DECEMBER 2009 issue 78

NZ GEOMECHANICS NEWS

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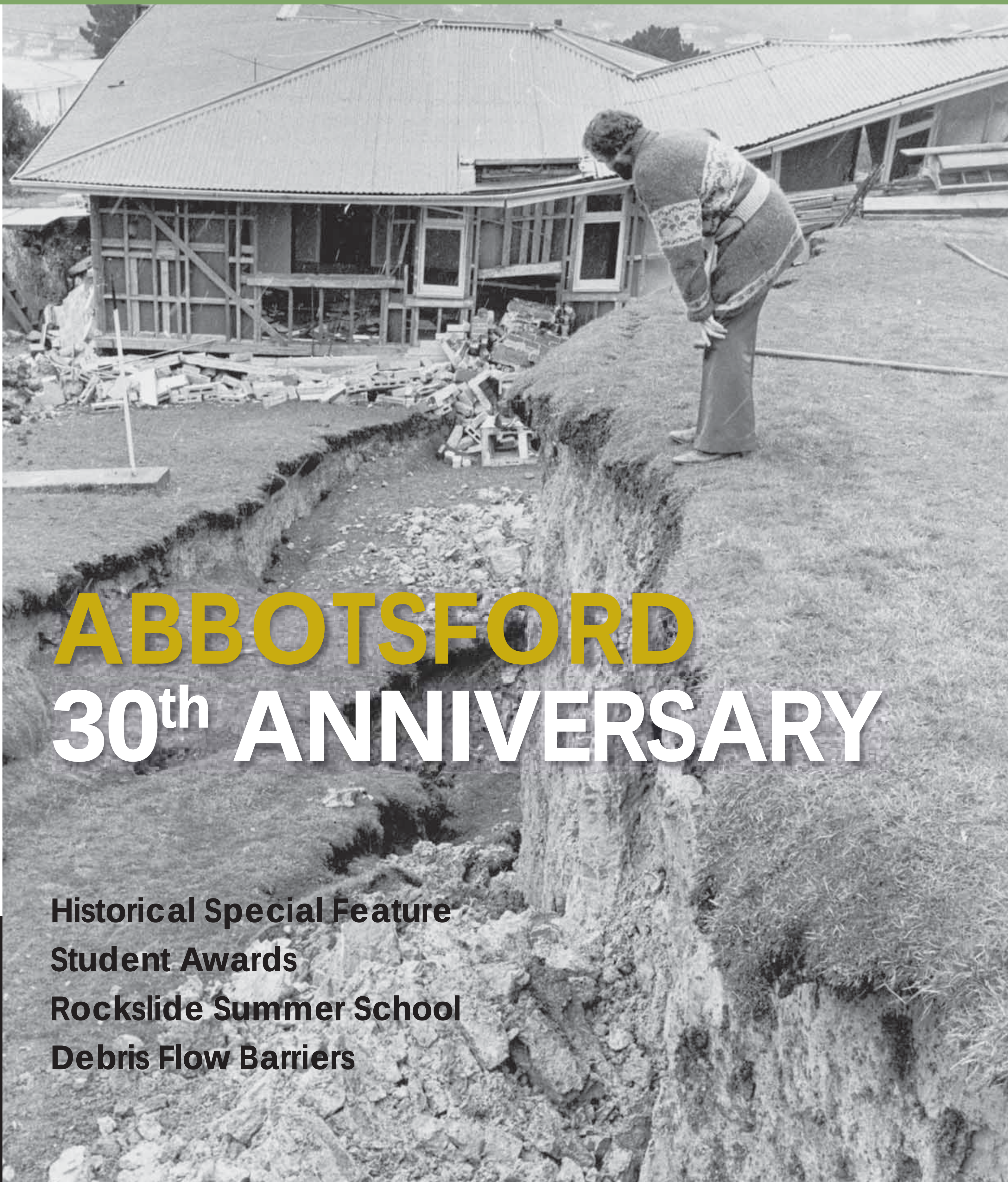

ABBOTSFORD 30th ANNIVERSARY

Historical Special Feature

Student Awards

Rockslide Summer School

Debris Flow Barriers

NZ GEOMECHANICS NEWS
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NEW ZEALAND GEOMECHANICS NEWS

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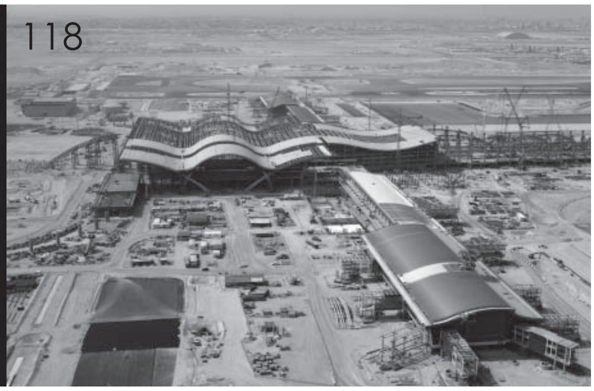

Cover image: Abbotsford , couresty of The Otago Daily Times

CHAIRMAN'S CORNER

Your Management Committee has been hard at work this year and I would like to take the opportunity to highlight some of the events, courses and seminars we have organized and to acknowledge some of the achievements of our Members.

Professor Michael Davies has begun his term as the ISSMGE Vice President for Australasia.

The 2008 NZ Geomechanics Award was presented to Misko Cubrinovski for his contributions to the paper "Psuedo-static analysis of piles subjected to lateral spreading" the other authors were Kenji Ishihara and Harry Poulos.

The YGP's have done an excellent job in promoting the Student Prize this year and we have had a good response from student members. The Student Awards for 2009 have just been completed as this goes to press and we congratulate all the participants and winners for both regions.

After some period of time finalisation of the 2008 Student Award has been determined, congratulations to Justin Wyatt and James Arthurs who shared the North Island Student Award for 2008. Jennifer Haskell was the recipient of the Southern Region Student Award for 2008 and best wishes for her studies in England.

The NZGS has provided input into the development of engineering knowledge by hosting (together with the University of Canterbury) a series of seminars by Colin Smith, Senior Lecturer, University of Sheffield UK, Visiting Erskine Fellow, University of Canterbury on "Advances in Geotechnical Limit Analysis and its Practical Application in Limit State Design".

A subcommittee of the NZGS completed the "NZ Geotechnical Seismic Design Guidelines" this year and a few members have provided input into earthworks aspects of the NZS4404 revision and Standards NZ revision of NZS3604:1999.

We also hosted Dr Chris Haberfield who presented his very interesting Nakheel Tower and EH Davis Memorial Lecture to our Auckland, Wellington and Christchurch branches.

In early November, Clyde Baker kindly agreed to present his Terzaghi Lecture entitled "Uncertain Geotechnical Truth and Cost Effective High Rise Foundation Design" to the NZGS and later in the month, a 1-day short course series was offered on the "Design and Analysis of Deep Excavations" with Professor Wong Kai Sin, Nanyang Technological University, Singapore. In December, Warwick Prebble has kindly offered to present a short course on field mapping.

Our plans for Next Year are similarly audacious. We have been talking to the local ICE representatives and the AGS about coordinating a series lectures by Tom O'Rourke the 2009 Rankine Lecture recipient. As you all know we will proudly be hosting the 11th Congress of the International Association for Engineering Geology and the Environment (IAEG), in Auckland in September 2010. Ann Williams and her team are working hard on preparing for more than 600 delegates, and I have every confidence that it will exceed all expectations.

The year's hard economic pressures have had an effect on many of our members. The Management Committee has agreed to keep our Membership Dues at the same rate at the 2008/2009 level. Here's hoping the economy improves next year.

Happy Holidays!

Philip Robins
Chairman, NZGS
Email: probins@golder.co.nz

Philip is a Senior Geotechnical Engineer at Golder Associates based in Nelson. Philip graduated more than 20 years ago from the University of Natal, Durban, with a BSc in Civil Engineering. After working in the field of civil/geotechnical engineering for a few years, Philip returned to study at the University of California, Davis leaving with a MSc in Geotechnical Engineering and a focus on earthquake engineering. Philip is a Member of the Institution of Professional Engineers New Zealand and a Chartered Professional Engineer (CPEng). He has been involved in the design and construction of major infrastructure projects in New Zealand, California, Hong Kong and Southern Africa. Philip's project experience ranges from planning and managing subsurface exploration programs including onshore/offshore drilling and sampling and in situ testing through site evaluation to the design of deep and shallow foundations, major earthworks, fill embankments and retaining structures. Philip is particularly proud of his contribution to the San Francisco-Oakland Bay Bridge East Span Seismic Safety Project and construction of Pier 400 Container Wharf at the Port of Los Angeles. When not working Philip spends most of his time chasing after his 7 year old daughter.

