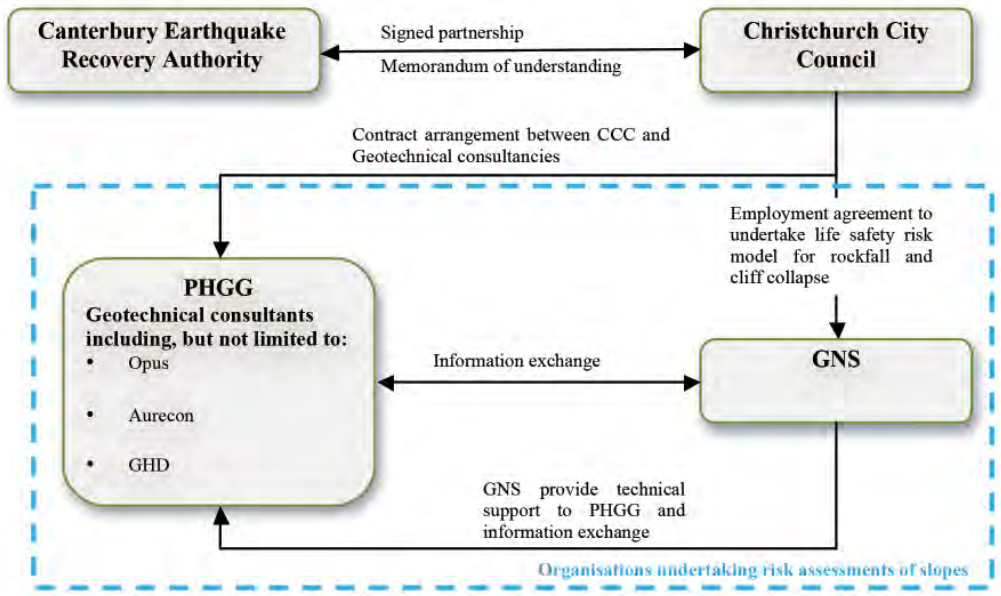


Figure 4: Interactions of organisations in Port Hills response after state of national emergency ceases (30th April 2011) after the 22nd February 2011 earthquake

Community Communication

Dissemination of information to the public regarding the emergency response was poorly addressed and heavily reliant on individual communication between geotechnical professionals and Port Hills residents. Providing information for the public was often an exhaustive task for geotechnical professionals who were highly involved with both emergency response and communication with residents at community meetings and on an individual basis. These professionals had limited previous experience with science communication. Public communication was important because residents needed to understand the risks associated with each of the slope failure hazards. The requirement for communication with residents was identified by the CDEM science liaison who became highly involved in the coordination of pubic communication. Because of the involvement of the science liaison, geotechnical hazard fact sheets were distributed and public meetings were established approximately two weeks after the 22nd February 2011 earthquake.

Port Hills Geotechnical Group

The initial priority of the PHGG was protection of life safety. Geotechnical consultancies were able to work towards common priorities at this time – altruistic values prevailed over competition. Early in the response regular meetings were initiated, first at CDEM response headquaters, later at the Opus offices outside the red cordon. These meetings were used to coordinate the geotechnical response across the Port Hills, provided an opportunity for discussion between consultancies and sectors, and obtain direction from the strategic level. This group provided a support system between geotechnical responders for the sharing of skills, observations, resources and lessons learned.

Sectors

The PHGG divided the Port Hills into 9 sectors, each of which was assigned to a separate consultancy. This facilitated uniform and systematic deployment and organisation of geotechnical assessment teams, divided the workload and decreased duplication of assessments by different consultancies. Coordination through sectors meant that geotechnical engineers or engineering geologists were not responding on an individual basis, rather within consultancies, and ensured that all areas of the Port Hills were included in the assessment. The development of sectors also facilitated communication with the CDEM response centre because it enabled information gathered from the call centre to be communicated directly to the geotechnical professional responsible for a particular area and vice versa. Communication between sectors was important and was undertaken during daily PHGG meetings.

Data Collection

Establishing a balance between formalised reporting and meeting the tactical requirements of the response was a challenge after the 22nd February 2011 earthquake. No formal data collection system existed for geotechnical response and as such observations were recorded as brief notations or verbally communicated to other personnel involved in the response. Although this can be an unreliable form of correspondence, in the immediate

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response there was little time for formalised reporting systems because of the extensive requirement for rapid life-safety protection. To address this issue the following tactics were implemented to improve consistency within the first week of the response: daily morning meetings to discuss developments in data collection, informal peer review through verbal discussions at meetings or on-site, and a standard assessment pro-forma for data collection was developed. The purpose of the pro-forma was to record consistent information and standardise the qualitative risk assessment procedure.

GIS Database

The GIS database was established immediately post-earthquake using local data sources. Its primary function was as a continuous record of sector data, providing information on where had been assessed, and a tool for managing spatial information. It also provided a visual record of high risk areas and facilitated the creation of hazard maps.

Conclusions and Key Recommendations

The response to earthquake induced landsliding was well executed but could have been improved. Through the lessons learned from the Christchurch earthquake sequence we can prepare for future response. Now is the time to use what we have learned from the Christchurch Earthquake Sequence to Prepare for future earthquakes. We can do this by addressing the successes and challenges from our and international experience.

The coordination evolved with the response, but would have been much more effective if a geotechnical response register had been in place. Such a register would allow CDEM to contact the necessary leaders and responders in the immediate aftermath of a disaster. A framework for flow of information would also mean that information would be effectively communicated in both directions.

The high demand for information from Port Hills residents regarding enforcement of evacuations, building use restrictions and risk from slope failure hazards indicates that a strategy for public communication should be implemented. This could include training for geotechnical professionals on the register and identification of a communication protocol.

The Port Hills Geotechnical Group provides a model for development of regional strategies to divide areas into sectors and identify key members of the register who are responsible for leading the response in each sector.

The GIS database was a critical tool for data collection,

CHALLENGES/ SUCCESSES	RECOMMENDATIONS
Coordination	• Create a register of Geotechnical Professionals who are avaliable and trained for emergency response – through IPENZ • Develop a system or framework for mobilisation of those on the register • Indentify key personnel within the register to lead the response
Geotechnical response initially detached from CDEM	• Develop a system to integrate the geotechnical response with CDEM • Make this a component of the CDEM response plan
Public communication	• Provide communication training for geotechnical professionals on the register • A possible model is the Communication Plan – Nelson 2011 Welfare with geotechnical aspects
Group management – personnel and spatial distribution	• Define an adaptive management strategy for response based on the geotechnical aspects and scale of the disaster • Pre-define sectors and sector leaders based on disaster scenarios
Consistency in data collection and slope assessment	• Consistency in data collection and slope assessment • Recommendations for geotechnical response guidelines – similar to Building Safety Evaluation Guidelines or ATC-20 • Develop templates for slope assessment and data collection
Availability of spatial data in aftermath of earthquake	• Facilitate the development of GIS databases for disaster response • Could be based on a national or regional platform that can be expanded when a disaster occurs

Table 1: Summary of challenges and successes identified for the Canterbury Earthquake Sequence and Recommendations for future preparation

interpretation and dissemination. Creating a national strategy for spatial data management during a disaster would make this process much more efficient, especially in the critical initial response. Consistent data collection is possible with the development of geotechnical hazard assessment pro-forma, such as those used for building assessment or the ATC-20 Procedures for Inspection of Geotechnical Hazards (Applied Technology Council 1995) used in California.

We strongly believe that as geotechnical professionals we have a key role to play in better preparing for disasters and the CES has highlighted what we have done well and how we can improve. A national dialogue within our profession and with strategic level organisations is needed for us to implement the recommendations given in Table 1.

ACKNOWLEDGEMENTS

This research was funded by Environment Canterbury. We would also like to thank all those who contributed to this research through their interviews. They cannot be mentioned by name, but this research would not have been possible without their enthusiastic participation. Everyone who worked on this project would like New Zealand to learn from this experience and become more resilient through this process. We would also like to thank Peter Kingsbury at Christchurch City Council for his support and for helping to disseminate the findings.

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Observations from the Design and Construction of Tubular Piles using Driven Precast Concrete Plugs for Bridges

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Dave is an engineering geologist with Beca, based in Christchurch. He completed a BSc (Hons) in Geophysical Sciences at the University of Southampton in 2003 and worked as a surveyor and then geotechnical engineer in the UK before relocating to New Zealand in 2008. Since then he has worked on a range of infrastructure projects across the country, including the SCIRT programme in 2011-2013, involved with a range of geotechnical and geological assessment and design, particularly pile design and construction.

Tom Angus
Tom is a Site Engineer with SCIRT Delivery Team McConnell Dowell Constructors based in Christchurch. After completing a BE (Hons) in Civil Engineering in 2011 Tom joined the SCIRT McConnell Dowell Delivery Team. He has worked on a range of infrastructure projects including drainage, pump stations, roading and bridges.

THE REPAIR OR replacement of earthquake damaged bridges in Christchurch has taken many forms, with the solution selected by the Stronger Christchurch Infrastructure Rebuild Team (SCIRT) being tailored to the particular ground conditions and degree of damage of each.

Here we discuss the challenges and outcomes from the collaborative design and construction of piles for two bridges undertaken by the design team and McConnell Dowell Delivery Team within SCIRT. One of these, Avondale Road Bridge, was repaired by constructing new abutments and utilising the existing piers. The other, Gayhurst Road Bridge, was replaced in its entirety due to more significant damage.

DRIVEN PRECAST PLUG PILES

Driven precast plug piles are a type of 'driven and cast-in-place' pile. A steel casing is installed, generally by vibrating hammer or oscillator, to the desired toe level. A precast concrete plug, of slightly smaller diameter than the inside of the casing, is then lowered or dropped to the base of the casing. The plug is then driven by dropping a hammer through the casing onto the plug. With this method, only the plug is mobilised by driving (the casing remains stationary), meaning that either a smaller hammer weight can be used or more efficient driving results from using the same hammer weight, compared to top-driving or using gravel plugs, due to the smaller effective pile weight.

A more common method of installing driven and cast-in-place piles involves placing a gravel or dry concrete plug in the base of the pile and driving the plug and casing until the required toe level and driving resistance is achieved. The advantage of driving the plug and casing together is that the temporary compression can be directly measured from recording the rebound of the casing. With precast concrete plugs, it is not possible to prevent water ingress to the casing, and therefore in many cases the hammer is falling through water, with associated energy loss.

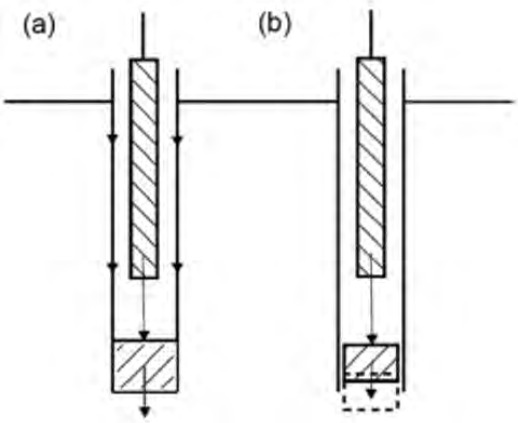

Figure 1: Pile bottom-driving methods: a) Gravel, dry concrete or cast insitu concrete plug; the force from impact of the falling hammer is transferred to the inside of the casing and casing is dragged downwards with the plug. b) Precast concrete plug; only the plug is driven downwards by impact of the falling hammer, the casing remains stationary. The plug is set above the toe of the casing and then driven beyond it.

These piles have been used on a number of projects in New Zealand, primarily rail bridges. One notable project in recent times in New Zealand was the Arahura Road Rail Bridge, completed in 2009. The bridge substructure comprised 1600 mm diameter steel tubular piles with driven concrete plugs.

AVONDALE ROAD AND GAYHURST ROAD BRIDGES

Avondale Road and Gayhurst Road bridges are located in eastern Christchurch, and both experienced significant damage as a result of the 2010-2011 Canterbury earthquakes.

Avondale Road Bridge is a 36 m long, three-span, concrete bridge constructed in 1961. The bridge experienced significant lateral displacement of the abutments due to lateral spreading of the banks of the Avon River. Lateral displacement of the abutment walls was observed to be 150-350 mm towards the river, with rotation of the wall at the base due to propping by the bridge deck. Away from the abutment, the unrestrained river bank was observed to have displaced up to 1.2 m due to lateral spreading. Settlement of the road carriageway immediately behind the abutment walls of around 250 mm occurred relative to the bridge deck, with ground settlement in the surrounding area approaching one metre. Structural assessment determined that the bridge deck and piers could be economically repaired, so only the abutments were replaced; given the permanent deformation of the abutment piles these could not be relied upon for any vertical load carrying capacity and had to be replaced. Following repairs, the bridge re-opened in 2014.

The 1954-built Gayhurst Road Bridge suffered irreparable damage as a result of the 2010-2011 earthquakes. The bridge is located on a bend in the Avon River, and lateral spreading of the young deposits on the inside of the bend was severe, with 800 mm displacement of the northern riverbank and 300 mm displacement of the abutment wall at the base relative to the top. Ground settlement on the northern side of the bridge (inside of the bend) exceeded 1 m, leaving a steep ramp up to the bridge. Settlement and lateral displacement on the outside of the bend to the south were relatively insignificant, demonstrating the lower liquefaction potential of the older, more consolidated, deposits there. The old bridge was demolished in 2014 and, at the time of writing, construction of the replacement bridge is nearing completion.

DESIGN OF BRIDGE PILES

The ground conditions at the location of Avondale Road Bridge are somewhat unusual in that a significant thickness of dense to very dense sand is present within the Christchurch Formation. At around 16 m thick, this was sufficient to found the new 1200 mm diameter abutment piles without the risk of the piles punching through to the underlying clayey silt that is typical of the base of this formation. Elsewhere in the city, the Christchurch Formation is characterised by deep medium dense sands. At Avondale Road Bridge, the depth of loose or medium dense sands was significantly less than for other bridge sites further east; this was assessed using a combination of CPT and SPT data. More confidence in the strength and liquefaction potential of the Christchurch Formation sands at depth was gained by the observation that the existing pier piles, which were subsequently re-used in the repair strategy, had not experienced any perceptible settlement in any of the 2010-2011 earthquakes. These ground conditions allowed a significant saving in pile length compared to founding in the underlying Riccarton Gravel. The dense sand of the Christchurch Formation has even in some cases provided higher driving resistance than some parts of the Riccarton Gravel (as described by Green et al, 2015).

Each bridge pier is supported on a row of eight driven 406 mm-square precast concrete piles. The pile toe level was estimated from the original construction drawings, and was assessed to lie near the top of the dense sand layer identified by ground investigations, where the average SPT N-value was 47. This explained the lack of observed settlement of the piers during the 2010-2011 earthquakes. Based on an assessment of SPT data, a geotechnical ultimate end bearing for driven piles of 12 MPa was adopted for the upper 8m portion of dense sand, increasing to a maximum of 15 MPa with embedment into the underlying 8 m thick very dense layer.

Following detailed design analyses, coupled with the Early Contractor Involvement (ECI) process involving the SCIRT Delivery Team in the planning, the design featured 1200 mm steel tubular piles, with precast concrete plugs. The intention was to install the casings by any appropriate means to the toe level, and excavate the material from inside the casing to approximately 500 mm above the toe of the casing (to minimise heave). The plug would then be dropped to the base and driven beyond the end of the casing until a 'set' demonstrated the required driving resistance was achieved.

At Gayhurst Road Bridge, the ground conditions were less favourable, with the more typical deep loose to medium dense sand and silt of the Christchurch Formation being present. Analysis of strength and liquefaction potential using extensive CPT and SPT data revealed no suitable founding stratum within the Christchurch Formation, leaving no economical option but to found piles in the Riccarton Gravel, present from 27 m below deck

level. The design for the replacement bridge included two 1200 mm diameter driven tubular piles for each abutment and the central pier. Extrapolated SPT N_{55} values for the Riccarton Gravel exceeded 75, however based on experience of driving piles into Riccarton Gravel (refer Green et al, 2015) a design geotechnical ultimate end bearing of 15 MPa was adopted.

PILE CONSTRUCTION

The structure of the new Gayhurst Road Bridge consisted of six piles each driven to a depth of approximately 30 m below bridge deck level. The outer steel casings were installed by vibration using an ICE 44/50 hammer. They were then excavated out using a clam bucket to confirm soil profile down the length of the pile and also the soil type at the base of excavation (approximately 500 mm above the toe of the casing). Following this a precast concrete plug was installed and driven underwater to the required set to achieve the ultimate bearing resistance. The reinforcing cage of the pile was then lowered in and tremie concrete installed.

The precast concrete plugs consisted of an outer steel shell and fully welded cap filled with concrete. The outer shell and cap were 12 mm and 25 mm thick respectively. Once the pile had been fully excavated the plug was lowered into position by a rope running through a 180 degree steel pipe bend that had been cast into the concrete. Holes were cut in the steel cap to allow the rope to run through and up to the top of the pile.

It was found that the optimal depth for pile excavation was between 500-800 mm above the casing toe. Both heave at the toe of the casing and minimum penetration requirements of the precast plug had to be considered and were a function of when to terminate excavation and begin driving. The depth that was chosen sufficiently mitigated heaving and also allowed the required set measurements to be achieved with approximately 1 m penetration of the plug beyond the casing toe into the gravel. The height of soils left in the casing before placing the plug was limited by the need to drive the plug beyond the casing toe. With increasing the excavation depth and reducing distance to casing toe the risk that the precast plug will be driven completely out the bottom of the casing increases.

DETERMINING PILE DRIVING REISISTANCE

A challenge arose in the method of determining the driving resistance for the driven precast plugs. For other types of bottom-driven pile where the plug is in contact with the casing, such as when using a gravel or cast in-situ concrete plug, the casing is dragged downwards during driving and a 'set' and 'rebound' can be directly measured on the casing and correlated to driving resistance using the Hiley Formula. For the precast plug this is not possible as the casing remains stationary and only the plug itself is driven downwards.

It was initially attempted to measure the permanent set and temporary compression directly from a steel 'leader' bar resting on the plug. This method was unsuccessful, however, as the plug was driven downwards with greater velocity than the bar fell under gravity. The bar then impacted the plug and bounced, leading to erroneous results. A more robust method was later employed with some success; a 90 mm steel drill rod was welded to the top of the steel cap and ran the length of the pile casing to the surface. In this way the temporary compression was directly measured for the first pile installed, and this was used as an assumed value for subsequent piles. The measured value was 5 mm, which was in agreement with the assumed value based on the temporary compression values recommended by the Auckland Structural Group and also the measured (albeit with poor reliability) values obtained using the 'leader bar' method. Despite this agreement, the results were variable and the values of rebound adopted were considered to have a high degree of uncertainty; therefore approaches to reduce the uncertainty were investigated.

Prior to construction of the bridges, it was considered to allow for water drag by applying an arbitrary reduction factor to the drop height of the 6,800 kg hammer falling in air, and then estimate driving resistance using the Hiley Formula. As it was found to be impractical to reliably measure rebound, attention turned to gaining more confidence in the other parameters used as inputs to dynamic pile driving formulae, with the aim of reducing the significant uncertainties involved. To achieve this, the energy of the hammer at impact with the plug was directly measured as a more reliable alternative to the arbitrary factor approach. A graduated scale was attached to the hammer, with a high-speed camera set at a known frame-rate of 60 frames per second to record the position of the hammer against time. The velocity at impact was thus calculated and combined with the known mass of the hammer to determine kinetic energy at impact. The kinetic energy at impact was calculated for two proposed drop heights; the calculated energies were 33 kJ and 127 kJ for a 3 m and 9.5 m drop, respectively. These values represent 17-20% of the energy that would be estimated for the same hammer falling through air by the product of weight and drop height. There may also be further minor energy losses in the energy transfer from hammer to plug, including compression of water between the hammer and plug at impact. The permanent set was also measured by taking levels on the hammer when resting on the plug following each blow. These measurements gave more

confidence in the input parameters to dynamic driving formulae, particularly given the lack of information on temporary compression.

Because the pile was being driven in water instead of air an equivalent drop height had to be calculated so that the Hiley Formula could be used to calculate ultimate driving resistance. In order to calculate an equivalent drop height the velocity as the piling hammer contacted the plug had to be ascertained. To do this a high speed camera was set up of the bank of the river to measure the drop time of the piling hammer. The actual fall distance of the hammer was 3.5 m, this was marked either on the hammer itself or on the wire crane rope in line with the video camera, with bulldog clips and spray paint.

By assuming the plug fell under constant acceleration (i.e. did not reach terminal velocity) the following equation of kinematics was able to be used:

$$d = \frac{v_i + v_f}{2} \times t$$

Where:

v_i = 0. Initial velocity of the hammer.
v_f = Final velocity as hammer contacts plug.
d = Drop height; marked on crane wire with steel ferrules.
t = Drop time; measured by the high speed camera.

This equation was then reduced to:

$$v_f = \frac{2 \times d}{t}$$

The drop time was then measured over a series of 15 drops and the time from this data giving 85% confidence used to calculate the impact velocity. An equivalent drop height in air which would equate to the same impact velocity was then determined and applied to the Hiley Formula to determine required set and temporary compression values.

Finally, the driving resistance was estimated using the Hiley Formula. At Gayhurst Road Bridge, driving resistance of between 16,900 kN and 19,100 kN was achieved, equating to ultimate end bearing of 18-20 MPa for the 1100 mm diameter plugs. This driving resistance was achieved following a large displacement of the plug of up to 1.5 m from its starting position.

ANALYSIS OF ALTERNATIVE DRIVING FORMULAE

As the temporary compressions (rebound) could not be reliably measured at Avondale Road or Gayhurst Road bridges, three alternative methods (as shown in Table 1) were considered as a check of a Hiley-based calculation, which used an assumed rebound. The methods considered were based on research undertaken by Long et al (2009) for Wisconsin Department of Transportation that compared the original Gates Formula with more modern derivatives. Long et al concluded that the original and modified Gates Formulas may under-estimate pile capacity where driving resistance exceeds around 3,300 kN, and proposed the 'WSDOT Method' as a reliable approach for a wide range of pile capacities.

The driving resistance of the precast plugs was estimated using the methods outlined in Table 1. The results of the comparison of driving resistance estimates

Method	Inputs Required	Formula
Hiley (Auckland Structural Group, 2002)	**Hammer energy** **Blow efficiency** **Permanent set** **Temporary compressions**	$R = \frac{E_h e_h}{s + \frac{c}{2}}$ R = Estimated driving resistance (kN) E_h = Hammer energy (kJ) e_h = Blow efficiency s = Permanent set (mm) c = Sum of temporary compressions (mm)
Gates (1957)	**Hammer energy** **Blow efficiency** **Permanent set**	$R = a \sqrt{E_h e_h}$ (b-log s) a = 104.5 and b = 2.4 (for SI units) e_h = 0.75 for drop and 0.85 for other hammers
Modified Gates Formula, 2001 (Long et al, 2009)	**As Gates (1957)**	$R_{(Gates\ (modified)} = 0.25 R^{1.35}_{(Gates\ (original)}$
WSDOT Method (Long et al, 2009)	**Hammer energy** **Blow efficiency** **Permanent set**	$R = 6.6 E_h e_h \ Ln(10N)$ R = Estimated driving resistance (kips) E_h = Hammer energy (kip.ft) N = Blows per inch

Table 1: Calculation methods considered for assessment of driving resistance for Avondale Road Bridge

for Avondale Road Bridge are presented in Figure 2. This shows that the WSDOT Method (Long et al, 2009) shows good agreement with the lower bound of estimations using the Hiley Formula adopting an assumed temporary compression of 5 mm. The assumed temporary compression was based on the recommendations of the Auckland Structural Group, considering *medium driving*. The *Original* and *Modified* Gates Formulas gave significantly lower estimates of driving resistance.

Based on this comparison, the WSDOT Method is considered to give an estimation of driving resistance that is consistent with the lower bound of values given by the Hiley Formula using an assumed rebound recommended by the Auckland Structural Group, and therefore for practical purposes may be used as a moderately conservative alternative to the Hiley Formula where reliable field measurement of rebound is not possible. This has the advantage that all the inputs can be directly determined for precast plugs.

The driving resistance estimated using the WSDOT Method for the abutment piles at Avondale Road Bridge was around 14,000 kN, equating to an ultimate end bearing (remembering that for driven isolated plugs the driving resistance represents only the end bearing component as the plug is isolated from the casing) in the order of 15 MPa for the 1100 mm diameter plug, which is consistent with the ultimate end bearing value adopted in design.

CONCLUSIONS

The determination of the energy delivered by a steel mandrel dropped through water to drive precast plugs showed that the energy delivered can be less than 20% of the energy delivered by the same weight falling in air. It should be noted that this energy reduction is specific to the geometry of the weight and casing for this particular arrangement, and viscosity of the fluid within the pile, for other sites the energy reduction may differ significantly from this. For such a method of bottom-driving piles, the

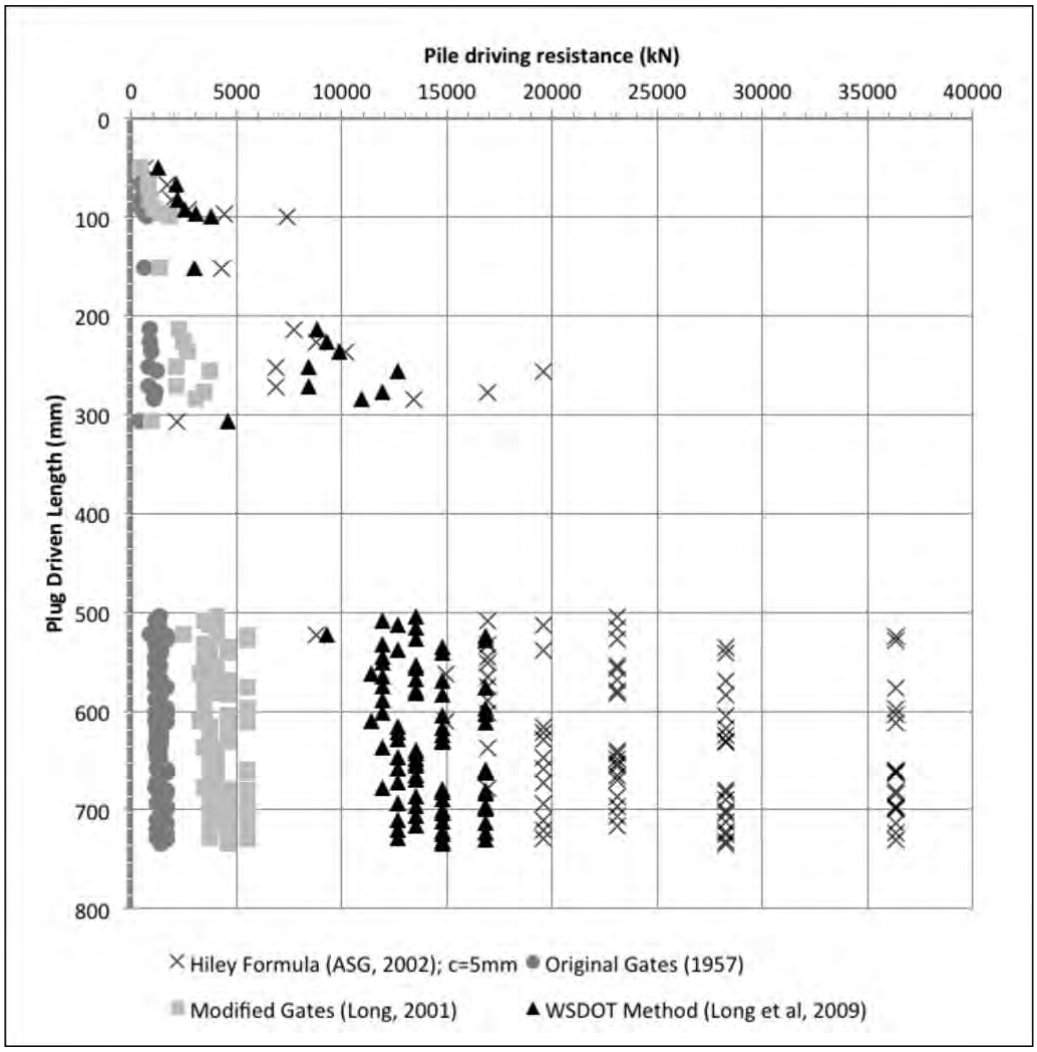

Figure 2: Estimates of driving resistance for Avondale Road Bridge using different calculation methods

direct measurement of velocity and mass to calculate the kinetic energy at impact is essential; otherwise significant error in the estimated driving resistance can result. The use of arbitrary factors to approximate water drag is therefore not recommended.

Experience has shown that field measurement of rebound for driven precast plugs is not practical. In such situations, consideration should be given to alternative dynamic driving formulae, with appropriate factor of safety for each based on the level of uncertainty involved with the input parameters. The WSDOT Method provides a reasonable moderately conservative method of estimating the driving resistance based on measured parameters. This method gives results that show reasonable agreement with estimation using the Hiley Formula, either using measured parameters or assumed temporary compressions recommended by the Auckland Structural Group. Analysis of the driving resistance of precast plugs that are isolated from the casing provides a direct estimate of ultimate end bearing capacity. The end bearing values obtained agreed well with those adopted for design based on a combination of SPT data analysis and experience of driving piles in similar soils.

Experience with driving precast plugs into dense granular soils has shown that the large displacement, in the order of 0.7m to 1.5m, of the plug is often required to achieve the design end bearing capacity, although this depends largely on soil type and strength at the pile toe. This is analogous to the difference recommended in literature between the end bearing capacity for bored and driven piles; the soil must be somewhat densified by displacement of the plug to be able to realise the benefits of a driven pile compared to bored. In top-driven piles this densification occurs during driving to the target depth and therefore is indistinguishable from the movement of the pile through the soil. The length of the plug must be carefully chosen to ensure that the top of the plug is not driven beyond the toe of the casing before the desired driving resistance is achieved. This must be balanced with the need to mitigate heave of the base prior to plug installation; for the particular piles considered here experience showed that the plug had to be set at least 500mm above the casing toe before driving to fully mitigate base heave.

In conclusion, the use of precast plugs had real practical advantages for the construction of the two bridges described here; other methods of driving these reasonably large diameter piles would likely have required impractical hammer weights to demonstrate the required capacity, or dewatering of sealed casings (with associated risk of failure). The primary disadvantage was the more challenging assessment of driving resistance. All methods of pile construction come with their own particular advantages, disadvantages and risks, and the method must always be chosen to suit the particular aims, practicalities and constraints of each project.

ACKNOWLEDGEMENTS

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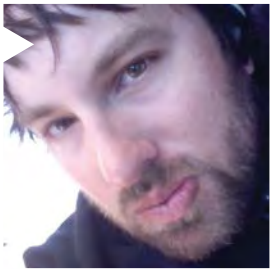
Common problems with Cone Penetration Test (CPT) data – a reference guide for CPT Practitioners and Geotechnical Designers

INTRODUCTION

Cone Penetration Testing (CPT) has become an increasingly popular method for characterising soils due to low cost, almost continuous data with depth, and established and robust correlations with soil properties, including liquefaction susceptibility. The CPT method is based on pushing an instrumented cone (with the tip facing down) into the ground at a controlled rate measuring tip resistance, sleeve friction, and often porewater pressure. The CPT method was first developed in Europe in the 1930s with the first electric cone developed by Fugro in 1965, the size and shape of which is the foundation for all contemporary CPT cones. The addition of the pore water sensor (piezocones), which enables the measurement of in-situ porewater pressure, was introduced in the 1970s increasing the overall utility of the method.