EDITORIAL

In this issue of Geomechanics News we look back 30 years to the Abbotsford Landslide - a seminal event for geotechnics in New Zealand - which the Minister of Civil Defence at the time described as New Zealand's biggest natural disaster in 25 years. This landslide influenced legislation and Councils approach to land development, and has become a case study for every class of graduating engineering geologists, and many engineers, since. Luckily, no one died but many homes were destroyed and talking with several of the surviving residents for this issue, it's obvious they were deeply affected.

The Historical Special Feature includes a comprehensive new paper on the Abbotsford Landslide by Graham Hancox, who's involvement extends back to New Zealand Geological Survey days and who, in a nice synchronicity, presented a talk on Abbotsford to the Geotechnical Society (or Geomechanics Society as it was back then) Wellington Branch in 1981 – the details of which we've reprinted in the Past Contributions section. Also included are some observations from Nick Rogers, a young geologist who arrived in Dunedin the day before the main slide, and Trevor Roberts - one of the members of the Commission of Inquiry into the disaster, who provides some very interesting insight into the internal workings of the Inquiry and the inevitable rise in emotions and finger pointing associated with such an event.

Obviously, New Zealand was a different place back in 1979; Robert Muldoon was PM, newspapers carried half page cigarette ads and a one-way airfare to Sydney was $2,000 in today's dollars, but it's interesting to consider whether, without the benefit of hindsight, we would recognise the instability and potential risks associated with Abbotsford more accurately than our colleagues did back then?

The other articles in this issue suggest that, in 2009, we at least have more tools in our toolbox to choose from when assessing ground conditions, and we have the bright young student minds needed to apply them - for example, there's news on a rock dilatometer now available in NZ to measure the strength characteristics of the soft weak rocks often present in challenging slope stability situations, a report on an international field course on large landslides, and reviews of three new texts which all include sections on slope stability.

The 2009 Student Awards recognise three interesting contributions looking to advance the understanding of ground motions for seismic hazard assessment; characterize foundation rocking for seismic foundation design; and classify collapse mechanisms in underground mine workings - all topics aimed at reducing risks and optimising engineering design in New Zealand.

As 2009 wraps up we can look forward to 2010. The 11th IAEG "Geologically Active" Congress, which is being sponsored by our Society, is less than a year away and, as reported on page 63, planning is well underway. The Congress focus will include the identification and evaluation of natural hazards, managing geological risk, state-of-the-art approaches to hazard mitigation from around the world and better bridging the gap between science and practise - all themes that have some resonance when we remember Abbotsford.

Finally, as Newsletter editors we'd be interested to hear what questions or issues you feel the geotechnical community in New Zealand should be discussing. We are always keen to receive from you, the members, letters to the editor, content suggestions, and technical articles. These can be submitted by email, letter, or via a soon-to-be-added field on the NZGS web-site.

Have a happy and safe Christmas.

Paul Salter & Kate Williams

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Co-editor: kwilliams@tonkin.co.nz

THE SECRETARY'S NEWS

As our membership has increased, somewhat dramatically in the last six months by 55 members, whom we welcome below, it becomes apparent that more is expected of the Society. More informed information, better access to events, up to date contact details, more relevant correspondence, and all in a timely manner, whilst generating less emails if possible! We suspect many members, especially those with multiple society memberships (e.g. SESOC, NZSEE as well as IPENZ), would appreciate fewer emails.

Well, you're in luck! Behind the scenes, work has been diligently undertaken over the last few months to up-date, out-style and super-size our website. As a mere observer (albeit keen contributor) of the process, I urge you to check out the new website and send us your feedback. At time of print – we are teetering on the edge of 'going live' – and should be by the time you read this. As we try to reduce emails – or at the very least attachments that can be downloaded on the web – please bear with us.

I'd like to remind you all that our Society has a hugely dedicated group of Branch Coordinators that work hard to organise presentations and events around the country throughout the year. Please remember to say thanks to these people who volunteer to find guest speakers, venues and sponsors – and perhaps lend a hand if the opportunity arises in your region.

And finally – congratulations to our recent Student Award winners, Thomas, Jawad and Matt, well done and best wishes for your studies and clearly bright futures.

Seasons greetings to you all for a happy holiday and summer – wherever you are!

New Members

Membership is now at 747 members and it is a pleasure to welcome the following new members since May 2009:

STUDENTS

MacKenzie; ML Melrose; AR Hope; TB Algie; KL Rait; MJ Arefi; DB Scott;

MEMBERS

R Roslee; P Malan; S Arumugiam; R MacFarlane; M Carter; P Forest; JSN McNeill; GR Mathur; AJ Holland; LM Paterson; CGC Hughes; FC Neeson; JMM Engel; ADJ Law; FA Wilson; AE Tonks; E Pearson; JM Keam; MD Collins; MJ Begbie; L Neville; JCG Seward; M Rahman; C McGhie; L McQuillan; AD Cabadonga; R Sonnenburg; B Hannah; BB Thompson; C Challenger; T Van Deelen; JLN Lavoie; JK Bennett; K Bristow; CS Yan, HA Abdul-Wahab; LM Wotherspoon; NM Amso; SM Roberts; NK Clendon; JD Bradshaw; B McDowall; G Gilmer; DP Mahoney; AL Sinclair; DB Scott; B Kafle; P Hardcastle

Please feel free to contact me for any assistance or with any queries you might have.

Amanda Blakey
Management Secretary
nzgeotechnicalsociety@xtra.co.nz

EDITORIAL TEAM

Amanda Blakey joined the NZGS in late 2008. Her role as the society's Management Secretary is varied and busy. Membership of NZGS now exceeds 740, and as well as providing administration support to the NZGS Committee and Chairman, she is the first point of contact for the NZGS's growing membership. International links with ISSMGE, ISRM and IAEG are also maintained by Amanda. During the course of the year Amanda's duties include responsibilities for the production of Geomechanics News, support for Branch events and presentations, organisation of NZGS short courses, assistance with awards, the production of guidelines and the NZGS Membership Information booklet.

Amanda Blakey
Management Secretary

Karryn Muschamp
Graphic design

EDITORIAL POLICY

NZ Geomechanics News is a biannual newsletter issued to members of the NZ Geotechnical Society Inc. It is designed to keep members in touch with matters of interest within the Geo-Professions both locally and internationally. The statements made or opinions expressed do not necessarily reflect the views of the New Zealand Geotechnical Society Inc. The editorial team is happy to receive submissions of any sort for future editions of *NZ Geomechanics News*. The following comments are offered to assist potential contributors. Technical contributions can include 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
- comments on papers published in *NZ Geomechanics News*
- descriptions of geotechnical projects of special interest

General articles for publication may include:

- letters to the NZ Geotechnical Society
- letters to the Editor
- articles and news of personalities
- news of current projects
- industry news

Submission of text material in Microsoft Word is encouraged, particularly via email to the Editor or on CD. We can receive and handle file types in most formats. Contact us if you have a query about format or content.

Diagrams and tables should be of a size and quality appropriate for direct reproduction. Photographs should be good contrast, black and white gloss prints or high resolution digital images. Diagrams and photos should be supplied with the article, but also saved seperately as 300 dpi JPGs. Articles need to be set up so that they can be reproduced in black and white, as colour is limited.

NZ Geomechanics News is a newsletter for Society members and articles and papers are not necessarily refereed. 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 submitted by members will be forwarded to the contributing member for a right of reply.

Persons interested in applying for membership of the Society are invited to complete the application form in the back of the newsletter. Members of the Society are required to affiliate to at least one International Society and the rates are included with the membership information details.

INTERNATIONAL SOCIETY REPORTS

International Society of Soil Mechanics and Geotechnical Engineering

Australiasia VP Report: November 2009

The 2009 meeting of the Council of the ISSMGE met in Alexandria, Egypt in early October immediately prior to the 17th International Conference on Soil Mechanics and Geotechnical Engineering. At this meeting the Council elected the President to serve for the period 2009 to 2013. In the months leading up to the Council meeting two very strong candidates had emerged as contenders for the post, Professor Jean-Louis Briaud from the USA and Professor Waldemar Hachich from Brazil, and it was Professor Briaud who attracted the greatest number of votes in the election. The appointments of the six regional Vice-Presidents who will serve on the Board of the ISSMGE for the next four years were confirmed at the meeting. The constitution permits the President to appoint three members to the Board and Professor Roger Frank (France), Dr Ikuo Towhata (Japan) and Professor Askar Zhussupbekov (Kazakhstan) together with the immediate Past President, Professor Pedro Pinto, complete the membership of the Board. The constitution also requires there to be a First Vice-President who can represent the President when necessary and I was delighted to be appointed to this post. The new Board came into effect at the end of the conference at which point I succeeded Professor John Carter as Vice-President for Australasia. As well as having responsibility on the ISSMGE Board for a number of issues for the ISSMGE as a whole, Professor Carter has been a strong advocate for the Australasia region and the AGS and NZGS are most grateful to him for the excellent contribution he has made as Regional Vice-President.

Above: Dr. Mamdouh Hamza Chairman, Organising Committee 17th ICSMGE Alexandria, Egypt during the closing session of the conference.

In one of his first acts as President, Professor Briaud has created three new board level committees. Two of these, the Technical Oversight Committee (TOC) and the Membership, Practitioners, and Academicians Committee (MPAC), will oversee the technical activities and membership issues of the Society, respectively. The third committee, the Innovation and Development Committee (IDC), will be the "think tank" of ISSMGE. Its task will be to develop ways to make ISSMGE progress in a manner that will "increase its usefulness to the members and provide excitement for the future of geotechnical engineering in ISSMGE". Each of these committees has a core membership of distinguished geotechnical engineers who will be inviting other members of the ISSMGE to join their committee.

Professor Briaud is eager to involve younger members of the profession in the activities of the ISSMGE and he has initiated a new group comprising of students and young members of ISSMGE which will meet with him on a regular basis (about 4 times a year by conference call and occasionally in person). This group is to be known as the ISSMGE Student and Young Members Presidential Group (SYMPG). Professor Briaud has asked each region to nominate three members for this group and, following consultation with the AGS and the NZGS, I am delighted to report that Owen Woodland and Brendan Scott from Australia together with Lucy Coe from New Zealand have been selected to serve on the SYMPG.

The President has proposed also the establishment of the ISSMGE Foundation. This is being 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 for example attending conferences, participating in webinars, purchasing geotechnical reference books and manuals. Professor Briaud and his wife have initiated the ISSMGE Foundation with a gift of US$10,000 and this has been followed by a number of corporate donations. More details of the ISSMGE Foundation and how to donate to it will be made available to ISSMGE members once these have been finalised.

In addition to the election of the President, the other major decision at the 2009 meeting of the Council of the ISSMGE was the selection of the location for the next quadrennial international conference (18th ICMSGE).

As with the election for the President, there were two contenders – from the Chinese and French member societies. However, the bid from China was ruled to be ineligible because the 16th ICSMGE was held in the same region (Osaka, Japan). The conference will, therefore, be held in Paris, France, from 1st to 5th September 2013.

The initiatives brought in by the new President are aimed at making the ISSMGE more "user friendly" leading to the involvement of more of the membership in its activities and, indeed, increasing the membership. This includes, specifically, encouraging the involvement of a greater number of young professionals in the activities of the ISSMGE. I am looking forward to working with members of the AGS and the NZGS over the next four years to deliver the President's vision for the ISSMGE.

Professor Michael C.R. Davies
ISSMGE VP Australasia

International Society for Rock Mechanics

Australasia VP Report: November 2009

1 BOARD ISSUES

Major activities for the Society for this year included:

- Professor Xia-Ting Feng from Wuhan, China, was elected as the ISRM President for the term 2011-2015
- The Society now has 6005 members which represents a 9% increase in membership over the previous year. The Society now has the highest number of members ever.
- The successful Sinorock 2 symposium held at the University of Hong Kong from May 19 to 22. The specific theme of this event was Rock Characterisation, Modelling and Engineering Design Methods. A total of 200 papers were accepted from the more than 500 abstracts submitted.
- The 2nd ISRM Lecture Tour was run in May. The tour lectured to over 1000 students, researchers and practitioners at 6 universities and research institutions in Beijing, Nanjing, Wuhan and Hong Kong, China. The lecturers were John Hudson and John Harrison from the UK, Tony Meyers from Australia and Resat Ulusay from Turkey.
- Progress negotiating permission from the copyright holders to upload the already digitised papers from 11 ISRM International Congresses and 31 International Symposia into an online database is well advanced. These papers will be made available on-line to members through a "virtual library".
- A more-or-less successful trial of a web based video conference was held between Board members using Cisco's WebEX system. Eventually it is hoped that video conferencing will make it easier and less costly for Board members to get together to discuss ISRM matters.

2 6TH MULLER AWARD

The award honours the memory of Professor Dr. Leopold Müller, the founder and first president of the Society. It is made once every four years at the ISRM Congress, in recognition of distinguished contributions to the profession of rock mechanics and rock engineering.

Candidates are nominated by the ISRM National Groups. Each National Group may propose one candidate, with no restrictions on who may be nominated. The deadline for presenting nominations is 16 April 2010.

The nominations will be voted by secret ballot at the next Council meeting which will be held in New Delhi in October 2010. The selected nominee will receive the award and deliver the Müller Lecture at the 12th ISRM International Congress in Beijing, in October 2011.

3 ROCHA MEDAL

An invitation has been extended to the whole Rock Mechanics community, for nomination for the Rocha Medal in 2011. Each nomination shall conform to the ISRM By-law No. 7, to be found at the ISRM Website

<www.isrm.net> and shall be received by the ISRM Secretary General not later than 31 December 2009.

4 INTERIM BOARD MEETING

There is a need to discuss the many activities that the Board are currently involved with. The President John Hudson has therefore decided to hold an interim Board meeting due to the long period between the May 2009 Board meeting in Hong Kong and the next meeting scheduled for October 2010 in New Delhi. The meeting is planned for February 2010 in Bogota, Columbia.

While in Columbia, and then repeated in Lima, Peru, each Board member will present lecturers to local members who don't often get the opportunity to participate in such events. These lectures will cover.

- Rock stress; its occurrence and variation.
- Joint statistics; what we have learned in the last 50 years.
- The strength of rock joints.
- The application of rock mechanics principles to rock engineering design.
- Application of limit state design principles for rock engineering projects.
- Intelligent rock mechanics.
- Design of large slopes in rock.
- Design of underground excavations for civil and mining projects.
- Failures and Repair of the Chingaza Water Supply Tunnels in Bogota, Colombia.

5 MEMBER COMMUNICATION

- There was robust discussion during the May Council meeting as to whether the largest individual expense for the Society, the one off annual cost to post the hard copy NewsJournal, represents good value for Members.
- As part of our modernisation program, the Board trying to make it easy for members to obtain news about the ISRM. A trial RSS feed of the ISRM News is now available and once fully up and running it should be a useful tool for keeping up to date with what's happening with the society.
- One matter currently being looked into by the Board and Secretariat is the use of web based technology to bring information to members. Webcasting will likely be used initially as it's the simplest form of communication to set up. Keynote presentations in rock mechanics will be videoed and then distributed on demand to members around the world via the internet using streaming media technology. It is hoped that eventually webinars will be trialed to enable real-time two-way communications between a presenter and interested participants. Communication would involve VoIP audio technology via the use of headphones and speakers. Webinars can involve many useful features not available in a webcast.

6 EUROCK 2009

The Croatian Geotechnical Society held the successful ISRM Regional Symposium EUROCK 2009 at the Hotel Croatia in Cavtat (Epidaurus) on 29-31 October. The historical location was a highlight, being close to the old Croatian town of Dubrovnik on the Adriatic Sea.

The first reference to Epidaurus dates back to 47 BC. The legend has it that the city was founded by Greeks and destroyed by barbaric tribes of Slavs and Avars. In 614 A.D., the refugees from the city founded the town of Ragusa in the better protected place nearby, which is now known as Dubrovnik. The area of Dubrovnik was chosen as venue for the symposium principally because it exemplifies the story of rock used as building material.

The theme of the symposium was *Rock Engineering in Difficult Ground Conditions* focussing particularly on the challenges of engineering within karst environments. In Croatia and the surrounding regions, karst is widely present. Several of the terms used to describe karst features such as polje, ponor and dolina, are actually Croatian words.