In contemporary New Zealand, CPT has proven to be a critical tool for characterising soil profiles and determining liquefaction risk in Christchurch following the Canterbury Earthquake Sequence of 2010-2012. The CPT is now commonly used in site investigations throughout New Zealand, not only for liquefaction hazard assessment purposes, but also for soil profiling, static foundation design and settlement estimates. Because all of New Zealand is active from a seismic hazard perspective, with no location immune to strong ground motions during earthquakes, it is important to undertake investigations to understand liquefaction hazards at sites for development or redevelopment due to potential liquefaction susceptibility. There are numerous computer programs available that can calculate soil parameters (often based on Robertson, 1990) and design values from raw CPT data and assist with the interpretation of the CPT data and the characterisation of various soil parameters or even direct to design values (such as pile capacity) and liquefaction assessment. It is convenient to use these programs, but consideration must be given to the quality and accuracy of the CPT data that is used in this manner. Furthermore, consideration should be given to the limitations of the test and the applicability of correlations in the soil type or geological setting that is



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John Cresswell

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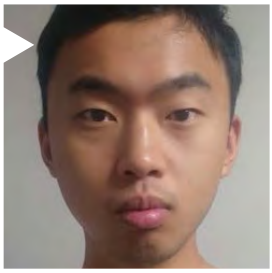

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being investigated. The CPT is not a 'silver bullet' that necessarily gives an appropriate geotechncial answer from blindly inputting the raw CPT data directly into computer programs.

As CPT data is often used to support significant buildings and other design decisions it is important that the agency commissioning the CPT has confidence in the quality of the results of any CPT investigation. It is therefore important to have an understanding of problems that can occur during CPT investigations or when CPT systems are not working correctly. This paper outlines a number of common testing problems and data errors to assist and inform geotechnical designers in CPT data quality assessment. This can form an important step in quality control and quality assurance procedures for the use of CPT data in geotechnical engineering applications. For the purpose of this review, we base the "correct" operating standards on ASTM D5778-12 standard for CPT, which is the standard most commonly used. British Standards and ISO also provide insight into CPT operations. ASTM also forms the framework most commonly used in New Zealand for liquefaction assessments. Importantly, the authors do not intend to educate the reader on the interpretation of CPT, however insight into CPT interpretation may inadvertently occur.

This paper considers the following general issues that may be encountered during the collection of CPT data:

1. Ground conditions
2. Zero load readings
3. Importance of cone calibration
4. Negative tip resistance (q_c) and sleeve friction (f_s) readings
5. Missing data (i.e. data gaps)
6. Cone dimensions and effects of wear on equipment
7. Pore pressure errors
8. Premature test termination

The dataset most frequently used in this paper was CPT data from 2011 to 2013 as part of EQC TC3 Geotechnical Investigation Project in Christchurch, although other in-house datasets were also used. During a quality review of the impressive EQC CPT dataset, a number of common problems with the data were observed that, in conjunction with CPT practitioners' experience, provide insight into CPT testing problems, how to identify them, and finally how to avoid them in the future. It should be noted that this paper does not give an exhaustive coverage of all problems that may occur, but covers some of the more common issues and is intended to promote one to think about the data that is presented from a CPT test rather than accept any dataset unquestionably as accurate.

1. GROUND CONDITION ISSUES

CPTs generally provide valuable geotechnical data in almost any soil type, however there are at least two geologic settings where soils or soil profiles have special characteristics that the geotechnical professional needs to be aware of when reviewing CPT data collected in these geologic settings. Much of the central North Island sediments are comprised of volcanic (including pumicious) and residual soils. Additionally, many of the alluvial soils in the North Island are redeposited or reworked residual and volcanic soils, and may retain some of the "problematic" characteristics of the parent material. Consequently, much of the North Island is potentially affected by these materials. These soil types may not necessarily affect the quality of the data collected, but may have an effect on the applicability of the interpretation of the data when using standard correlations that have been developed on "normal" sedimentary soils. Two examples of ground specific conditions that can affect CPT data are working in areas with high geothermal gradients (e.g. Taupo) and pumice-derived soils (e.g. Waikato).

In geothermal areas, the high temperature gradient may affect the electronic equipment in the cone. Modern CPT cones are generally temperature compensated, but there will be a limit to which they can tolerate heat without affecting the load cell calibration. Also, the temperature compensation is a delayed effect and does not cope well with transient temperature changes (when one part of the probe is hotter than another part). When working in geothermal areas, the tolerance of the cone used should be checked and the results of the CPT should be treated with caution. High temperature capacity cones could be used in this instance instead of conventional cones where CPT data may be considered of great importance for design.

In the case of pumice soils, the CPT probe tends to crush the soil structure and the results may then be meaningless in terms of representing the in-situ properties of the soil. Crushing is important because it is actively changing the soil structure (i.e. mechanical reduction of size), not just pushing through the soil column. Orense and Pender (2013) demonstrated that conventional CPT soil parameter correlations may not apply to pumice soils due to grain crushing effects.

One way of assessing the possible existence of pumicious soils is to combine the CPT with a seismic test (e.g. SCPT or downhole seismic surveys) to obtain an in-situ shear wave velocity profile. By comparing the measured shear wave velocities with those obtained by correlation from the cone data, one can see where problematic soils may exist. If the interpreted shear wave velocities, derived from the CPT data closely match the measured shear wave velocities obtained from seismic testing, then it is likely that the conventional soil parameter correlations will be applicable. If shear wave velocities don't match well, then there is a likelihood that the soil behaviour may not be predictable and caution should be exercised when deriving design parameters from CPT tests.

In all cases, whether problematic soils are expected or not, CPT data should not be used in isolation from other information (e.g. borehole data, natural exposures, geologic mapping, etc), unless the ground conditions are well understood from previous and nearby investigations. Doing a "blind" CPT test where there is no pre-existing data may cause incorrect use of the data or misinterpretation. For example, soundings through desiccated clay can provide interpretations of sandy or silt mixtures due to sleeve friction.

2. ZERO LOAD ISSUES

Before and after any test zero readings (also called zero load readings) are taken to provide an indication of the accuracy of the cone's load cell and/or potential operator error. If the zero readings have drifted beyond the acceptable margin of error or if there are signs of instability in the cone, then the cone should be taken out of service for repair or recalibration. The stability of the cone can be checked by monitoring the zero readings for a minute or two before starting the test. The zero readings should be stable.

3. IMPORTANCE OF CONE CALIBRATIONS

Only CPT cones with validated calibration certificates (and calibrated by a reputable CPT Engineering firm) should be used. Cones should be calibrated at least every 12 months, when the cone has gone out of calibration (visible on the zero load readings), or if damaged during use. Up-to-date calibration certificates for the cones being used onsite must be present onsite whenever testing is conducted for inspection by the client. A copy of the certificate should be included as part of factual CPT reporting. Last date of inspection and calibration, identification number of CPT cone, name of cone development company, and name of CPT contractor should all be clearly stated on the calibration certificate. A recent calibration certificate does not mean that the cone has not been damaged from recent testing. A review of the cone condition should be undertaken prior to testing to make sure that it is in good working order.

4. NEGATIVE CONE RESISTANCE (Q_C) AND SLEEVE FRICTION (F_S) VALUES

Sometimes negative cone resistances (q_c) or negative sleeve friction (f_s) can be recorded which is a general

indicator that something may be wrong with the data. Figures 1 and 2 show examples of negative q_c and f_s readings respectively.

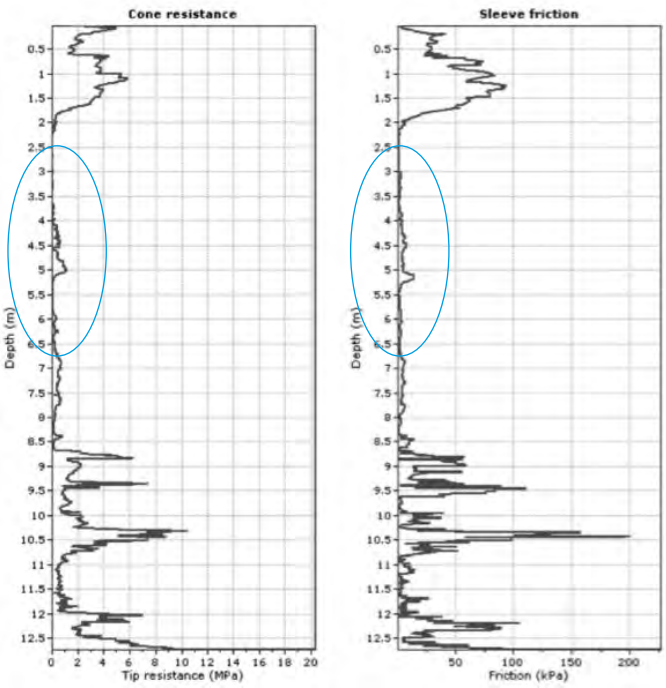

Figure 1: CPT Plot showing negative tip and sleeve resistance (2m to 6m).

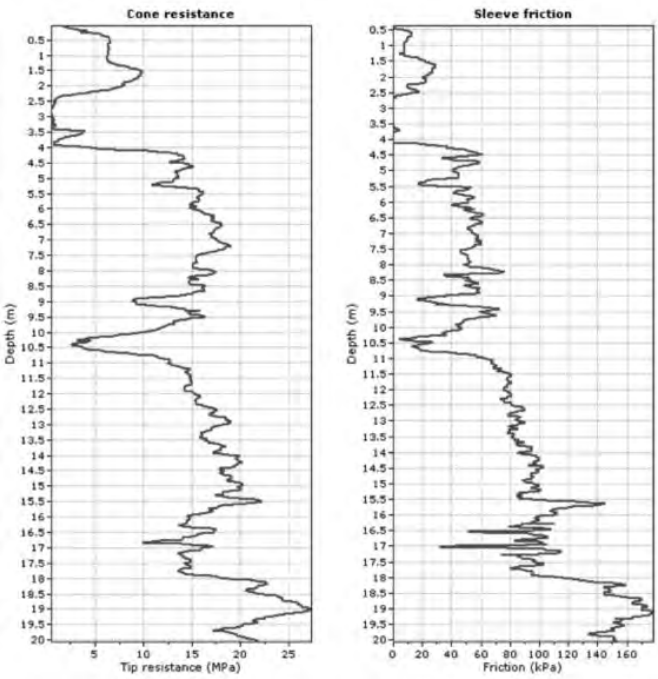

Figure 2: CPT Plot showing negative sleeve friction (2.5m to 3.5m).

Negative q_c readings are likely to be due to one of the following reasons:

- Faulty cone (instrument)
- Incorrect cone assembly or dirty cone
- Pre-loaded cone
- Temperature effects
- Load cell error

If the cone is not properly tightened during assembly, then this could also potentially cause negative readings. If the cone tip is preloaded (e.g. the cone tip is touching the ground surface at the time of initial zero readings) or the operator is holding the friction sleeve, then this may result in negative readings being recorded in softer soils. This effect should show in the final zero readings. If this is suspected, then the data could be adjusted using the final zero readings to correct the negative values. Importantly, a faulty cone may "recover" or phase in and out or may show erronous readings and then operate correctly. This must be carefully evaluated. However, any such correction should be clearly stated in the results and the results taken with caution. Ideally, the test should be repeated.

Temperature effects are unlikely to result in negative qc readings as cones are generally temperature compensated, but this is also a potential cause. Cones should be brought to a temperature close to that in the ground prior to starting the test. This is normally achieved by placing the assembled cone into a bucket of freshly drawn clean tap water that is as close to the ground temperature as possible.

It is important to be aware that all load cells have a margin of error that is related to their full scale output (FSO). Higher capacity cones are less accurate at the low end than smaller capacity cones. Higher capacity cones are usually required when testing through dense or coarse-grained soils (e.g. gravel), but may give less accurate readings when soft soils are encountered. In these situations, the cone may be perfectly within standard tolerances and tested in correct procedure but still show negative readings. Consequently, small negative readings may not necessarily be an indication of a faulty cone or operator error. Consideration should be given to the ground conditions and what information is required before specifying a particular capacity cone for a project.

Negative f_s readings are likely to be due to one of the following reasons (though this list does not cover all possibilities):

- Faulty cone
- Incorrectly assembled or dirty cone
- Ground interaction
- Temperature effects

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- Cross talk between sensors
- Split friction sleeves

As with the cone tip, the sleeve friction may be affected by incorrect assembly. Some cone types require the sleeve to be screwed on to the body of the probe. If not tightened properly, then errors in the sleeve friction (including negative readings) can occur. Figure 3 shows plots of sleeve friction where the sleeve is loose compared to a properly tightened sleeve. A dirty cone, where dirt has accumulated on the seal at the end of the friction sleeve can also affect the initial zero readings and potentially cause negative f_s values. Additionally, leaky seals or poorly fitting "x" rings may also contribute to negative f_s values.

Another common cause of negative sleeve friction can occur when pushing through gravels. The gravels are pushed aside by the cone tip and can roll back onto the sleeve putting a negative force on the load cell. A sleeve damaged by the ground conditions may also record negative readings. Sleeves should be post-testing in ground where gravels are encountered to evaluate any damage that requires refurbishment or may require recalibration.

The zero readings again will provide a clue as to the stability of the load cell. If the zero readings are outside the margin of error allowed for in the ASTM standard, then the test should be repeated. If the zero readings suggest unacceptable drift or instability, then the cone should be taken out of service and repaired. As with the cone tip, the sleeve is unlikely to be affected by temperature and thus cause negative readings, but it is a possibility and care should be taken in the preparation of the cone prior to testing.

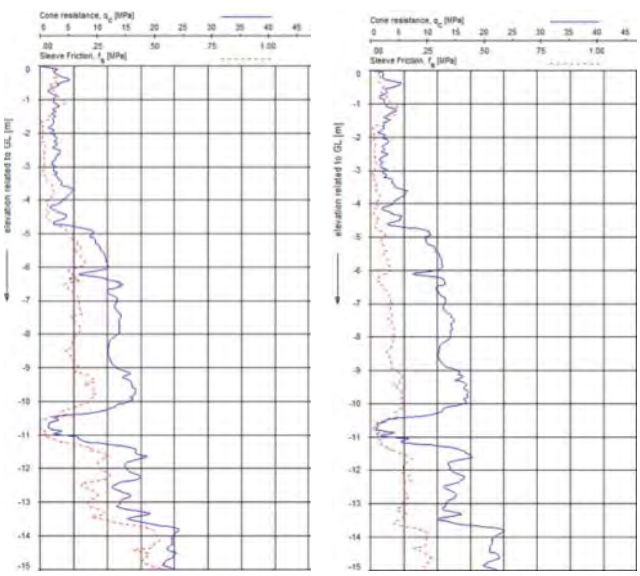

Figure 3: Two CPT Plots carried out nearby (~1m), one with loose friction sleeve on the left and the example on the right with correctly assembled CPT cone.

5. MISSING DATA (I.E. DATA GAPS)

Figure 4 shows an example where there appears to be a gap in the recording of data. This may be due to temporary loss of information due to a fault in the cone electronics, cable or acquisition system or a blip in the depth encoder. A gap in data does not necessarily mean that the remaining data is incorrect, but should be taken with caution. The test should be repeated.