Keynote presentations were:

- Georgios Anagnostou: Some rock mechanics aspects of subaqueous tunnels.
- Giovanni Barla: Innovative tunnelling construction method to cope with squeezing at Saint Martin La Porte Access Adit (Lyon-Torino Base Tunnel).
- Nick Barton: Metro construction at the most unfavourable depth caused a major metro station collapse in Brazil due to a unique sub-surface structure.
- Heinz Brandl: Rock engineering for structures in unstable slopes.
- Evert Hoek and Paul Marinos: Tunnelling in overstressed rock.
- John A. Hudson: Stresses in rock masses: a review of key points.
- José Muralha, Luís Lamas and Nuno Grossmann: Site characterization and rock testing for the evaluation of design parameters.
- Ivan Vrkljan: Rock engineering in Croatia: history, current status and special problems.

A total of 127 papers were presented on the following topics:

- 15 papers were presented on geological and hydro-geological properties of karst regions;
- 42 papers were presented on rock properties, testing methods and site characterization;
- 17 papers were presented on design methods and analyses;
- 12 papers were presented on geological monitoring

and back analysis;
- 18 papers were presented on excavation and support;
- 8 papers were presented on geotechnical engineering in karst regions; and
- 15 case histories were presented.

7 MINING NEWS

The mining industry is a major employer of rock mechanics practitioners. In later 2008 and early 2009, the international mining sector contracted significantly as fears that the 'Great Recession' could track the 'Great Depression'. Tens of thousands of mining company employees across all sectors lost their jobs.

Fortunately, world GDP appeared to have bottomed in the June quarter, about 15 months after its peak, and there were signs of stabilisation and recovery. Mining companies reported increased earnings from coal, iron ore and gold with many companies reporting record export earnings for the 2008/09 financial year. Reports now indicate that mining has one of the brightest employment futures of any industry with the possibility that there could be major skills shortages within the next two years.

There is an increasing number of companies gravitating away from the traditional mining areas towards developing countries like Mongolia, Ghana, Guinea and Libya. For example, Mongolia could be set to become a considerable supplier of copper, gold, coal and uranium to China. These countries are therefore growing rapidly in importance, if from a low base, as the world's new resource frontiers.

Any increasing in production must not be at the expense of safety. Recently there have been several disturbing incidents. For example, eighteen people died on Australian mine sites during the last financial year, almost double the annual average. This number included five BHP mining workers killed in unrelated incidents in Western Australia. The company's iron ore president, Ian Ashby, said. "Our safety performance has been abysmal and we are working feverishly on it." "What we all need to do is pay a little more attention, or a lot more attention, to the critical controls that mitigate these risks."

One way to reduce accidents is to remove personnel from high risk areas. Rio Tinto is taking a step towards this goal by launching a 15-month trial of the world's first fully autonomous commercial mine. If successful, the company hopes that it will eventually transform its vast Pilbara iron ore operations. Rio chief executive Tom Albanese believes technicians and engineers monitoring the whole process from a remote operations centre in Perth, 1500km away, will eventually control the "mine of the future". He says "This is all about Rio Tinto having safe, efficient production, meeting ever increasing tonnes that our customers require". Mr. Albanese said autonomous equipment would allow for more accurate and efficient mining and reduce wear and tear on trucks and other vehicles. He guaranteed that there would be no job losses but said it would help deal with the problems of sending people long distances to remote mine sites.

8 UPCOMING ISRM SPONSORED EVENTS

- **31 May-1 June 2010 3rd International Workshop** on Rock Mechanics and Geo-Engineering in Volcanic Environments, an ISRM-Sponsored Specialised Meeting. Organised by Spanish Rock Mechanics Society (SEMR) and the Government of the Canary Islands. http://www.citiesonvolcanoes6.com
- **15-18 June 2010 Lausanne, Switzerland**, ROCK'2010 - Rock Mechanics in Civil and Environmental Engineering. Organised by Swiss Society for Soil and Rock Mechanics (the ISRM NG SWITZERLAND) and Swiss Federal Institute of Technology Lausanne (EPFL). http://lmr.epfl.ch
- **25-27 August 2010 5th International Symposium** on In-situ Rock Stress, an ISRM-Specialised Conference. Organised by Inst. of Crustal Dynamics - China Earthquake Administration. http://www.rockstress2010.org
- **23-27 October 2010 New Delhi, India.** 6th Asian Rock Mechanics Symposium on Advances in Rock Engineering (ARMS6). Organised by Indian National Group of the ISRM and Central Board of Irrigation and Power http://www.cbip.org

Tony Meyers
ISRM VP Australasia

International Association for Engineering Geology and the Environment Australasia VP Report: November 2009

Above: Attendees at 7th Asian Regional Conference of IAEG

Venue, regional conference and field trip

The annual IAEG Executive and Council meetings were held in Chengdu, in China on 7 and 8 September 2009. The meetings were followed by the International Symposium on Geological Engineering Problems in Major Construction Projects (in combination with the 7th Asian Regional Conference of IAEG). Chengdu is close to the epicentre of the Magnitude 8, Wenchuan Earthquake which occurred in May 2008. There were keynote lectures and special sessions on the earthquake at the Symposium and a three day field trip to the area. Ann Williams, Fred Baynes, Mark Eggers and I went on the field trip and saw some of the terrible damage caused by the earthquake and the associated landslides.

Financial position

Overall the IAEG is still in a sound financial position and the Council has agreed to spend some of the available funds on helping people from low and middle income countries attend the Auckland Congress.

IAEG Bulletin

The IAEG Bulletin is doing well. It now publishes more than 60 papers a year from many different countries on a wide range of topics. There was a proposal to drop the abstract in French in the hard copy and to allow submission of abstracts in two other languages of the authors choice in the online version. The proposal was not passed by Council after a somewhat confusing debate. Most people agreed that the move to online publishing (with abstracts in up to two other languages of the author's choice) is a good idea. Debate continues within the executive and bulletin language issues may be discussed and voted on again at the Executive and Council meetings in Auckland next year.

Subscribers to the Bulletin now receive passwords, which give access to electronic versions of all back copies, which is a significant benefit to members. Unfortunately the electronic access system has had some teething problems, which had not been resolved at the time of writing this report.

IAEG Commissions and Website

Several new IAEG commissions have been established but progress on most of the existing and new commissions has been slow. Some commissions have been disbanded and remaining commissions have been encouraged to become more active and report their activities on the IAEG website. The website has improved in the past 12 months but more useful content (including commission reports) and links are required.

Membership database

The value of a world-wide data base of individual members was discussed and it appears likely that the IAEG will attempt to establish the data base soon.

IAEG Congress and Richard Wolters Prize

Ann Williams has prepared a separate report on the 11th IAEG Congress which will be held in Auckland from 5 to 10 September 2010. 872 abstracts were received and registration for the congress is now open.

The Richard Wolters Prize has been awarded biannually since 1986 to a younger member (under 40) of the profession. In the past it has been primarily awarded on the basis of theses or publications. At the annual meeting in Madrid last year, the IAEG Council decided to try out a new procedure at the Auckland Congress. The new procedure involves a paper and a short presentation at a one day conference (limited to 16 people) the day before the congress. National Groups have been asked to nominate candidates for the prize by 5 March 2010. The IAEG hopes that National Groups will support their nominees financially. A copy of the procedure to award the Richard Wolters Prize in 2010 is attached to this report.

The Next IAEG VP for Australasia and IAEG Liaison

My term as IAEG Vice President for Australasia (VP) ends at the end of 2010. IAEG will be calling for nominations from National Groups for the next VP soon. The NZGS and the AGS (Australian Geomechanics Society) have agreed the sequence of nominations for VP roles for the International Societies and the next Australasian IAEG VP should be selected by NZGS. Consequently the AGS will not be making a nomination for the next IAEG VP for Australasia and is looking forward to hearing who NZGS will nominate.

The AGS will select an Australian to assist/liaise (IAEG Liaison) with new VP.

Alan Moon
IAEG VP Australasia

PROCEDURE TO AWARD THE RICHARD WOLTERS PRIZE 2010

The **Richard Wolters Prize** has been awarded biannually since 1986 to commemorate the life and work of Dr Richard Wolters, for his significant achievements in the advancement of engineering geology and for his important role in the development of the IAEG.

The **Richard Wolters Prize** specifically recognises meritorious scientific achievement by a younger member of the engineering geology profession (less than forty years old on 1st January of the year that the prize is awarded) and is awarded to honour Dr. Wolters' many contributions to international understanding and co-operation. (Please refer to the IAEG website for further information on the Richard Wolters Prize. http://www.iaeg.info/AboutIAEG/Awardsandprizes/RichardWoltersprize/tabid/74/Default.asp).

At the Council meeting in Madrid, September 2008, the Council decided to try out in Auckland 2010 for the first time a new procedure leading to the award of the Richard Wolters Prize. Together with the Organising Committee of IAEG 2010 the following procedure has been worked out:

1. Selection by each National Group of IAEG of their most meritorious young engineering geologist under the age of forty years at 1 January 2010. The selection procedure may vary for each national group, but the selection is based mainly on the presentation of one scientific publication (MSc or PhD thesis, or paper on a case history submitted to a conference or published in a (refereed) journal) A curriculum vitae must also be provided and will be considered. Nomination by the National groups of their candidate for the Richard Wolters Prize in an official letter with documentation on the paper to be presented to the IAEG Secretary General by **Friday 5 March 2010**
2. The maximum number of nominations is sixteen. If the number of nominations exceeds sixteen, then the Executive Committee will select the sixteen best candidates by **Monday 5 April 2010** on the basis of the documentation provided
3. The National Groups will be informed by **Friday 9 April 2010** whether their nominee is invited to present her/his paper in Auckland
4. Oral presentations by up to sixteen young National Group representatives of their paper in the "Young Engineering Geologists Pre-Congress meeting" on **Sunday 5 September 2010**. Presentations must be given in English, however the quality of the spoken English will have no bearing on the decisions of the panel
5. Selection of the one to three best presentations by a panel appointed by the IAEG Executive Committee
6. Award of the Richard Wolters Prize and the runner up prizes will be made during the official opening session of the Congress on **Monday 6 September 2010**. The winner will give his/her presentation at this session
7. It is expected that the IAEG National Groups support the travel costs, accommodation and fee for Congress participation of their candidates financially. National Groups from low income countries and National Groups with very small numbers of members can apply for (partial) financial support by the IAEG solidarity fund in a letter to the IAEG President
8. Registration fees for Richard Wolters Prize candidates will be at student rates irrespective of whether the candidate is a student or not. Special cheap accommodation arrangements can be made, if required

May we ask your cooperation as National Group to send to the Secretariat General the Nomination by your National Group of your candidate for the Richard Wolters Prize in an official letter with documentation on the paper by **Friday 5 March 2010**.

Furthermore we draw your attention to the point No. 7, stating that it is expected that the IAEG National Groups support the travel costs, accommodation and fee for Congress participation of their candidates financially.

Yours very sincerely,
Secretary General of the IAEG

NZGS BRANCH ACTIVITIES

Auckland Branch Activity Report

Another busy year in Auckland is drawing to a close and it is time to reflect on the activities from the year. A transfer in the Auckland Organising Committee has taken place with Yan Chan handing the reins over to Lucy Coe, Ross Kendrick and Pierre Malan. On behalf of the present Auckland Branch organising committee, NZGS and all the members we offer our thanks to Yan for his dedication and service to the Auckland branch. His handover has been excellent and, along with Rodney and Steve has offered ongoing support to the new committee.

The year has been busy with events almost every month and a hectic finish to the year. By the time Geomechanics News goes to print we will have had six events including two overseas lecturers in five weeks! We are currently working on ways of getting digital versions of lectures to other branches if the lecturer cannot travel for some reason. Watch for more on this in the upcoming year.

MAY

Rolly Orense, Auckland University

The 26 May event gave Rolly Orense of the University of Auckland an opportunity to share his experience in Asia and present a lecture on "Lessons from Recent Geotechnical Disasters in Asia". This talk focused on three large-scale disasters in Japan and Philippines, with plenty of impressive photographs for the audience.

JUNE

Colby Barrett, Soil Nail Launcher Inc.

For the 23 June event, Colby Barrett came out from the USA to share his knowledge on Slope Reinforcement - thanks to the support of Hiway Stabilizers Environmental. Colby presented an engaging talk, covering a vast amount of technical and practical information on soil nails and mechanically stabilised earth.

AUGUST

Andrew Campbell, AECOM

Ground Support Design and Dewatering at Bendigo Mining

Andrew Campbell of AECOM presented a lecture on Ground Support Design and Dewatering at Bendigo Mining. This talk focused on design of ground support for capital development, life of mine infrastructure and stoping. Consideration is also given to risk management associated with development of new workings in the near vicinity of flooded historic workings and overall dewatering strategy.

Nick Wharmby & Tony Sage, Brian Perry Civil

SITE VISITS to New Lynn Rail Trench Project

Nick and Tony opened up the New Lynn Rail Trench site for NZGS invasion! This was a unique opportunity to visit the site and to get an overview of the project, see the diaphragm walling operation, observe the excavation and exposed diaphragm wall. The trench is being formed using a pair of diaphragm walls that are propped by a thick floor slab and a top prop where required. In addition, conventional bored piles and H piles have been used to support the wall, bridge structures and resist long-term uplift forces. A very interesting design and challenging construction process.

Chris Haberfield, Golder Associates

Foundations in Rock and the Nakheel Tower

Dr. Haberfield shared some fascinating insights into pile design in rock and challenged a number of the underlying assumptions currently used around the 'factor of safety' concept. Following a piling design philosophy discussion, a case history of design for the 'supertall'; Nakheel Tower in Dubai was presented. The tower presented some fascinating challenges founding in materials that a number of the audience would initially describe as marginal. A very interesting and challenging lecture.

SEPTEMBER

Sjoerd Van Ballegooy, Tonkin & Taylor Ltd

Kawakawa Bay Landslip Remedial Measures

This fascinating lecture presented a case history of ongoing slope stability remedial works at Kawakawa Bay, Auckland. The design concepts, assessment of landslide and design remedial measures were well documented in a very interesting presentation.

The year has a busy finish. See below for indicative dates ;

8th December	NZGS Christmas Gathering at Old Government House	
10th December	Warwick Prebble	TBC
11th December	Onsite field mapping 1-day course	TBC

We are already planning another great schedule for next year.

Ross Kendrick
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Ross is an Engineering Geologist with AECOM NZ Ltd based in Auckland. He grew up in Auckland and graduated from Auckland University with an MSc. He states, yes, I am an Aucklander through and through, and as some people may refer to us, a JAFA! Ross initially worked in London, then moved to Scotland, returning to New Zealand to work in Auckland.

Lucy Coe
Auckland Branch Coordinator
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Lucy Coe is a Geotechnical Engineer with Beca Infrastructure Ltd. After graduating from the University of Canterbury, Lucy moved up to Auckland and as been here ever since. Lucy has undertaken geotechnical investigations, design and construction monitoring on major infrastructure projects such as CMJ, Upper Harbour Bridge, Northern Busway and New Lynn Rail Trench.

Pierre Malan
Auckland Branch Coordinator
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Pierre is a Geotechnical Engineer with Tonkin & Taylor Auckland. Pierre graduated from the University of Canterbury with a M.Eng and has subsequently worked around Auckland and throughout the United Kingdom and Ireland. He has worked on major infrastructure work, design and build contracts as well as a range of small to medium projects.

Bay of Plenty/Waikato Branch Activity Report

On 17th September, James Arthurs a Ph.D. candidate at the University of Auckland presented his findings to date from studies on the origin of sensitive volcanic soils in New Zealand and in particular the western Bay of Plenty. This study focused on silty, air-fall deposits related to the eruption of the Kidnappers ignimbrite. Field and laboratory test results were analysed and conclusions related to the usefulness of particular test methods and the reasons for the sensitivity of these deposits were made. Comparisons were also drawn between New Zealand sensitive soils and those from Canada and Norway.

Up coming Events

28 October 2009		Tauranga Harbour Link MSE Wall Design and Construction - Yolanda Thorp (URS), Gavin Midgley and Matt Fairweather (Fletcher Construction). Presentation followed by short site visit
19 November 2009	TBC	Pike River Mine Tunnel, by Evan Giles of URS.
Later in 2009 early 2010	TBC	Presentation/Tour of advanced wastewater treatment systems factory – Devan Group.

Kori Lentfer
Bay of Plenty Branch Coordinator
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Kori has taken over the role of Waikato/Bay of Plenty Branch Co-ordinator in June 2009.

Kori is a consulting Engineering Geologist who works for Coffey Geotechnics. He graduated in 1998 with a BSc(Tech) degree 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 U.K., Europe and the Middle East. On return to the home-land he joined Foundation Engineering in Orewa, which was acquired by Coffey Geotechnics in 2007. In April 2008 Kori transferred to the Tauranga office for the lifestyle and diverse geotechnical challenges.