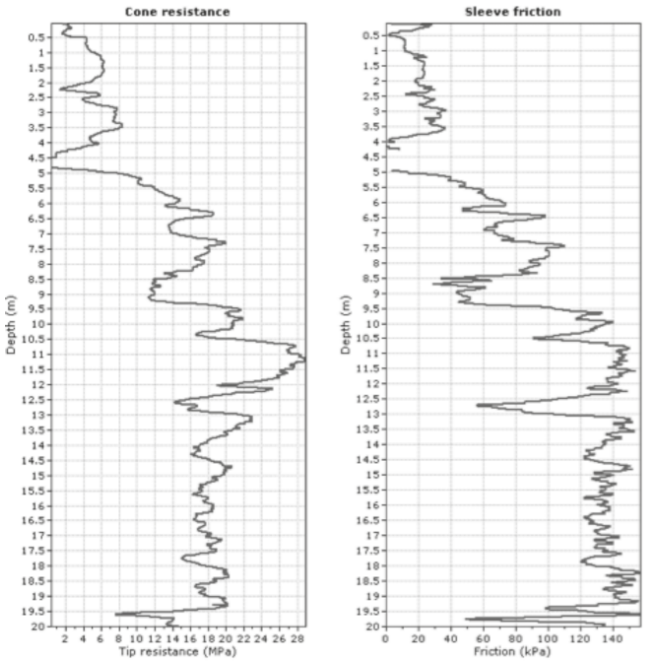

Figure 4: CPT Plot showing gap in data (4.3m to 4.9m).

6. EFFECTS OF CONE DIMENSIONS AND EFFECTS OF WEAR ON EQUIPMENT

The tolerance for the dimensions of the cone tip and friction sleeve are specified in the standard ASTM D5778-12. With use, the cone tip and sleeve will wear. The wear on the cone and sleeve will affect the readings and once wear is beyond the allowable tolerances, the readings are then not considered to be in accordance with the standard. It should be noted, however, that there is a wide tolerance range allowed for in the standard. Cones worn or those manufactured to different dimensions (but still within the standard tolerances) may show significantly different readings. Figure 5 shows the effect on sleeve friction readings from sleeves of various diameters. In addition, the roughness of friction sleeves will also affect f_s readings. Consequently, cones of different manufacturers may show different readings with CPTs on the same site. CPT operators should take regular measurements of their cone dimensions and report these in their results as well as reporting the manufacturer of the cone used. Additionally, the surface condition of the cones should also be taken into consideration. Is the cone "worn out" looking or are

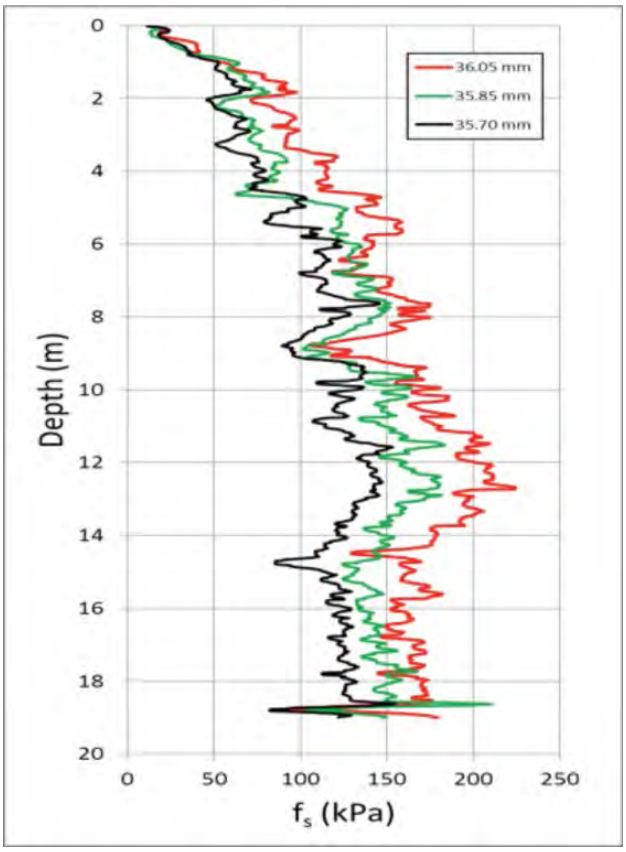

Figure 5: CPT plot showing the effect that different sleeve diameters have on f_s measurements.

there any burrs or other imperfections on the surfaces of the cone? If so, these could lead to changes in the surface area/cone angle and provide inaccurate readings.

7. PORE PRESSURE (U) EFFECTS

The pore pressure transducers (U) that are used in modern CPT cones (CPTu or sometimes also called piezocone penetration tests, PCPT) are significantly more sensitive than the load cells used in the cone tip and sleeve. Consequently, the U2 measurement (which is often but not always used for correlations) is the most responsive to changes in porewater pressures of the three measurements of the CPTu as long as the cone is working correctly. It is completely de-aired during assembly, and saturation is maintained during the test, which is often difficult to achieve. Even careful assembly in accordance with the standard does not necessarily mean that the cone will be totally free of air. Using a vacuum chamber on site to de-air the cone is the best method to ensure good saturation, but this is not always practical. What is more difficult to control is the effect of the ground conditions on saturation during the test. Often the test is pushed through metres of unsaturated soils at the surface, which is often the main factor in loss of saturation. Loss of saturation can also occur when pushing through dense sand or heavily overconsolidated soils (e.g. dilatant conditions by cavitation) or through layered soils of sands and clays. Consequently the main issue with pore pressure measurements is maintaining saturation. Unfortunately, there are no "tricks" for dealing with saturation, there are really only two ways to deal with this: 1) If the groundwater table depth (GWT) is known, the operator can push the cone to just below this level and pause the test to wait for the pressure to come back (although this rarely works) and 2) the operator can push the CPT cone down to the GWT, followed by extraction of the cone, then clean water can be poured down into the hole (while resetting the cone filters), however this does not often work as the hole may collapse during resetting of the cone. Generally, the result of poor saturation is slow response of the transducer to changes in the porewater pressures generated by the cone as it is pushed in the ground.

If the pore pressure system is fully saturated, then you will often see a spiky (or peaky) plot because the transducer is responding immediately to continuous variations in the pore pressure (Lunne et al., 1997). If the cone is not properly de-aired, then the response will be sluggish and the plot will be smoother, generally with distinct steps where the rod changes have occurred. Figure 6 below shows three examples of U2 plots at the same site.

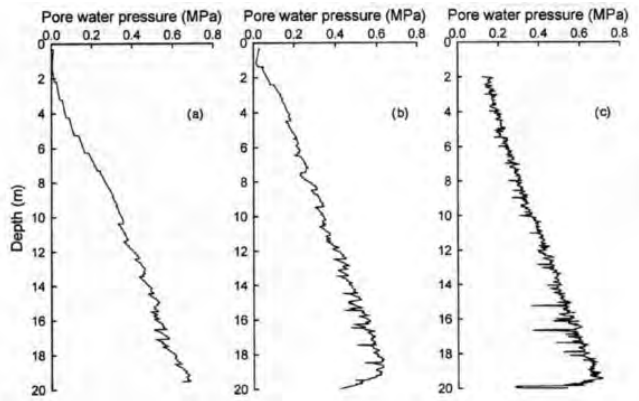

Figure 6: Examples of saturation on U2 measurements, Plot (a) is poorly saturated, and Plot (c) showing good saturation. Plot (b) is at a saturation state somewhere between (a) and (b). (*After* Lunne, et al. 1997).

Sometimes a sudden large negative pore pressure can be experienced when pushing into a dense sand or gravel. This can cause dissolved air in the groundwater to come out of solution and form air bubbles within the system. This is called cavitation (this is not to be confused with de-saturation). Once this occurs, the pore pressure measurement becomes unresponsive. An example of cavitation is given in Figure 7 below. The

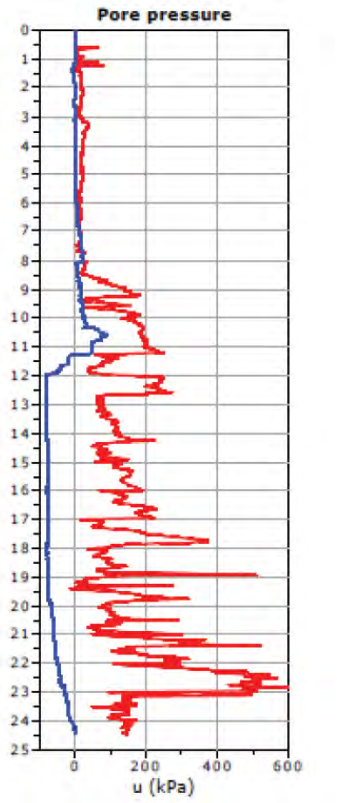

Figure 7: Example of cavitation in U2 plot.

two plots are from side by side CPTs. At about 11 m depth one of the plots goes into sudden negative pore pressure and is unresponsive below that with gradual return of pore pressure below about 20 m. If a cone becomes de-saturated (or cavitation occurs) during a test, penetration of the cone should be stopped (provided it is below the water table) until the pore pressure re-establishes itself. It should be noted that negative pore pressures are sometimes measured as a normal response of the ground, oftentimes in dilatant soils. Thus in these cases, it does not necessarily mean that there was de-saturation or cavitation unless there was a sluggish response following the negative pressure.

Incorrect zero readings can also affect U2 measurements. In Figure 8 the two plots show U2 readings based on incorrect zero readings. The plot to the left shows a jump in pore pressure shortly after pushing below the ground surface. This is believed to be due to residual negative pore pressure following removal from a vacuum chamber and then a condom immediately placed over the cone. This caused the initial zero reading to be too low, which then measured higher values once the condom broke in the ground. The graph to the right shows the plot corrected by applying the end zero readings.

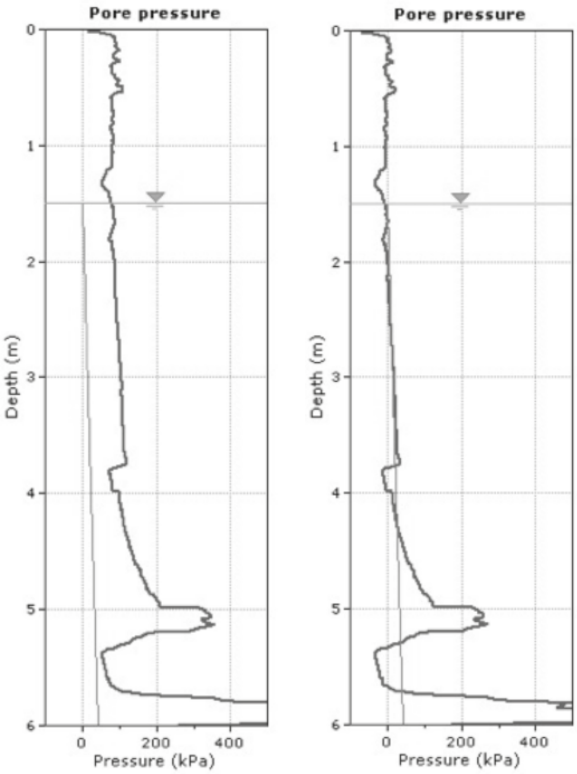

Figure 8: Plot of U2 illustrating incorrect zero reading in left hand plot.

8. PREMATURE CPT TERMINATION

The refusal of a CPT (i.e. termination of a test) is an important technical parameter in addition to being a health, safety and environmental item during geotechnical testing to help ensure that people, equipment, or the environment are not negatively affected by CPT testing. Setting refusal criteria ensures that testing is terminated *prior* to any damage, injury (to people or the environment) or potential loss of equipment. Refusal criteria for CPT testing should be agreed upon by both the CPT operator and the geotechnical designer prior to fieldwork and the reasons for test refusal reported with submission of any field data (raw or otherwise) and in any factual or interpretive reporting using these data. Acceptable criteria for test termination are as follows, unless specifically agreed otherwise prior to completion of testing: a) as instructed by Client/Client Representive prior to reaching desired depth (e.g. reaching a target formation e.g. Riccarton Gravels), b) reaching a target depth (e.g. taking a test to 20 m), c) reaching maximum capacity of the thrust machine, reaction equipment, push rods and/or measuring sensors (i.e. CPT cones) – these items will be manufacturer dependent, d) a sudden increase in penetrometer inclination (>5 degrees in 1 m) or 15 degrees total inclination within a test (as per ASTM Standards), e) other circumstances including a combination of the above which are at the discretion of CPT operator, such as risk of damage to apparatus and/or safety of personnel or the environment based on professional experience.

Examples (a) through (d) are objective and can be easily outlined at the proposal stage, monitored during testing, and reported during the reporting stage. As these can be quantified, they are fairly easy to track. However, if the termination of the test provides the geotechnical designer with enough information about site due to premature termination is another question entirely. Example (e) is less straightforward and is very subjective. Sometimes (e) can be from a combination of the quantitative criteria in addition to "feel" of the CPT rig during testing, and can be sufficient justification to terminate a test. This is something that can be common during drilling, for example with a driller stating that "something just didn't feel right with the torque", etc. Experience using the geotechnical equipment gives the operators incredible insight as to when things are going well or not (using of course the computers that log the CPT data as well as gauges on the CPT rigs in addition to human reasoning), and it is important to rely on this prior to things going wrong. Leaving broken CPT rods in the ground due to exceeding the penetration inclination is something that can be avoidable in most cases. In summary, the justification for CPT termination should be provided to the geotechnical designer even as a combination of the aforementioned variables which can also provide additional insight into ground conditions.

CONCLUSIONS

CPT is one of the best methods for the collection of high-quality geotechnical data. However, because it is an in-situ test that includes electronic and electrical equipment, problems can occur during data collection that can influence data quality and accuracy. Both the geotechnical designer and the CPT operators should be aware of the items discussed in this paper before, during, and after testing. Of course, any successful geotechnical investigation will always be based on good communication with expectations clearly outlined at the planning stage.

To recap, the three main areas that should be taken into consideration before any successful CPT investigation are:

1) A review of the equipment – is the cone calibrated and in good working condition?
2) Are the methods being used the right methods? (i.e. are zero loads being done, is the cone being pushed at a constant rate, etc).
3) At the data review stage, did the equipment and methods work well?

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If geotechnical designers working with CPT practitioners can answer the above questions, both can be confident that these data are of sufficient quality to be used for design. Ultimately, CPTs are powerful geotechnical tools, but is not a "silver bullet" that necessarily provides the right answers in all situations. There is always some margin of error with any subsurface investigation, irrespective of how well the methods, equipment, operators, and ground conditions mesh together. The geotechnical designer should work closely with the CPT contractor so that the most appropriate equipment is used for the project (e.g. cone size, rig size, etc) and the limitations and potential errors are clearly understood. By working together and having quality control systems in place, CPT operators, geotechnical designers, their clients, and consenting bodies can have confidence in the quality and utility of CPT data for any given site in New Zealand. The examples provided here will hopefully help the New Zealand geotechnical community continue to raise the geotechnical game and be an example of cutting edge site investigation practices that the rest of the geotechnical world can look to for guidance.

ACKNOWLEDGEMENTS

The authors thank MBIE and NZGS support of the Geotechnical earthquake engineering practice Module 4 (Guideline for Residential and Commercial Geotechnical Investigation for Liquefaction Assessment Purposes) which provided the impetus for this work. Thanks to Robert Waugh for a critical review. Additional thanks to the Earthquake Commission of New Zealand "EQC" for supporting the EQC TC3 Geotechnical Investigation Project which provided an comprehensive CPT dataset and additional insight into potential challenges with the collection and use of CPT data.

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The horizontal-to-vertical spectral ratio technique: Application and limitations

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IN THE DECEMBER **2014 issue of NZ Geomechanics News HVSR & seismic zonation: a case study based on current NZS1170.5 subsoil classification methods (Mazzoni et al. 2014) was published. This article provided an overview of the horizontal-to-vertical-spectral ratio technique (referred to as the HVSR, H/V or Nakamura method), and presented two case studies of its use in Christchurch. Here we provide further discussion of the application and the limitations of the H/V spectral ratio technique, particularly in the context of Christchurch. We do not cover the theoretical aspects of the technique in detail, with readers directed towards the studies of Nakamura (1989), Field et al. (1990), Field & Jacob (1993) and Sánchez-Sesma et al. (2011). The definition of site period in the context of NZS1170.5 (SNZ 2004) is presented first as this controls the interpretation of appropriate site subsoil classes for seismic design in New Zealand. The influence of equipment capabilities on the technique is then outlined, and the appropriateness of different equipment for different geologic settings. We then discuss the characterisation of site period in Christchurch, and revisit the Hagley Park case study presented in Mazzoni et al. (2014) utilising experimental data from other studies.**

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1 SITE PERIOD ACCORDING TO NZS1170.5 (2004)

NZS1170.5 (SNZ 2004) uses a site period (or natural period) of 0.6 seconds as the preferred approach to delineate between site subsoil classes C-Shallow Soil and D-Deep or Soft Soil, where:

"The low amplitude natural period may be estimated from four times the shear-wave travel time from the surface to rock, be estimated from Nakamura ratios or from recorded earthquake motions, or be evaluated in accordance with Clause 3.1.3.7 for sites with layered subsoil, according to the hierarchy of methods given in Clause 3.1.3.1."

This indicates that site period should be defined for the entire geotechnical profile between rock and the ground surface when classifying a site. The preferred approach is the direct use of the shear wave velocity profile to rock, with the Nakamura or H/V spectral ratio approach from ambient noise a subsequent, but less preferred approach. For both these approaches, a key component in the definition of site period is a good understanding of the underlying geology of a location. The definition of site period being from the surface to rock has particular

importance for the H/V spectral ratio technique, as this method can often identify multiple spectral peaks, and not just one peak corresponding to the site period. These peaks correspond to different modes of vibration, many of which involve significant deformation over a localized depth range relative to the overall soil profile, due to the variation in the thickness and velocity (and hence impedance contrast) of different geotechnical materials. This is discussed further in Section 3.

One of the main motivations for the use of site period in NZS1170.5, instead of the time averaged shear wave velocity to 30 m depth (VS30), which is used for site classification in other countries (such as the NEHRP classifications in the United States (BSSC 2003)), is that many New Zealand profiles have sedimentary (or volcanic) soils beyond 30 m depth that significantly contribute to the site period and hence the near surface seismic response. The geologic setting in Christchurch is the very situation that motivated the use of site period that incorporates all geology above bedrock.

2 EQUIPMENT SPECIFICATIONS AND CAPABILITIES FOR H/V MEASUREMENTS

An important factor when performing H/V spectral ratio measurements is a clear understanding of the limitations of the equipment that is being used. Measurements of the required Fourier amplitude spectra can be made using geophones, accelerometers, or broadband seismometers that record ambient vibrations in the vertical and two horizontal degrees of freedom, with each type of equipment having different resolution over different frequencies of vibration. Equipment can cost from several hundred (geophones) to tens of thousands of dollars (broadband seismometers), with an increase in cost typically corresponding to an increase in the maximum site period that can consistently be measured. The main advantage of lower cost equipment (i.e. geophones, accelerometers) is that it generally allows for more efficient testing, due to a reduced time for the sensors to 'stabilise' before accurate measurements of low frequency ambient noise can commence. In contrast, more expensive equipment (i.e. broadband seismometers) often require a greater stabilisation time.

Lower cost equipment is usually appropriate for locations where rock exists at relatively shallow depths (less than a few hundred metres) and/or there are consistent high ambient noise levels. In deep sedimentary basins with basement rock at greater than several hundred metres depth, more expensive equipment is usually required to be able to detect stable H/V spectral peaks at longer periods. In ideal conditions (i.e. high levels of ambient noise at longer periods caused by wave and atmospheric phenomena, and low levels of noise at shorter periods due to anthropogenic activities), lower cost equipment may be able to measure longer site periods, however such conditions rarely exist. Another important factor is the length of time that records are taken at a site, with longer records required to identify a stable H/V spectral peak for sites with longer site periods.

The above discussion points to the importance of utilising equipment that is fit-for-purpose with regard to the specific geological conditions at the testing locations considered.

3 CHRISTCHURCH SITE PERIODS

In order to provide context to the study of Mazzoni et al (2014), this section compares their results with the H/V spectral ratio measurements across Christchurch using broadband seismometers by Wotherspoon et al. (2014, 2015) and Cox et al. (2015). These studies showed that the H/V spectral peak/peaks in the city likely correspond to (1) the site period for the entire soil profile down to basement rock (a significant impedance contrast); and/or (2) the site period of shallow sandy soils above either Quaternary gravels or Miocene volcanics (which are the strong shallow impedance contrasts in the Canterbury region).

Across much of Christchurch, a significant impedance contrast exists between the top of the Riccarton Gravels and overlying looser sediments (Christchurch and Springston Formations) and result in a significant mode of vibration that has a much shorter period than the period of the entire soil column down to basement rock. Surface wave measurements have shown that the Riccarton Gravels have a shear wave velocity (VS) of approximately 400-500 m/s (Wotherspoon et al. 2014), and therefore should not be considered engineering bedrock (typically VS > 760 m/s), let alone geological bedrock.

H/V spectral ratio data from measurements located less than 350 m from the base of the Port Hills (indicated by the solid red line) are summarised in Figure 1 (Site L1, L2 and L3). There is a clear, large-amplitude, H/V spectral peak at each location, which is likely to be resulting from the significant impedance contrast between soils/gravels and the underlying Miocene volcanics, with estimated site periods ranging from T=0.7-1.0 seconds. Because the depth to volcanics increases rapidly moving away from the Port Hills, this suggests that any locations further than a few hundred metres from the Port Hills will have site periods to rock well above the T=0.6 second threshold between site subsoil Class C and D. A summary of site periods to rock across Christchurch and Northern Canterbury is presented in Wotherspoon et al. (2015).

4 HAGLEY PARK CASE STUDY

The Hagley Park case study presented in Mazzoni et al. (2014) is a good example of a situation where a clear understanding of the underlying geology at a site is needed to properly interpret H/V spectral ratio measurements. Mazzoni et al. suggest that H/V spectral peaks in Hagley Park "may lead to amplification factors closer to a Class C soil classification". However, multiple hydrological wells in the region, up to 150 m in depth, pass through interbedded gravels and sands/silts, with no indication of rock (Brown & Wilson 1988). Therefore, even without H/V spectral ratio measurements, this area would be classified as site subsoil class D based on the material depth limits of Table 3.2 of NZS1170.5 (depth limits that have been shown to be overestimated by 10-20% for cohesionless soils by Larkin & Van Houtte (2014)).

Figure 1 presents H/V spectral ratio data from Site L4 on the edge of Hagley Park, approximately 3 km from the base of the Port Hills. There is a clear short period spectral peak at 0.75 seconds, which corresponds to the response of the soil layers above the Riccarton Gravels. This short period peak is similar to that identified by Mazzoni et al., however the peak in Figure 1 is more clearly defined. A second much longer period spectral peak was identified at 2.9 seconds, likely corresponding to the natural period of the deposits above bedrock. Re-examining the Mazzoni et al. results, there appears to be a minor increase in the H/V spectral ratio over a broad range at long periods, however nothing that would be able to clearly resolve this longer period peak.

On the basis of the above discussion, it is the author's strong opinion that the estimated site period in Hagley Park suggested by Mazzoni et al. is the response of the soil layers above the Riccarton Gravels that are present at approximately 20 m depth, and not the site period above bedrock. Their observed H/V peak is from a strong shallow impedance contrast, but this peak does not correspond to the mode of vibration with the longest period. Hence, the period measured by Mazzoni et al. does not correspond to the site period parameter from which the NZS1170.5 C/D soil classifications are based. Possible reasons that Mazzoni et al. may not have been able to coherently resolve the longer period peak include limitations in the performance of the equipment used, the records taken at each site may have not been long enough to resolve a stable H/V peak, the ambient noise characteristics may not have been desirable, or a combination of these factors.

While the site period is necessary for classifying site subsoil class C or D, VS profiles are also of interest in site classification under NZS1170.5 for the boundary between site subsoil class D and E (>10 m of material with VS>150 m/s). From their Hagley Park H/V spectral ratio data, Mazzoni et al. develop a simplified VS profile that was constrained using "the first major contrast of seismic impedance", which in this case was most likely to be the depth to the Riccarton Gravels. This approach may be acceptable if time averaged shear wave velocity is the metric of interest for site classification (such as VS30). However, this method of calculating the VS profile is less applicable to the NZS1170.5 framework, where more accurate determination of the profile is required. If the VS calculations/inversions are not constrained by knowledge of actual site layering, the calculated properties may be smeared close to the ground surface, and in some cases it could lead to incorrect classification of sites in Christchurch where soft deposits below the site subsoil class E VS limit exist. Other site investigation approaches such as downhole seismic testing, seismic CPT or multi-channel analysis of surface waves (MASW) may be more appropriate, with accurate interpretation of these methods also very important (but this is outside the scope of this article). It should also be noted that geophysical testing characterises the low strain properties of the soil profile but does not identify the geotechnical nature of the materials, and insight into the modulus-strain relationship is needed for high strain ground response analyses.

6 SUMMARY

In summary, the H/V spectral ratio technique is an efficient tool for assessing the overall dynamic characteristics of a site, however care must be taken when interpreting this data. The key factors to ensure high quality conclusions are drawn based on H/V measurements are:

Background knowledge of the underlying geology of a site.

A clear understanding of the limitations of the equipment, and the use of equipment appropriate for the geologic conditions (specifically maximum resolvable period of the equipment relative to the expected site periods to be measured).

If shear wave velocity profiles are to be developed based on H/V spectral ratio data, fit for purpose constraints on subsurface layering should be utilised.

In Christchurch, locations greater than a few hundred metres from the base of the Port Hills are likely to have a site period greater than 0.6 seconds, and hence site subsoil class D and E are the appropriate classifications for seismic design for locations across the rest of Christchurch (Wotherspoon et al. 2015).

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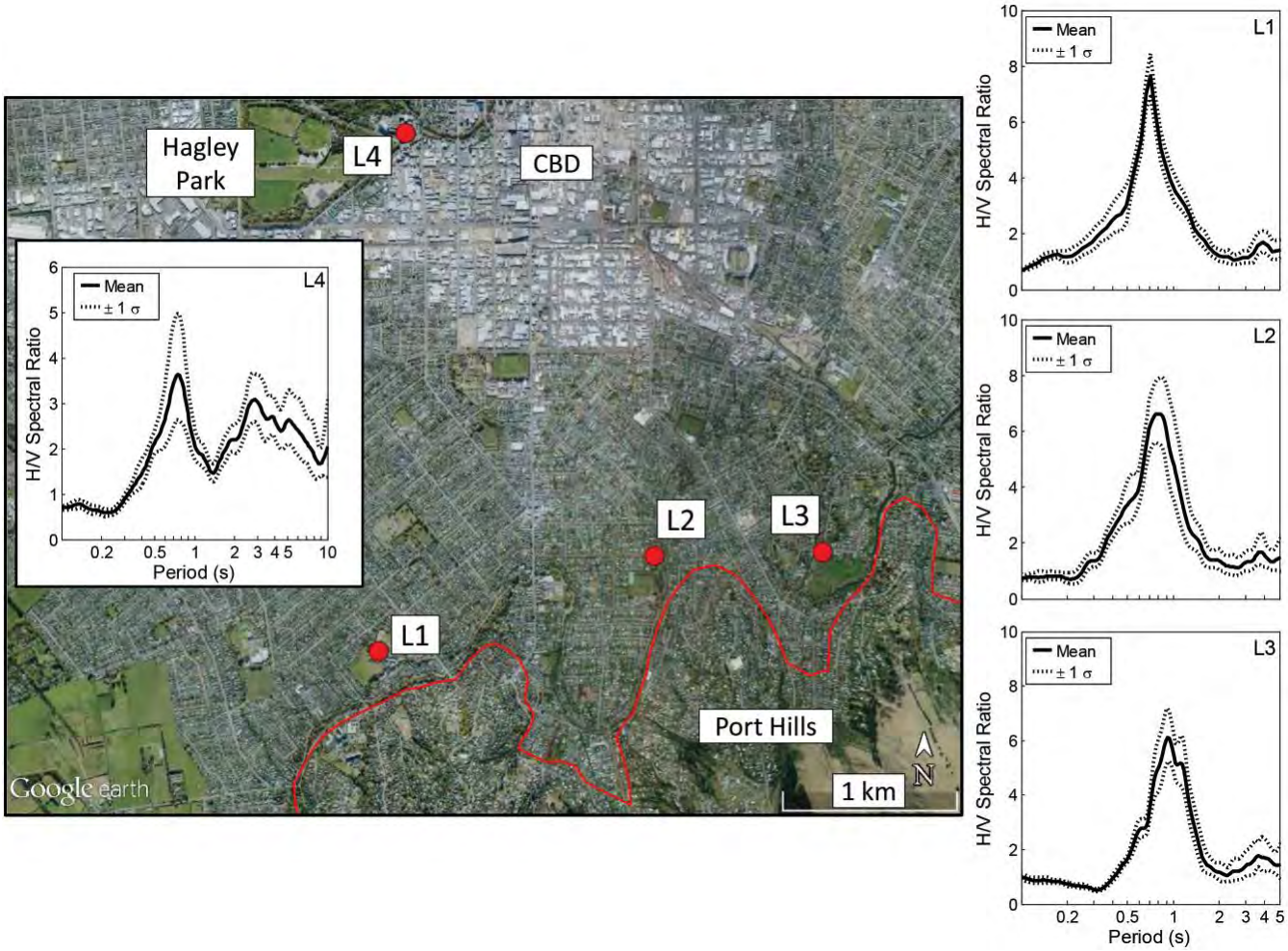

Figure 1: Map of southern Christchurch showing location of H/V spectral ratio measurements referred to in this discussion paper, with the red line indicative of the base of the Port Hills. H/V spectral ratio for locations in the vicinity of the Port Hills at locations L1, L2 and L3, with spectral peaks indicating site periods above bedrock of T=0.7-1.0 seconds. H/V spectral ratio for site L4 near Hagley Park with dominant peaks at 0.74 and 2.9 seconds.

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Above: Prof Antonio Gens presented a welcoming address on behalf of the ISSMGE

THE TECHNICAL HIGHLIGHT of the calendar for geotechnical professionals in New Zealand must be the Australia New Zealand Conference (ANZ) on Geomechanics. Held every four years the conference brings together the best expertise from Australia, New Zealand and beyond to educate, entertain and inform.

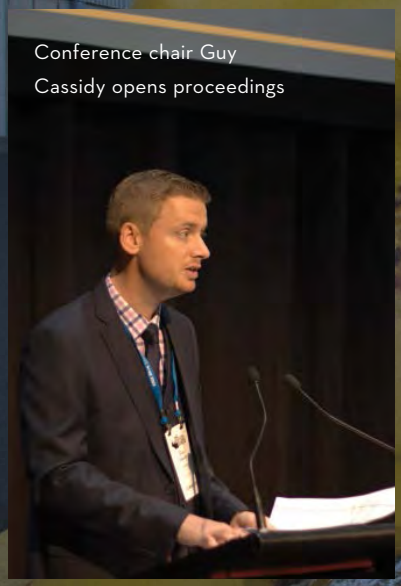
At this years' ANZ conference the presence of the board of the International Society of Soil Mechanics and Geotechnical Engineering for their annual meeting demonstrated the high regard held internationally for this conference.