Wellington Branch Activity Report

David Stewart
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David is a Senior Geotechnical Engineer/Engineering Geologist with Opus International Consultants in Wellington. David completed an MSc in Engineering Geology at Canterbury University and then worked in site investigations in the UK, returning to NZ to work on the Cromwell Gorge Landslides project. He then worked as an engineering geologist for GNS in Dunedin, followed by 2 years at Macraes Gold Mine. After a stint in Auckland picking up a BE, he joined Duffill Watts & Tse in Wellington in 2001 and has been at Opus since mid 2004.

We have continued monthly meetings throughout the year, meeting at the Opus Architecture Meeting Space in the Majestic Centre. Attendances have generally been good with around 25 at most meetings.

Since the last report we've had the following meetings:

21 May, Ian McPherson of Aurecon (formerly Connell Wagner) gave his talk – '*Screw Piles at Daly Street*'. This talk looked at the use of screw piles for building foundations in a Hutt Valley site underlain by soft compressible soils. Potential liquefiable soils and the affect of the foundations on the Hutt Aquifer were some of the issues that were addressed. Piezometers with water level data loggers inserted allowed monitoring of tidal fluctuations in groundwater levels.

9 June, Colin Smith from the University of Sheffield (currently Erskine Fellow at Canterbury University) presented a 1 day Seminar '*Advances in Geotechnical Limit Analysis*' outlining developments in computational limit analysis methods including the new numerical analysis procedure, Discontinuity Layout Optimization (DLO) co-developed by the presenter.

24 June, three speakers from Opus spoke on aspects of the recent *Transmission Gully* preliminary geotechnical assessment. David Stewart and Beverley Curley outlined the mapping and site investigations (including 88 boreholes) carried out during 2007-2008 along the 28 km route. P Brabhaharan (Brabha) then covered some of the geotechnical concept design and related route security aspects of this proposed new highway route immediately north of Wellington.

22 July, Graham Hancox, of GNS Science Lower Hutt, presented his talk '*Abbotsford Landslide in Retrospect*' based on a paper he has recently published (in "Landslides 2008"). Graham was involved in carrying out part of the original geotechnical investigative work at the time of the landslide, which occurred almost exactly 30 years prior to the date of his talk.

26 August, Chris Haberfield of Golder Associates Melbourne, presented his talk covering aspects of his *E H Davis Memorial Lecture* as well as design and construction aspects of the foundations for the *Nakheel (Tall) Tower* in Dubai. This was a very full and thought provoking lecture in which Chris challenged many of the practices and assumptions used in design of pile foundations in rock. If you missed his lecture, his powerpoint presentation is currently available on the NZ Geotechnical Society Website.

30 September, Nick Peters of Tonkin and Taylor Ltd, Wellington gave a talk – *'Kapiti Views Subdivision - Subdivision Design & Construction in Geologically Hazardous Terrain'*. Nick covered the engineering geological assessment, risk assessment, design and construction aspects of this steep and variable site near Waikanae.

27 October GNS, NIWA and Victoria University staff presented an update of the *"It's Our Fault"* research project, and in particular, an update on the Likelihood of earthquake events on the main active faults in the Wellington Region. Whilst this was not a Geotechnical Society event – a considerable number of members attended at short notice (which no doubt contributed to the depleted turnout to our monthly meeting the next day).

28 October, Evan Giles from URS presented his talk on the *Pike River Coal Mine* project; to a disappointingly small turnout. This was a really interesting talk on a fascinating project. The salutary lessons learnt regarding the predicted versus actual conditions experienced in the construction of the access tunnel and ventilation shaft should hopefully stick in the memories of those (particularly the engineering geologists) who attended, for a long time.

Programme for the rest of 2009 is

Thursday, 26 November	**Prof K S Wong** Nanyang Technological University, Singapore.	The Nicoll Highway Collapse in Singapore
Friday, 27 November	**Prof K S Wong**	Design & analysis of deep excavations (1 day course)
10, 11 December	**Warwick Prebble** Auckland University	Engineering Geological Mapping Course (in Auckland)

In 2010, possible topics include: in mid 2010 - a talk by GNS in on the geotechnical implications of recent research by GNS and others on the seismicity of the Wellington region – the next stage of the "Its our Fault" project.

Many thanks to those who have helped facilitate the Wellington branch activities during the year; whether it is putting out chairs, arranging or sponsoring refreshments, or tidying up after meetings and especially to the speakers. Particular acknowledgement to my colleagues at Opus for their help in many and varied ways through the year.

Next year I hope to be able to spread the venues and workload around various organisations. If anyone is interested in being part of a team to coordinate the Wellington branch activities please get in touch with me. Also, please pass on any ideas for talks or site visits.

Canterbury Branch Activity Report

This year we've had a wonderful suite of presentations in the Canterbury Branch and at the time of writing these notes (early November) I look at my fixtures list and see at least three more events to come before the year is out.

I take great personal satisfaction organising the events, but the success of the Branch is the result of a wide group of people playing different roles in the process. I would like to acknowledge the role of the presenters themselves for taking the time to talk to us with passion about their chosen subject, and to you – the Branch members – for attending the events and contributing to the discussions that I find add a very rewarding dimension. There is usually a great ferment of discussion amongst the attendees prior to the meetings that is also an important facet. I would also like to thank the many others that have contributed to this year's success including members of the Society's Management Committee.

This year we've covered a suite of topics including rockslides, tunnelling, soil reinforcement, sinkholes and foundations, and at the time of reading this you may well have attended and gained from presentations on the design of retaining walls and deep excavations.

If your attendance at meetings has led you to reconsider your own practices or stirred you to look into a new topic then we can look back over the year knowing we have comfortably addressed two aims of the Society, viz:

1. To advance the education and application of soil mechanics, rock mechanics and engineering geology among engineers and scientists, and
2. To advance the practice and application of these disciplines in engineering.

If not, then I am all ears! The effectiveness of the Branch and the Society as a whole is purely a function of our collective efforts. As I often mention at meetings if you have suggestions for presentations or different ways of running the show then please let me know.

I have already signed presenters for February, March and April next year so we are in good shape to kick off 2010. I would like to see more site visits in the calendar so please contact me with suggestions.

That's all for now. I wish you all a well-earned break over Christmas and New Year. Hopefully, I'll see you in February at the presentation on the Deep Fault Drilling Project – Alpine Fault.

Nick Harwood
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Nick is a consulting Geotechnical Engineer who works for Coffey Geotechnics. He graduated in 1990 with a BEng(Hons) degree in Engineering Geology & Geotechnics, followed by a MSc in Soil Mechanics & Engineering Seismology from Imperial College in 1994. Nick started out as a graduate working for British Waterways before moving onto Brown & Root (London) and Buro Happold (Bath) before finally escaping to New Zealand in 2002. He loves living and working in New Zealand, a place that combines sublime scenery and diverse assignments.

Nelson Branch Activity Report

Very quiet year in Nelson with only a handful of presentations. Hope everyone had a half decent year negotiating the credit crunch and best of luck for next year. Merry Christmas and a happy new year to all.

Tim Coote
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Tim Coote is an Engineering Geologist for Tonkin & Taylor in Nelson.

Otago Branch Activity Report

Otago had a great talk in June from Philip Kirk, formerly of the Geotechnical Engineering Office, Hong Kong Government. The talk was entitled, "Unforeseen Ground Conditions: A case study from Tung Chung, Hong Kong" and got tongues wagging about;

i) the need for cooperation of competing consultancies on a common job
ii) warnings on using generic guideline methods for logging geological materials without formal training
iii) inability to abandon our conceptual models in the face of adversity.

Our two talks so far this year have been largely based in geology. Otago is hoping to have another talk before the holiday season so if anyone is keen to present a talk with more of a geotechnical design swing pipe up. Otherwise we've got potential for a talk on alluvial fans in Otago.

Markus Hanz
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Markus is a consulting Geotechnical Engineer with Opus International Consultants in Dunedin. Markus came to New Zealand from Germany in December 2006 and has extensive experience as a consulting engineer in a wide range of geotechnical projects. Markus is the Geotechnical Workgroup Leader in Dunedin and has been kept busy designing retaining walls and pile foundations, assessing rock slope instability and holding the reins for the Opus Dunedin Geotechnical Team since October 2007.

Shane Greene
Otago Branch Coordinator
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Shane is an Engineering Geologist with Opus International Consultants in Dunedin. Shane came to New Zealand from Canada in January 2006 and has been working with the Opus Geotechnical Team since that time. Shane has specialisations in Hydrogeology and Contaminated Land Assessment however since coming to New Zealand has turned his hand to everything from foundations to slope stability investigations.