The last ANZ Conference held in Wellington was in 1980. Thirty-five years later delegates from 26 countries poured into Wellington to drink the best flat whites in the world. Fuelled by a desire to learn and a surfeit of caffeine, the 362 delegates and exhibitors were treated to a smorgasbord of presentations, exhibitions and networking opportunities.

Wellington, like most cities in the world, is experiencing significant change and challenge as a result of mankind. The conference theme 'geomechanics and human influence' reflected this challenge and focussed attention on efficiency and sustainability, while remaining broad enough to provide technical subject matter for almost every interest group.

The keynote speakers, Prof. George Gazetas, Dr. Fred Baynes, Prof. Jonathan Bray and two award lectures, John Carter (AGS's John Jaeger Lecture) and John Wood (NZGS' Geomechanics Award Lecture) lived up to their billing, providing presentations combining technical rigour and entertainment in perfect proportions.

George Gazetas' discussion on 'Avoiding Over-Conservatism and Conventional Dogmas in Seismic Geotechnical Design' was perfectly balanced by Jonathan Bray's 'Turning Disaster into Knowledge'. Between them they discussed the cost of over-conservatism and the balance needed to avoid the horrific consequences of unconservative design, while Fred Baynes enlivened proceedings (and did an amazing job of keeping the delegates alert at the end of a long day) with his inspirational



Conference chair Guy Cassidy opens proceedings

Dr Fred Baynes delivers his keynote speach

The keynote speakers kept the audience captivated

'Deconstructing Engineering Geological Models for Continuous Improvement in a Changing World'. Their work should be considered essential reading for all geotechnical professionals in New Zealand.

The conference committee of Guy Cassidy (Conference Chair), Lucy McChesney (Technical Programme Chair), Bev Curley (Social Programme Chair), Pierre Malan (Sponsorship and Exhibition Chair), Doug Mason (Treasurer), Andrew Kennedy (Social Programme), and Graham Ramsay (Proceedings, Editor) are to be congratulated on running a spectacularly successful event which was greatly enjoyed by all the delegates. They are now deservedly relaxing and enjoying becoming re-acquainted with their friends and families after a number of years' hard work.

During the conference the Australian Geomechanics Society announced that it will again bid to host the International Conference on Soil Mechanics and Geotechnical Engineering. Having narrowly missed out on the 2017 conference to South Korea they now aim to bring the quadrennial conference to Sydney in 2021 with the support of the NZGS. All our members are encouraged to add their support this great initiative to bring such a large and prestigious conference to our side of the world. The quality of the ANZ 2015 conference should certainly have impressed the board of the ISSMGE. Our experience from the conference shows the huge benefit such events bring in learning

Dai Henwood performed a well recieved set at the Gala dinner at Te Papa.

The ISSMGE board were guests of honour.

The audience were still amused

and networking. The strength of our profession can only be improved by hosting such events.

The Opening Address, delivered by Mike Stannard of MBIE, is of particular relevance to all practicing geotechnical professionals. Entitled "How the geotechnical profession can better influence New Zealand resilience", it is reproduced in full below.

"WHAT IS A GEOTECH ENGINEER?

Five years ago, most New Zealanders outside the engineering profession would have struggled to answer that question. That changed in September 2010 with the Darfield earthquake. And again four years ago yesterday the more devastating aftershock that killed 185 people and caused widespread damage.

This catastrophic natural event catapulted the geotech engineer into the spotlight. It also gave geotech engineers a platform from which their community can influence New Zealand's built environment.

Good Morning. On behalf of the Ministry of Business, Innovation and Employment, it's my pleasure to be a sponsor of the Australia New Zealand Conference on Geomechanics and to speak to you today.

Following the earthquakes, the NZ government established a royal commission. This looked at issues around

the adequacy of the current legal and best practice requirements for the design, construction and maintenance of buildings in the context of earthquake risk.

Seven volumes of reports were published with 189 recommendations. Of these, 175 sit with MBIE to execute. About a fifth of these recommendations relate to geotechnical issues and the way geotechs work.

The royal commission sparked a large and critically important work programme for MBIE and we have engaged many external geotechnical advisors to assist. From these interactions and through this work, we have identified four potential ways geotechs can work differently:

First, geotechs can be more proactive about getting involved in land use planning discussions. Geotechs have a responsibility to learn from the Canterbury experience and share what they've learned with building, policy and land-use decision makers to better secure New Zealand's future.

The liquefaction hazard in Canterbury had been reasonably understood for a long time. A number of liquefaction reports and maps had been produced from the early 1990s and liquefaction had been observed in the Canterbury Plains following earthquakes from the late 19th century. But the impact on the development of urban Christchurch was seriously underestimated and large areas of vulnerable land were developed after the 1990s. We don't want to repeat that mistake.

Geotech engineers are now providing expert evidence for the update to the Christchurch City Council's District Plan. But they should be providing this same support to other councils. Geotechs can prevent the red zones of the future by proactively approaching other councils and working with them on district planning policy.

The second opportunity for geotechs is to collaborate on the collection of information.

A project that illustrates this extremely well is the Canterbury Geotechnical Database. This has been a low-key but significant success story in the rebuild of Christchurch. The online database is a searchable repository for existing geotechnical information. It is also a website for all new geotechnical information and supporting geotechnical applications for building and resource consents. It's run on a voluntary, terms-of-use basis. Registered users can download information, but in return must upload new information. We now have assurance that this government owned initiative will not be disbanded at the end of the rebuild and MBIE along with the support from EQC via their research and education mandate will be considering how this can be rolled out for the rest of New Zealand.

Christchurch realised that rather than competing on the collection of information, the information could be shared, and companies could compete on other aspects such as smart design. They became data users, not data gatherers. The scale of the earthquakes and complexity of rebuilding generated a substantial amount of goodwill across the community to share geotechnical information. There simply was neither the time nor resources to work in isolation. Using this approach nationally would ultimately lower building costs and result in a more productive and resilient economy.

My third point is the opportunity for geotechs to increase building resilience by influencing building decisions.

After the quakes, it was apparent that there was insufficient guidance on how to design foundations for buildings experiencing seismic actions. It became clear that specific technical guidance was needed. MBIE sought input from our most experienced engineers and established a technical engineering advisory group through which we could collaborate on solutions. The guidance was designed for Canterbury but the principles can be applied in any high hazard area. As a result, MBIE is now determined that the lessons from Canterbury aren't lost, and that further guidance will be available in collaboration with NZGS on how to design foundations and structures appropriate to site conditions.

Mike Stannard
Mike Stannard is the Chief Engineer at the Ministry of Business, Innovation and Employment (MBIE)

The field trip to the MacKays to Peka Peka bypass, lead by Philip Robins, was very well recieved. These 3m diameter piles were seen in various stages of construction.

Above: Bridge piers in various stages of construction highlighed the many and varied challenges overcome on this project.

My final point is to emphasise the importance of geotechs building stronger relationships with the building community.

MBIE's realised that it needed closer and more formal links with the engineering community. This would enable MBIE to develop speedier solutions, and the results have opened our eyes to the benefits of collaboration.

MBIE recently signed Memorandum of Understandings with both the NZ Geotechnical Society and the NZ Society for Earthquake Engineering. Work is underway to form similar agreements with others. While these agreements are not "contractual" they will do much to create a shared understanding of each organisation's objectives and drivers and help align them.

In summary, the rawness and despair felt in Canterbury and the rest of NZ following the Canterbury earthquake sequence, created a unique opportunity. That is for the geotechnical professional to actively influence the way we build in NZ so we have a more sustainable and robust environment that can better manage future catastrophic events."

NZ Geotechnical Society

2015 PHOTO COMPETITION

The 2015 theme is

Awards

Show us your Award winning style and tell us which award you think you should get - e.g. Best in Show, Darwin Award, Academy Award, Bravery Award ...

ENTRIES CLOSE *October*

Win $250

The winning photo and the top runners-up will be printed in the December 2015 issue of *NZ Geomechanics News*

SEND YOUR ENTRY TO

- Email to: editor@nzgs.org (send as jpgs)
- Entries close 16 October 2015
- Clearly mark your entry with your name and provide a caption for your photo

CONDITIONS OF ENTRY

1. Only amateur photographers may enter.
2. Photos must be taken by the entrant.
3. No computer generated pictures.
4. Any photographs received may be published in subsequent NZ Geotechnical Society publications and material.
5. Winning entries will be final and no correspondence will be entered into.
6. NZ Geotechnical Society members only may enter.

GEOSYNTHETICS 2015 CONFERENCE

Review by:
Michael 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.

THIS YEAR'S GEOSYNTHETICS conference in Portland, Oregon, was widely acclaimed to be a huge success with more than 2,700 attendees and more than 240 exhibitors, shattering previous conference records.

I was privileged to attend this biennial event which explored advances and innovations shaping the geotechnical, Civil and geo-environmental communities. There was a host of educational components including technical sessions and six full-day short courses offering participants professional development hours. Also included in the conference were plenary and technical sessions covering everything from the Oso, Washington landslide to installation techniques, environmental applications and soil improvement. Noted Geosynthetics professionals from around the world conducted the plenary sessions.

A highlight for me was the excellent Plenary Session titled "We will see a significant growth in Geosynthetic use if..." A panel of four industry experts from consulting, academia, manufacturing and contracting, along with significant audience participation held a robust and extremely valuable discussion on what is holding back this excellent technology, and what we can do as an industry to unleash the potential and value. It's been thousands of years since the concept or soil reinforcement was first used, more than half a century since geosynthetics were introduced, more than four decades since they have been widely used in separation, filtration, reinforcement and drainage applications, and 30 years since the formation of the International Geosynthetics Society. Yet many engineers continue to be unfamiliar with the core design principles of Geosynthetics and Geosynthetic-centric systems, with the result that they are still underutilised.

There was a consensus among the panel on 3 key points. We will see significant growth in Geosynthetics use if...

1. We increase Geosynthetic undergraduate education and exposure for all civil engineers, so by the time of graduation they will at least be familiar with the technology
2. We provide integrated solutions that incorporate Geosynthetics into solutions rather than selling isolated product
3. We promote (or create) complete design methods for a broader range of applications than are currently available so design engineers can follow accepted methodologies for more applications

Other points were raised where there was not clear consensus which made for an interesting and stimulating discussion, such as whether there is advantage in geosynthetics being a less proprietary commodity type product or continuing to be highly engineered and design intensive products.

One interesting comparison was raised in the field of geogrid reinforcement, with contrast made between the use of geogrids in reinforced soil structures and slopes, and geogrids in pavement construction. The industry representing Reinforced Soil Walls has cooperated significantly over the years, with well-defined and accepted design methods and programs by the likes of the NCMA and ADAMA engineering among others. Properties required by the geogrid are well understood and standard test methods established. This has led to significant growth in the market segment and general acceptance by almost all government authorities. Conversely, the market for pavement geogrids is fraught with competing claims by leading players who cannot even agree on the form of the Geosynthetic required, let alone the index properties required. This leads to vague marketing practices relying on generalised "my product versus their product" type claims which do not necessarily represent the weight of international research. As a result the market is under-developed and somewhat stagnated and the engineering community, getting mixed messages about what is best practice, is tending to promote other technologies. However even in this field there is progress and we now have reputable and non-commercial research available.

Earth Pressure and Earth Retaining Structures – Chris Clayton, Rick Woods, Andrew Bond, Jarbas Milititsky

Review by:
Kevin Anderson
Kevin has specialised in geotechnical and tunnel engineering since graduating in Glasgow in 1996. After 5 years working in Scotland, he has been based in Auckland, working in most sectors of civil engineering and many projects across NZ and overseas. Kevin has been an NZGS Management Committee member since 2013 and has spearheaded progress on a number of Seismic Design Guidelines. Kevin became CPEng in 2005 and is a practice area assessor for IPENZ.

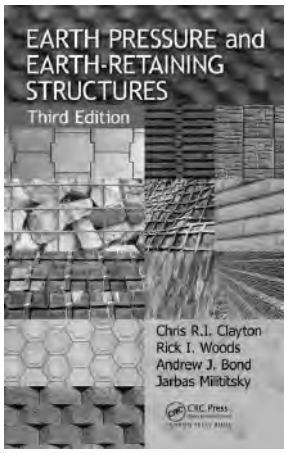
ALMOST 30 YEARS have passed since this well-known book on retaining walls was first published. The third edition includes new material, for example on soil nails and anchored walls, as well as providing commentary on Eurocode 7. The book has also been re-organised into two main parts; fundamentals and design.

As the title suggests, the book focuses on earth pressure and retaining structures. These sections are excellent and there is a well balanced mix of introduction, theory, design practice and recommendations. Some worked examples are threaded throughout the book. The style of the text and the layout of the book lead to easy reading and is generally straightforward to follow. Typically, it starts with a description of the problem, suggests the solution, then discusses the issues before the design aspects and details. This is a book written for the design engineer rather than an academic treatise.

The book by no means covers everything. The soil mechanics section is a good introduction to the subject, but rather limited and does not discuss heterogeneous soils let alone volcanic, residual or tropical soils. There is little mention of the geological model, geomorphology or earthquakes. The recommendations for geotechnical investigations would rarely be achieved in any country but there are good suggestions for investigating uphill and downhill of the alignment. The discussions on software are also quite limited and generic.

These limitations should not discourage the reader. The coverage of different wall types is excellent and there are a lot of useful equations and charts for the design of many walls, particularly the gravity and embedded walls. The discussion of evaluating safety and the different methods is very thorough and educational, for example: Eurocode 7 (limit state partial factors); mobilised strength; and various ways of deriving and using the factor of safety such as gross pressure, net pressure, embedment and soil strength.

Overall, this book is well worth having on the desk as it provides many useful equations, chart and recommendations to help with determining earth pressure and designing retaining walls. It should not be assumed to cover all aspects of design, particularly for the New Zealand environment. However, the topics covered are generally presented very well. The style of the book is a particular strength as it is written for the retaining wall designer.



Title	Earth pressure and earth retaining structures
Authors	Chris Clayton, Rick Woods, Andrew Bond, Jarbas Milititsky
Publisher	CRC Press
Year Published	3rd edition, 2013
Length	476pp plus about 100 appendices
ISBN	9781466552111
Web shopping	https://www.crcpress.com/product/isbn/9781466552111
Price	Paperback £35, eBook £24.50 or £16 for 6 months

NZGS Awards

NEW ZEALAND GEOTECHNICAL SOCIETY GEOMECHANICS LECTURE

The NZ Geomechanics Lecture is the premier award of the New Zealand Geomechanics Society. It is presented by a person prominent in Geomechanics who can, in the presentation, contribute a statement of significance and value relevant to New Zealand. The lecture is to be presented at intervals of up to four years at a minimum of three venues in New Zealand and is promoted to attract as wide an audience as possible. Following its presentation, the lecture is to be published.

NEW ZEALAND GEOTECHNICAL SOCIETY GEOMECHANICS AWARD

The New Zealand Geotechnical Society Geomechanics Award is awarded every three years and shall be presented at the Society's Annual General Meeting. The award shall be made to the Society member or members producing the adjudged "best" published paper during the three years ending 31 July preceding the date of the Award, in any publication at the discretion of the Management Committee. The winning paper will be that considered to be distinguished in its contribution to the development of geotechnics in New Zealand.

NEW ZEALAND GEOTECHNICAL SOCIETY SCHOLARSHIP

The NZGS Management Committee has agreed to provide funding for a scholarship that would enable a member of the Society to undertake postgraduate study in New Zealand that would advance the objectives of the Society. Through this scholarship, the Society hopes to encourage members to enrol for post-graduate study or research.

NEW ZEALAND GEOTECHNICAL SOCIETY SCHOLARSHIP
Applications for consideration by the Committee should be submitted to the Management Secretary by 31st October. Check the website for full details http://www.nzgs.org/awards

NEW ZEALAND GEOTECHNICAL SOCIETY YOUNG GEOTECHNICAL PROFESSIONALS FELLOWSHIP

This fellowship is awarded to the author of the best paper by a New Zealand representative at each Australia-New Zealand Young Geotechnical Professionals conference. The recipient must be a member of the New Zealand Geotechnical Society and be below the age of 35 at the time of presenting the paper at the conference.

YOUNG GEOTECHNICAL PROFESSIONALS CONFERENCE AWARDS

The Earthquake Commission Research Foundation and the NZ Geotechnical Society have awards available for New Zealanders attending the Young Geotechnical Professionals Conference.

NEW ZEALAND GEOTECHNICAL SOCIETY STUDENT AWARDS

The New Zealand Geotechnical Society Student Awards are presented to recognise and encourage student participation in the fields of geotechnical engineering and engineering geology. The details of this award are currently being reviewed, as discussed on page 101 by the Awards Officer, Sally Hargraves.

For more details please visit the Awards page on the NZGS Website and watch for Society announcements about closing dates, changes to awards and winners.

IPENZ Fellow

NZGS MEMBERS CAN be heartened by the news that C Y Chin (known as Chin) was among the recently announced 2015 IPENZ Fellows. This prestigious level of IPENZ Membership, recognising outstanding individual engineering achievement and contributions to the profession, couldn't be bestowed on a more worthy, or humble, fellow. For the purposes of this article, a reluctant Chin was questioned by his colleague and friend Geoff Farquhar, and the following is the resulting compilation of several talks.

Chin admits that he has a 'naturally inquisitive' personality. He does not rest until he understands how things work, and he credits his early days at Arup where he learned to enquire when faced with a problem. He suggests that 'Arup people were not proud and they listened to what one had to say. Then, everyone got together to solve the problem'. Chin believes this mindset helped nurture and cement his enduring inspiration for research.

When asked what advice he would have given himself at age 25, Chin immediately suggested undertaking a postgraduate degree. He recalls that Arup elders advised him to complete an MSc DIC at Imperial College, to 'bring his knowledge up to scratch'. Instead, he decided to apply for a PhD and connected with Professor Malcom Bolton. He remembers that he discovered how little he knew, and insists that after 25 years he still doesn't know all the answers. His admiration of Professor Bolton continues and he suggests this is due to the Professors' 'ability to see through the fluff and get to the root of a problem and his ability to explain things in simple terms'. Chin maintains that learning and researching continue to be important to him, and perhaps some of this passion can be traced back to his time with Professor Bolton.

Perhaps, not surprisingly, Chin maintains that during his years as a volunteer on the Management Committee he is most proud of establishing and developing an NZGS Scholarship for research purposes. The scholarship funding enables a member of the Society to undertake postgraduate research in New Zealand that would advance the objectives of the Society.

We asked Chin what key lessons he has learnt from the many projects he has worked on:

- He described how he learnt to better appreciate and understand *geology* from two projects. Firstly, the US$1B Nam Theun 2 hydroelectric project in Lao, where the tunnels were designed in Karst formation and subsequently there were difficulties with tunnel boring in those formations. Secondly, the significant rock profile variations in hydrothermally altered Andesite at Te Puru resulting in changes in bored pile rock designs. Chin stresses the importance of having an experienced geologist to provide sound geological advice.
- Chin told how he learnt not to underestimate *topographic amplification* effects during a seismic event resulting in large vertical and horizontal accelerations. He observed first-hand the damage to residential properties at Port Hills, Christchurch following the recent earthquakes.
- Another lesson was to undertake *sensitivity runs* for geotechnical analyses which reflect likely construction tolerances. Sometimes, achieved construction tolerances can cause a design to fail. Chin says redundancies in geotechnical design are becoming uncommon as commercial pressures mount to provide the cheapest solution. He suggests it is useful to pause and ask ourselves what the weakest link is in the solution presented and what if this fails?
- Chin notes that geotechnical engineers often deal with high

CY Chin

Chin is on secondment to the University of Canterbury Quake Centre researching the seismic design of retaining walls using OpenSees with a view of determining appropriate fractions of PGA for pseudo-static determination of dynamic earth pressures for acceleration time-histories appropriate to New Zealand. He is an editor for IAEG's Bulletin of Engineering Geology & the Environment and is on the editorial board of the IPENZ Transactions. He has particular interest in forensic geotechnical engineering and seismic engineering.

levels of uncertainty and risk as designs are frequently based on interpretations of boreholes and soil tests done on a minute proportion of the land being considered. Advice is subsequently given based on judgement and experience. Chin has found it invaluable to have *experienced reviewers* early on in the design process provide feedback and a second opinion on interpretations and solutions.

Chin believes that one of the most important aspects of geotechnical engineering in the future will be finding better solutions for the geotechnical effects of seismicity. We suspect that Chin's high standards, innovation, excellence and commitment to the engineering profession will be a valuable part of that knowledge journey.

We congratulate Chin on his recognition as a Fellow of IPENZ, on his unwavering support and dedication to NZGS, and on his enthusiasm for his profession.

NEW ZEALAND GEOTECHNICAL SOCIETY SCHOLARSHIP

Applications for consideration by the Committee should be submitted to the Management Secretary by 31st October. Check the website for full details here http://www.nzgs.org/awards/new-zealand-geotechnical-society-scholarship.htm

Student Awards

BEING A NEW member of the NZGS Committee, I've been tasked with getting up to speed with the NZGS awards, past and present; and to determine if there are opportunities for improvement.

One of the areas targeted for a review is that of Student Awards. There have been a few different systems tried over the years to try and encourage more participation, not always successfully. In 2012 the awards were altered to a poster competition. Students of recognised tertiary institutions in New Zealand were encouraged to submit an abstract then prepare an A1 size poster that clearly and concisely presents their work on any aspect or topic in the fields of geotechnical engineering or engineering geology. The award is open to both undergraduate and postgraduate students and is judged by both a panel and a vote of attending members, with 1st, 2nd and 3rd places being awarded large cash prizes. Despite this, entry numbers have been disappointing.

Prior to this, the award was for the best presentation of a relevant topic, with the judgement being on the clarity of their presentation as well as the quality of a one page written synopsis submitted prior to the presentations. A winner was selected from each of the North and South Islands.

Having now touched base with each of the relevant Tertiary institutions in New Zealand, I've discovered that timing is one of the key factors; as is consistency across all fields of research and university departments. It may be perhaps that the award can be morphed to fit around judging the final year projects. An element of presentation is considered important as the communication of problems and solutions would be a key component. There will be another round of discussion around this topic before it's finalised. Any comments or suggestions would be warmly received.

Sally Hargraves
Sally is an engineering geologist who has recently set back out on her own. She completed a BSc (Hons) in Geology at the University of Birmingham in the UK and followed it up with a PhD in Slope stability analysis – developing a subroutine for an existing computer model to add in the effects of geogrid reinforcement (in Fortran and then C++).
sally@tfel.co.nz

NZGS Young Geotechnical Professionals

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!
ygp@nzgs.org

PROFESSIONALS (YGP) GROUP has been formed to represent, support and provide a voice for the young professionals in the NZGS. We represent a lively, increasingly influential and rapidly growing section of Geotechnical Engineers and Engineering Geologists nationwide. Through a social culture of innovation, integrity, networking and the pursuit of excellence, we anticipate facilitating in the professional and personal development of the young professionals. If you are a NZGS member under 35 years of age, you are automatically a YGP!

LATEST ACTIVITIES:

Student Awards

This year the NZGS committee are reviewing the Students Awards and our newly appointed Awards Co-ordinator Sally Hargraves has been hard at work liaising with the relevant tertiary institutions. Once this consultation is complete we'll advertise the 2015 Student Awards.

YGP Events at ANZ2015

YGPs at ANZ2015 were fortunate to benefit from a small interactive session with Geoff Farquhar of URS/AECOM and Mark Eggers of Pells Sullivan Meynink. During this session Mark addressed issues such as Professionalism and Leadership. This was great food for thought and no doubt the attendees all came away with key messages such as taking opportunities as they arise and that getting the right mentoring at the start of our career is fundamental. Mark also introduced the concept of quiet leadership and illustrated that you don't have to be an extrovert to be a successful leader. Geoff provided his views in more of a Q&A style session directed by questions from YGPs and in particular addressed queries regarding CPEng and PEngGeol registration.

I would like to thank both Mark and Geoff for their "unplugged" presentation; it was thought provoking and provided many enlightened answers to common queries posed by YGPs.

Regional Events

We are still keen to hear from YGPs about ideas for regional events so if you have any suggestions please get in contact. We've got with the times and now have a dedicated Facebook page so please visit and like the page https://www.facebook.com/nzgsygp. Don't forget to keep an eye out for, and support events in your area!

11th ANZ YGP Conference

Planning is underway and a small committee formed for the 11th Australia New Zealand YGP Conference to be held in New Zealand in 2016. 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 conference is also a lot of fun and I encourage all YGPs to start thinking about preparing a paper for the conference. Keep an eye on your inbox and the December issue of Geomechanics News for further details. To vote for your preferred conference location, visit our Facebook page!

International Society of Soil Mechanics and Geotechnical Engineering

ISSMGE BOARD MEETING, 12 SEPTEMBER 2014

The third Board Meeting of the term of the new President, Prof. Roger Frank, was held in Wellington, NZ and coincided with the 12th ANZ conference. The Board very much appreciated the warm welcome and hospitality given by the NZGS and AGS.

WEBINARS

A webinar was presented on Feb 1, 2015 by Peter Robertson from the USA entitled In-situ *Testing Using the CPT*. Past webinars are available from: http://www.issmge.org/en/resources/recorded-webinars.

The next webinar will be presented on April 15 by Prof. Misko Cubrinovski from the University of Canterbury on the Impacts of Liquefaction in the 2010-2011 Christchurch Earthquakes.

INNOVATIONS AND DEVELOPMENT COMMITTEE

The IDC, under the leadership of Dimitios Zekkos from the USA, is progressing several initiatives, such as uploading more digital versions of conference proceedings on the ISSMGE website. ISSMGE members are encouraged to submit papers to the *International Journal of Geoengineering Case Histories* (see http://www.issmge.org/en/resources/international-journal-of-geoengineering-case-histories).

TECHNICAL COMMITTEES (TCS)

The ISSMGE currently has 32 TCs examining a wide range of geotechnical topics. Whilst members of the NZGS represent a number of TCs, many have no NZGS representation. If you are interested in representing the NZGS on any of the TCs below, please contact Prof. Mick Pender (m.pender@auckland.ac.nz), the NZGS's ISSMGE Liaison, and also send him your CV. As stipulated in the ISSMGE's Technical Society Guidelines:

> *Each Member Society may appoint up to two (2) members to serve on the TCs... The Chair may also accept more than two members from a Member Society. These additional nominations are considered as Corresponding Members. The corresponding members may attend TC meetings but have no voting rights.*

TCs with NZGS representation:

TC203 – Earthquake*; TC204 – Underground Construction; TC206 – Interactive Design; TC211 – Ground Improvement; TC212 – Deep Foundations; TC217 – Land Reclamation; TC302 – Forensic.

(*: 2 representatives + 1 corresponding member. All others have one representative)

Details of each TC are given at: http://www.issmge.org/en/committees/technical-committees.

The revised *Guidelines for ISSMGE Technical Committees and ISSMGE Honours Lectures* were approved at the Wellington Board meeting and were distributed to TCs on March 2 and are available from http://www.issmge.org/en/committees/technical-committees.

ISSMGE FOUNDATION

The Foundation was established a little over 5 years ago and was created to provide financial help to geotechnical engineers throughout the world who wish to further their geotechnical engineering knowledge and enhance their practice through various activities, which they could not otherwise afford. These activities include attending conferences, participating in continuing education events and purchasing geotechnical reference books and manuals. The financial assistance is in the form of a "bursary" which the successful applicant can use to pay registration costs, and travel and accommodation expenses, for example.

ISSMGE members are encouraged to seek assistance from the Foundation. Details are given at http://www.issmge.org/en/issmge-foundation/application-form.

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.

International Association for Engineering Geology and the Environment

THE LATEST IAEG meetings were held in conjunction with the 12th IAEG Congress in Turin, Italy in September 2014. Ann Williams as Australasian Vice President in 2014 attended the Executive meeting while both Ann and Mark Eggers (our new Australasian VP) took part in the Council meeting held on 13 and 14 September 2014 respectively. The election of the Executive Committee took place in the Council meeting by secret ballot. The new office members of the Executive Committee are listed below.

- President: Scott Burns (USA)
- Secretary General: Faquan WU (China)
- Treasurer: Jean-Alain Fleurisson (France)
- VP for Asia: Yogendra Deva (India)
- VP for Africa: Louis van Rooy (South Africa)
- VP for Australasia: Mark Eggers (New Zealand and Australia)
- VP for North Europe: Rafig Azzam (Germany)
- VP for South Europe: Giorgio Lollino (Italy)
- VP for North America: Jeffery Keaton (USA)
- VP for South America: Maria Heloisa B. Oliveira Frascá (Brazil)

REPORT OF THE 12TH IAEG **CONGRESS, TURIN ITALY SEPTEMBER 2014**

The 12th IAEG Congress was held in the Lingotto Conference Centre in Torino, Italy on 15th - 19th September, 2014. The Congress was attended by more than 1060 delegates from over 60 countries. The Congress also celebrated the 50th Anniversary of the IAEG. The theme of the Congress was "Engineering Geology for Society and Territory" with 8 topics, 21 keynote speakers, 141 sessions, 14 workshops and 7 field trips. The Congress received more than 1760 abstracts with the proceedings published by Springer in a series of 8 volumes.

As part of the anniversary celebrations the **IAEG 50th Anniversary Book** was published as a reflection on the past, present and future of engineering geology and IAEG. The book describes the situation that motivated the founding of the IAEG, its evolution throughout the years, the people who have directed it, the services provided by its publications, congresses and meetings, and the awards given. It also reflects on the Association's present situation among the societies of earth sciences and geoengineering and its perspective for the future.

In the awards ceremony, IAEG awarded the **Hans Cloos Medal 2014** to Prof. Roger Cojean (France). The Hans Cloos Medal is the senior award presented by IAEG, given to an engineering geologist of outstanding merit in commemoration of the "founder of geomechanics". The recipient should therefore be a person of international repute who has made a major contribution to engineering geology in his/her written papers or to the development of engineering geology. The medal is awarded every two years.

IAEG awarded the **Marcel Arnoud Medal 2014** to Brian Hawkins (UK), editor-in-chief of IAEG Bulletin from 1998 to 2012. The first time this Medal has been awarded, the Marcel Arnould Medal, awarded every two years, is named after one of the founders of IAEG, a former President and Honorary President, and as such is awarded to an IAEG member who has made a significant contribution to the engineering geology profession in their region and given outstanding service to IAEG.