RICHARD WOLTERS PRIZE NOMINATIONS

The NZGS is calling for nominations for a Richard Wolters Prize finalist to represent New Zealand.

The Prize is awarded for meritorious scientific achievement by a younger member of the engineering geological profession and will be contested on 5 September 2010, as part of the IAEG2010 Congress. More details of the prize and requirements can be found on the Young Professionals page of www.iaeg2010.com.

Nominees:

> Must be members of the IAEG less than 40 years old at 1 January 2010
> Will be judged in part on curricula vitae; and

On one scientific publication

> MSc or PhD thesis
> Paper/ case history submitted to a conference or referred journal.

If you would like to apply or to make a nomination, please submit the following details to NZGS Management Secretary by 5 February 2010 by email to **nzgeotechnicalsociety@xtra.co.nz**:

1. *Name and contact details of the nominee*
2. *Name of institution/ organisation*
3. *Date of birth*
4. *Copy of the scientific publication or title and access arrangements for viewing thesis*
5. *CV*

In the event of a number of nominations being received, the Management Committee may request that selected nominees give a presentation as the final basis for selection.

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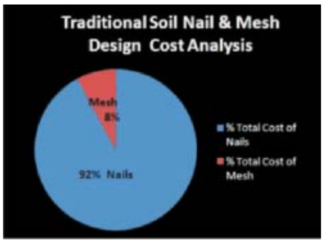

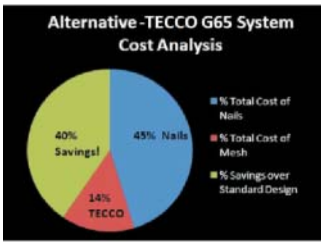

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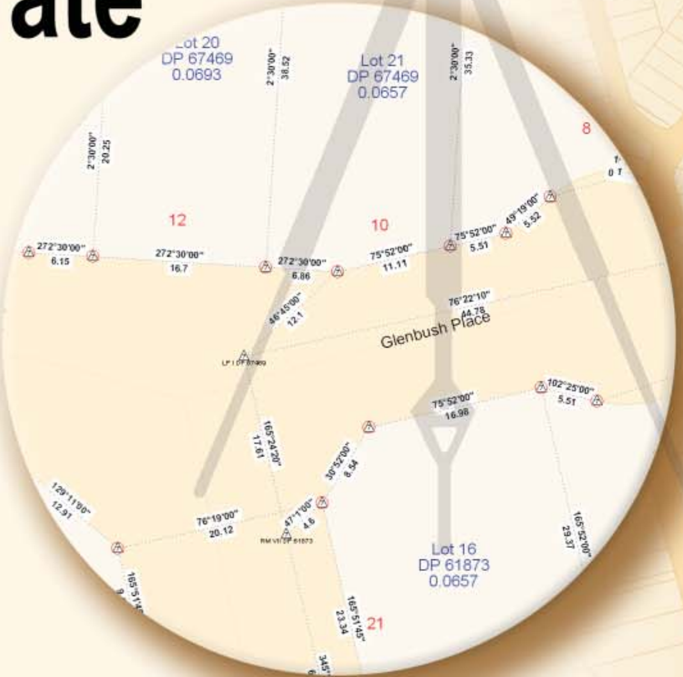

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1979

2009

ABBOTSFORD
30 years on…

The 1979 Abbotsford Landslide in Dunedin, New Zealand: A Retrospective Case History Thirty Years after the Disaster

– Graham T Hancox[1]

Abstract

The Abbotsford Landslide of 8 August 1979 occurred in an urban area of Dunedin, New Zealand, causing considerable damage to houses and urban infrastructure. Rapid failure occurred after weeks of preliminary movements, resulting in the formation of a 5 million m^3 block slide. It caused the loss of 69 houses, with an overall cost of about 15 million (NZ) dollars (2009). Prior to the failure, geological mapping, survey monitoring, and drilling was undertaken to establish the nature of the landslide and rates of ground movement. Insufficient geotechnical and piezometric data were collected to allow the landslide to be analysed, remedial actions taken, or full evacuation of the area before the final rapid movement occurred. Seventeen people on the slide mass at the time were fortunate to survive without injury or loss of life. Investigations carried out after the failure to determine the cause of the landslide and long-term stability of the area included cored drill holes with inclinometers and piezometers, test pits, Calweld shafts, soils testing (grading, bulk density, consolidation, permeability, Atterberg limits, shear strength, and mineralogy), and limit equilibrium stability analyses. A Commission of Inquiry subsequently found that unfavourable geology, weak clay layers in a 7 degree dip slope, was the main underlying cause of the landslide. A former sand quarry at the foot of the slope and a leaking DCC water pipeline above the slide area were found to be man-made factors that contributed to the failure. Stability analysis showed that the excavation of c. 300,000 m^3 of sand from the slide toe reduced stability of the overall slope by about 1%, while a 1 m rise in the water table across the area would have reduced stability by 2–3%. Because the quarry closed 10 years before the landslide occurred, it is concluded that a long-term rise in groundwater levels, due to increased rainfall over the previous decade and leakage from the DCC pipeline controlled the timing of the failure, and in this sense is considered to have triggered the landslide.

Keywords: Abbotsford Landslide, block slide, groundwater, causal factors, landslide risk, Dunedin, New Zealand.

1. Introduction

Rapid movement of the Abbotsford Landslide in the Abbotsford suburb of southwest Dunedin, New Zealand (Figure 1) occurred on 8 August 1979, following several weeks of slow movements. No one was injured or killed by the landslide, but because it occurred in an urban area

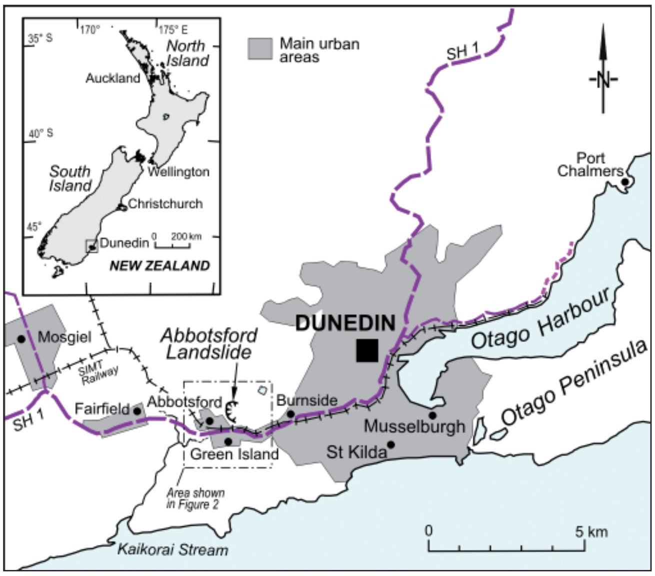


Figure 1: Location of the Abbotsford Landslide area.

and caused damage to many houses and infrastructure in the area, it is New Zealand's most notorious landslide in the last 50 years. The total cost from destruction of houses, damage to infrastructure, and relief was about 15 million (NZ) dollars (2009). It was the subject of a Government Commission of Inquiry (CoI), whose primary tasks were to determine the cause of the disaster, whether actions by man contributed to its cause, and the adequacy of measures taken before, during and after the event. Thirty years after the disaster, this landslide still offers valuable lessons about the causes of such events, both natural and man-made, and possible measures that are needed to avoid or mitigate the effects of similar disasters in the future.

The author was involved in investigations after the landslide to determine its cause, and was co-author of several reports prepared for the Commission of Inquiry into the disaster by New Zealand Geological Survey, Department of Scientific and Industrial Research (DSIR), which is now part of GNS Science. These reports and others were considered by the CoI, which sat for 58 days between 28 September 1979 and 27 June 1980, and presented its findings in a report on 19 November 1980 (CoI 1980).

Because the CoI report and other geotechnical reports on the Abbotsford Landslide are not widely available, in 2008 the author published two papers dealing with different aspects of the landslide. One paper (Hancox 2008a) presents a historical summary of some of the geotechnical aspects of the Abbotsford Landslide, including discussion about its development, the underlying preconditions that contributed to the failure, and likely triggering factors. The other paper (Hancox 2008b) describes the geotechnical investigations carried out before and after of the landslide, and discusses and evaluates the risk assessment and response before the final rapid movement occurred.