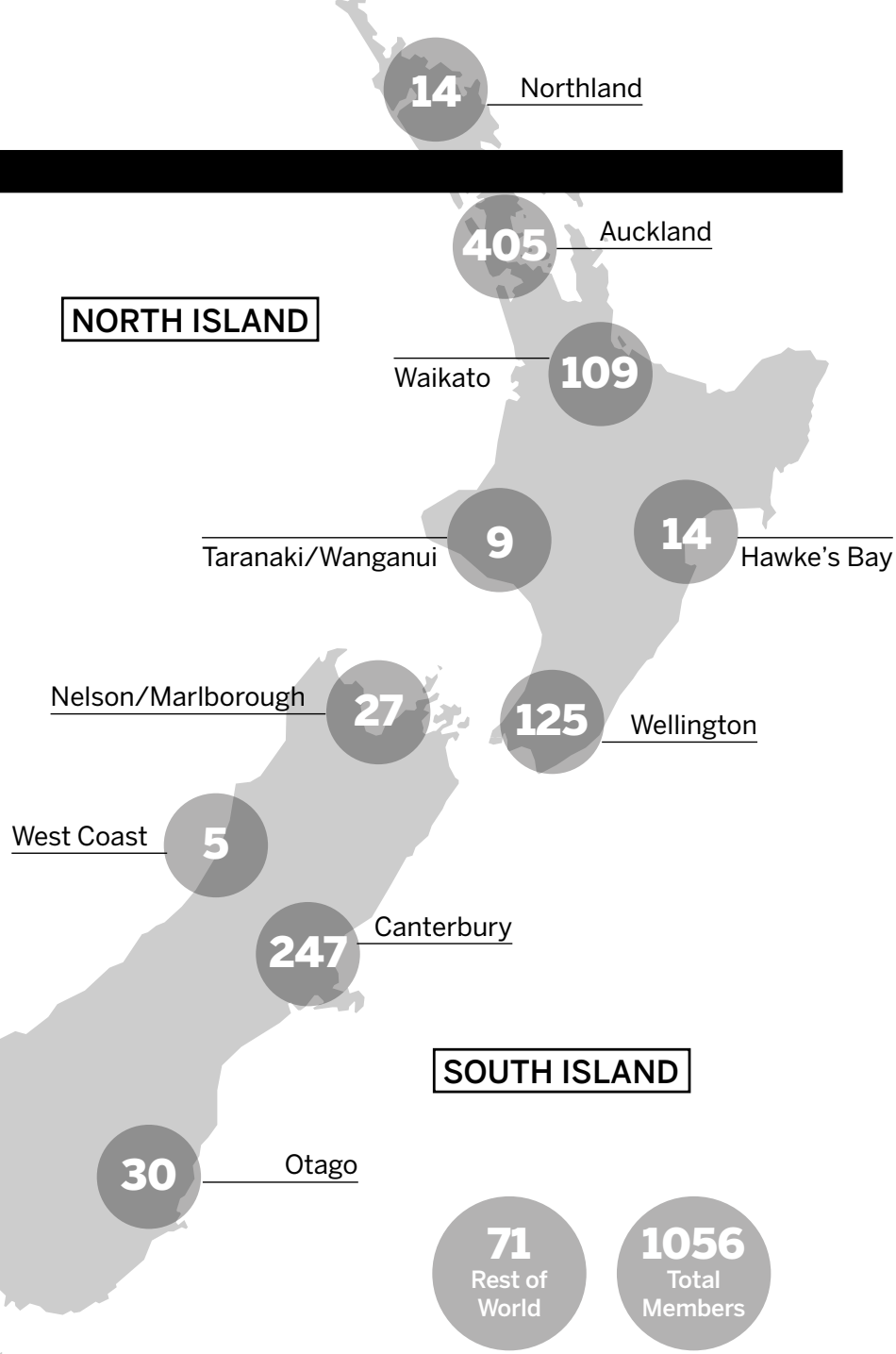
Mark Eggers
Mark is the IAEG Vice President for Australasia. His career started as a field geologist for the NZ Forestry Service, before moving to Australia where he teaches geotechnical engineering and engineering geology at the University of New South Wales.

The **Richard Walters Prize** contest was held on 16 September at the Congress. The Jury was led by Prof. Osipov (Russia) and the Jury members were Resat Ulusay (Turkey), Jeffery keaton (USA), Chigira Masahiro (Japan), Silvina Marfil (Argentina) , Rafig Azzam (German) and John Stiff (South Africa). There were 6 qualified candidates nominated by National Groups including Chris Massey from GNS New Zealand. The prize was won by Louis N.Y. Wong from Nanyang Technological University in Singapore.

During the Congress a number of presentations delivered in the main hall were video-recorded and the recordings are available on the IAEG website: http://www.iaeg.info/iaeg2014/web-streaming/

The 13th IAEG Congress will be held in San Francisco, California in 2018.

Branch reports



AUCKLAND

Auckland branch has a new face in Eric Torvelainen. Eric is a geotechnical engineer from Tonkin & Taylor and replaces Pierre Millan, joining Luke Storie and Kim Rait.

There have been some great presentations in the Auckland region of late. In February Jim Griffiths presented the 14th Glossop Lecture entitled Feet on the Ground: Engineering Geology, Past, Present and Future to a packed auditorium at Beca. **In April**, Greg Hempen (AECOM, USA) very kindly offered to present while on holiday in NZ. Greg was awarded the Jahn's Distinguished Lecturer in 2013/2014 by the USGS and spoke to us about the creation and use of seismic hazard maps developed from Modified Mercalli Index data. The Auckland branch has also recently hosted Guy Houlsby, who presented his Rankine Lecture as part of his tour around NZ and Australia.

Several technical presentations are in the pipeline for this year. **In June**, Tam Larkin will present on the topic of the paper he and Chris Van Houtte won the 2014 NZGS Geomechanics Award for – Determination of Site Period for NZS1170. Later in the year we hope to provide a presentation in regard to the new MBIE guidance documents on retaining walls. Other exciting presentations to come include various project updates from across the region and academic presentations to keep the branch up to date with new and innovative ideas.

WAIKATO

It has been a busy few months for the NZGS Waikato branch. We had possibly the best turnout to a branch event for a talk on **Volcanic Soils** by Evan Giles and Wataru Okada. It was an excellent talk that stimulated an active discussion afterwards. A small group from the Branch recently had the opportunity to visit the Rangiriri Section of the Waikato Expressway. This site visit, organised by Tom Bunny of MWH was an excellent reminder of some of the challenges presented by the soils of volcanic origin discussed in the previous talk. The talk finished at the Rangiriri Pa which was a great place to reflect on the impact that infrastructure and development has had on significant cultural sites and the ways in which these sites can be protected and preserved for future generations with clever engineering.

A number of talks are in the pipeline including a talk on a suspected new fault found running under Hamilton (June 10th). Keep us posted if you have an interesting talk to share. Kori Lentfer has taken leave from his role as branch coordinator while he and his family support their son recovering from a sudden illness. We wish them all the best and a speedy recovery. Any questions or suggestions for branch activities should be directed to **Andrew Holland, andrew@hdc.net.nz, 0220488441.**

HAWKES BAY

The Hawkes Bay NZGS Branch co-sponsored (with the IPENZ

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

Above: NZGS field trip group looking at Waimea Fault exposure beside the Richmond East High Level Reservoir Access Road

Hawkes Bay Branch) a site visit and presentation on a high-profile redevelopment project in Havelock North called "The Village Exchange" on 22 April. The event was very well attended, and both the site visit and presentations (by Strata Group and Tonkin & Taylor) were very interesting and informative.

Upcoming events:
We are planning a meeting to discuss "limitations" and "use of report" wording in geotechnical report with an aim to better understand how we need to establish the purpose and limitations of our reports for the particular project studied. This meeting is tentatively scheduled for June/July 2015, with the discussion being facilitated by William Gray of Opus, who has been presenting on Geotechnical Assessments in Support of Land Development courses for IPENZ.

We are also planning to co-sponsor a professional development event with the local IPENZ Engenerate branch. This event would see young geo-professionals and engineers presenting to a group about a current project they are working on. The presentations would be brief, with an award(s) given to the best presentation(s). The aim would be to help young geo-professionals and engineers to develop their professional public speaking skills and to network with other like-minded professionals. Members of the IPENZ Branch would be sitting on a panel to help judge the presentations.

WELLINGTON
The Wellington Branch has enjoyed a number of evening and lunchtime presentations recently including:

- Ingenuity and Intelligent Risk Assessment for Resilient Geotechnics presented by Dr. David Oliveria. David presented via live-streaming from Sydney, Australia and his presentation is available to be downloaded / viewed via the NZGS website.
- 3D Groundwater Modelling to Predict the Impact of the Proposed Waikato Expressway presented by Chris Woodhouse. Chris gave an interesting presentation discussing some of the hydrogeological complexities associated with the development of the expressway.
- An Extensive Geotechnical Exploration for the California High Speed Rail presented by Janet Kan. A great discussion of a large scale engineering project through diverse geological conditions.
- 54th BGA Rankine Lecture Interactions in Offshore Foundation Design presented by Prof. Guy Houlsby. Using various case studies, Guy demonstrated how a multidisciplinary approached can enrich geotechnical engineering.

As usual, all of the above events were well attended and we would like to thank our presenters, and members and sponsors alike, for their ongoing support. It is much appreciated. To help us continue this great start to the year, we kindly ask for members to get in touch with any ideas they may have for presentations.

Over the coming months we have a few events in the pipeline.

- (possible) McKays to Peka Peka project field trip for Wellington NZGS members.
- (possible) PEngGeol information evening

NELSON
A field trip on **18 March** was well attended by 17 local NZGS members representing a good mix of local consultant geotechnical engineers and engineering geologists.

The field trip was led by Paul Wopereis, Principal Engineering Geologist MWH, and visited the new Richmond East High Level Water

Above: Waimea Fault showing Richmond Group sedimentary rock (on right) thrust up over fan gravels and Bishopdale Conglomerate.

Reservoir Project being constructed for Tasman District Council by Hawkins Construction and designed by MWH New Zealand Ltd. The new 20m diameter steel reservoir tank is situated on a cut platform at 120 masl in the foothills behind Richmond. The field trip also looked at two new exposures of the Waimea Fault (revealed on the reservoir access road), a landslide exposure and discussed cut batter instability issues.

A further field trip is planned for later in the year.

CANTERBURY

The Canterbury branch has seen a few changes recently. Shamus Wallace 'retired' at the beginning of the year and was farewelled at the AGM by the Committee and local attendees. The Branch welcomed Sam Glue, who has quickly come up to speed. Our second long-serving co-ordinator, has now also resigned... moving on to greener pastures. Ed Ladley, also after many years of dedicated service, is moving with his family to Nelson. The new recruit, Tim Farrant, a geotechnical engineer from Riley Consultants joins Sam (now the 'old hand'). We wish both Shamus and Ed well for the future, and thank them for their dedication and fine management of the Canterbury Branch in recent years.

In February Jim Griffiths presented the 14th Glossop Lecture entitled Feet on the Ground: Engineering Geology, Past, Present and Future at the University of Canterbury. In March, Christchurch hosted the NZGS AGM and followed it up with a presentation by Josef Tootle (ENGEO, USA) on the Treasure Island, San Francisco development from a US naval base to a major residential/commercial and parkland development over historical reclaimed land. The Christchurch branch has also recently hosted Guy Houlsby, who presented his Rankine Lecture as part of his tour around NZ and Australia.

In June, we will be running a mini ANZ2015 conference with a selection of presentations completed by members in the Canterbury area. This is a great chance to catchup on the conference if you were unable to make it in February.

OTAGO

A meeting of the Otago Branch was held in Dunedin on 21st April 2015. Lee Paterson of MWH presented a thought-provoking account of instability problems encountered on the main road on Otago Peninsula, geotechnical design solutions, and some lessons learned.

★ GEO-NEWS WEEKLY E-NEWSLETTER ★

Our new weekly email lists all notices and Branch announcements normally sent to members, but in one email. Please send items to include to **secretary@nzgs.org**

NZGS will be at the 6ICEGE in Christchurch in November. Please come and say hello, meet some of the exhibitors and of course, have a great time at the Conference. Don't miss this big international event in beautiful Canterbury!

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

Tim Farrant
Tim is a Geotechnical Engineer with Riley in Christchurch. As a Christchurch local, Tim studied Civil Engineering at the University of Canterbury, graduating with a BE (Civil) in 2011. Since then Tim has been actively engaged with the Canterbury earthquake recovery, gaining 4 years of geotechnical earthquake engineering experience in Christchurch.
tfarrant@riley.co.nz

Sam Glue
Sam is a Geotechnical Engineer working for Tonkin & Taylor in Christchurch with 9 years experience working throughout New Zealand and Australia. Sam graduated from Canterbury with a BE (Civil) in 2006 and is passionate about being involved in the construction of major infrastructure projects that will withstand the test of time and earthquakes.
SGlue@tonkin.co.nz

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 is the NZGS Secretary and works from home in Glendowie, Auckland. Amanda enjoys assisting the Society and its members keep up to date with local and international society news, Committee activities, and conference and branch events. She manages the Society website and assists with the production of the biannual Bulletin. In her spare time she enjoys base jumping, deep sea caving and Antarctic trekking...hang on, she means reading, cycling and cooking.

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

Management committee

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Elected Member	Ken Read	Ken.Read@opus.co.nz
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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/

ADVERTISING

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

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National and International Events

2015

13-17 SEPTEMBER, 2015
Edinburgh, Scotland
XVI European Conference on Soil Mechanics and Geotechnical Engineering

24-25 SEPTEMBER, 2015
Island of Ischia, Italy
4th Workshop on Volcanic Rocks and Soils. An ISRM Specialised Conference.

26-27 SEPTEMBER, 2015
10th memorial Asian Regional Conference
Kyoto JAPAN

7-10 OCTOBER, 2015
Salzburg, Austria
EUROCK 2015 - 64th Geomechanics Colloquy, An ISRM Regional Symposium.

13-16 OCTOBER, 2015
Rotterdam, The Netherlands
5th International Symposium for Geotechnical Safety and Risk ISGSR 2015

18-21 OCTOBER, 2015
Brazil
ABGE - 15th CBGE (Environmental and Engineering Geology Brazilian Congress)

27-29 OCTOBER, 2015
New Delhi (NCR), INDIA
International Conference on "Engineering Geology in New Millenium" (EGNM)

2-4 NOVEMBER, 2015 ★ Christchurch, New Zealand ★
6th International Conference on Earthquake Geotechnical Engineering (6ICEGE)

3-5 NOVEMBER, 2015
Brussels (Belgium)
2nd International Workshop on Geomechanics and Energy

9-13 NOVEMBER, 2015
Fukuoka, Japan
New Innovations and Sustainability

15-18 NOVEMBER, 2015
Buenos Aires, Argentina
Sixth International Symposium on Deformation Characteristics of Soils

26-28 NOVEMBER, 2015
Tirana, Albania
Geo-Environment and Construction European Conference

3-4 DECEMBER, 2015
Singapore
International Conference for Soft Ground Engineering ICSGE2015

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

25-28 MAY, 2016
Reykjavík, Iceland
The 17th Nordic Geotechnical Meeting

31 MAY – 3 JUNE, 2016
Subang Jaya, Malaysia
19TH Southeast Asian Geotechnical Conference & 2Nd Agssea Conference

12-19 JUNE, 2016
Napoli, Italy
12th International Symposium on Landslides

25-27 JULY, 2016
Shandong, China
4th GeoChina International Conference 2016

1-6 AUGUST, 2016 AT 9.00AM
New Delhi (NCR), INDIA
6th International Conference Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics

29-31 AUGUST 2016
Cappadocia, Turkey
EUROCK 2016, An ISRM Regional Symposium

4-7 SEPTEMBER, 2016
Guimarães, Portugal
3rd International Conference on Transportation Geotechnics

5th International Conference on Geotechnical and Geophysical Site Characterisation
5-9 September, 2016
Queensland, Australia

1 OCTOBER 2016
Bali, Indonesia
ARMS 9 – the 9th Asian Rock Mechanics Symposium, an ISRM Regional Symposium

2017

12-17 FEBRUARY, 2017
Cape Town, South Africa
AfriRock 2017 - International Symposium

13-15 JUNE, 2017
Ostrava, Czech Republic
International Symposium EUROCK 2017

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