In this retrospective case history, prepared thirty years after the landslide disaster, the issues covered by both recent (2008) papers are discussed more fully. It includes a summary of the landslide's development and investigations carried out to determine its nature and cause, and the hazard and risk it presented, and discusses retrospectively the effectiveness of the investigations in assessing and managing the risk of catastrophic failure as the landslide developed. There is also more comprehensive discussion of the cause(s) of the landslide, and description of the methods, parameters, and assumptions used in the stability analyses to determine the relative contributions of possible causal factors.

The present paper is based largely on the DSIR reports (Bishop et al. 1979, Hancox et al. 1980, Salt et al. 1980), and also draws on the Commission of Inquiry report (CoI 1980) and other relevant but largely unpublished information. The paper describes the geology and topography of the Abbotsford area where the landslide occurred, the history of ground movements, and the nature and features of the Abbotsford Landslide[1] as it developed. Conditions that are thought to have contributed to the cause of the landslide are outlined, and factors that may have triggered the final movements are discussed. In addition, the landslide is compared to some similar landslides in New Zealand, and its relevance to landslide hazard and risk assessment and response is discussed.

2. Location and geological setting of the landslide

The Abbotsford Landslide is located in Abbotsford, a residential suburb of southwest Dunedin (Figure 1). The terrain is hilly with gentle to moderate relief. The landslide lies on the eastern side of a gently sloping spur on the northern side of Kaikorai valley, and is bounded to the east by Miller Creek. From the floor of Kaikorai valley the spur rises steeply (20–25°) for about 100 m (elevation.), but flattens off at the top of the slope near Abbots Hill Road. The spur that failed had earlier been modified at its southern end by a sand quarry (Harrison's Pit), and further north by the prehistoric Sun Club Slide (Figure 2).

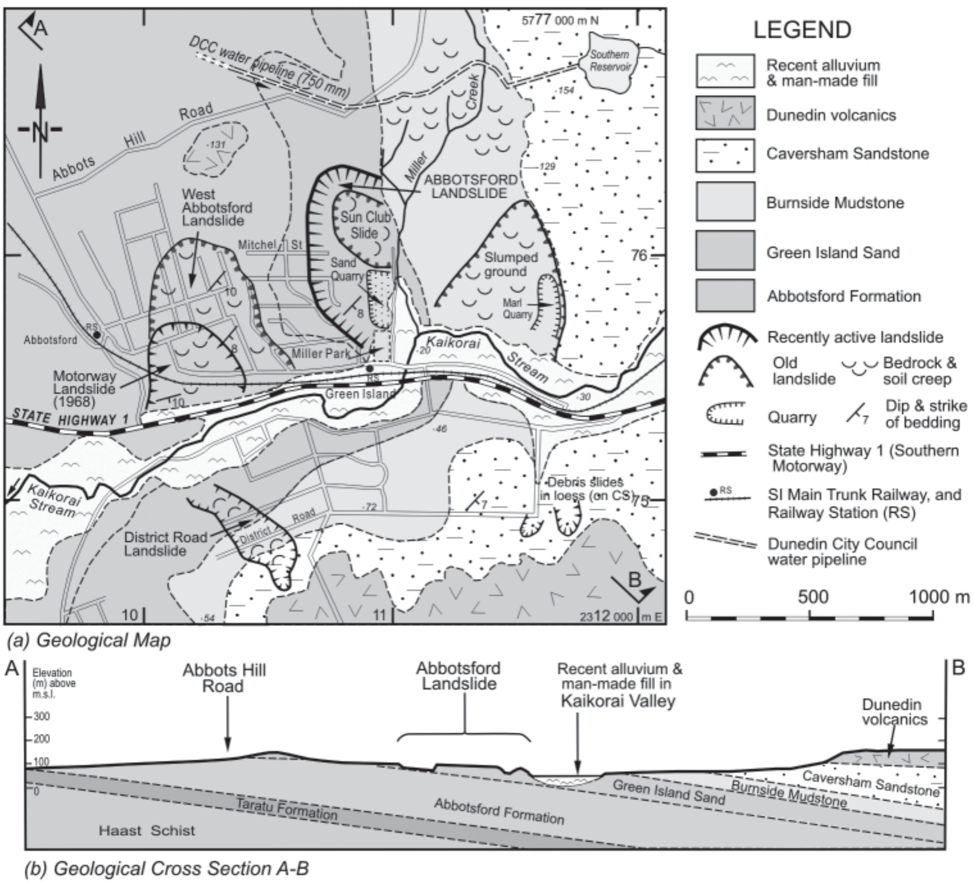


Figure 2: (a) Geological map of the Abbotsford area of southwest Dunedin. It shows the locations of the Abbotsford Landslide and several other landslides in the area. Thick deposits of loess and bouldery colluvium overlie gently south east-dipping Tertiary strata across most of the area (not shown on the map).
(b) Cross section A–B shows the topography and geological structure in relation to the Abbotsford Landslide. The slope on which the landslide formed is underlain by Tertiary strata which dip downslope 7–10° into the incised valley of Miller Creek, and is a type of (oblique) dip slope.

[1]*The Abbotsford Landslide was referred to in many reports as the East Abbotsford Landslide, to distinguish it from the West Abbotsford Landslide (Fig. 2).*

2.1 Geology of the landslide area

The Abbotsford area is underlain by a sequence of weak, poorly consolidated sedimentary rocks of late Cretaceous to middle Tertiary (Miocene) age (Benson 1968, Hancox et al. 1980, McKellar 1990, Glassey et al. 2003). The rocks have a uniform regional dip of 7–10° to the southeast. At Abbotsford the sequence is about 600 m thick overlying Jurassic age schist basement rock (Figure 2). The Tertiary rocks underlying the landslide include the c. 250 m thick Abbotsford Formation – a weak, relatively dense, green-grey mudstone, with sand lenses and thin montmorillonitic (smectite) clay layers. This is overlain by up to c. 100 m of Green Island Sand – a weak, non-cohesive, pale yellow brown, clayey to silty sand.

Figure 3: Aerial photo of the Abbotsford landslide area before the final movement on 8 August 1979. It shows the developing compression ridges (*R*) in the old sand quarry, tension cracks (*tc*) crossing farmland, and part of the Sun Club Slide. In the toe area, the Patterson house (*H*), which was later destroyed by the landslide, may have been affected by initial creep slide movement as early as 1972, or possibly 1968. Breakage of water mains caused by the developing ground cracks (C) became apparent in 1978 *(Photo by courtesy of Aeropix, Dunedin)*.

Although not shown in Figure 2, most of the Tertiary rocks are mantled by Quaternary age surficial deposits of dense, clayey bouldery colluvium up to 10 m thick, which is overlain by up to 7 m of stiff, clayey loess. These deposits result from solifluction and aeolian processes during the Last Glaciation, which ended about 14,000 years ago.

2.2 Other landslides in the Abbotsford area

Deep-seated slides and flows are common in Tertiary rocks in the Dunedin area, particularly in the Abbotsford Formation and Burnside Mudstone (Benson 1940, 1946; Hancox et al. 1980; McKellar 1990). Prior to 1979 several large landslides and areas of soil creep had been identified in the Abbotsford area (Figure 2). The West Abbotsford Landslide is a large prehistoric (~9,000 years old) earth flow in Abbotsford Formation mudstone. The toe of this slide was reactivated in 1968 by a cut for the State Highway 1 motorway, causing considerable damage to houses. The Sun Club Slide is a prehistoric landslide (at least 10,000 years B.P.) in Green Island Sand next to the sand pit, on the western slopes above Miller Creek (Hancox et al. 1980). These landslides clearly demonstrated the tendency for slope instability in the Abbotsford area.

2.3 Harrison's Pit Sand Quarry

Between 1934 and 1969 Green Island Sand was excavated from Harrison's Pit sand quarry at the toe of the southern half of the slope that failed in 1979 (Figures 2 and 3). Approximately 330,000 m^3 of sand was excavated from the base of the slope, most of which was removed between 1964 and 1969 for construction of a nearby motorway. Much of the sand was used to stabilise (buttress) the toe of the West Abbotsford Landslide, which was reactivated in 1968 (Figure 2). The sand quarry excavation was about 90 m wide, with four batters constructed with an overall slope of about 25°, similar to the original slope (Figure 3). The quarry was closed in 1969, ten years before the final catastrophic movement in August 1979.

Figure 4: Aerial photo looking south on 5 August 1979. It clearly shows the developing tension cracks which eventually defined the graben at the head of the landslide. Investigation drilling (*D*) and test pit excavations (*TP*) are in progress at the end of Christie St. *(Photo by courtesy of Aeropix, Dunedin)*.

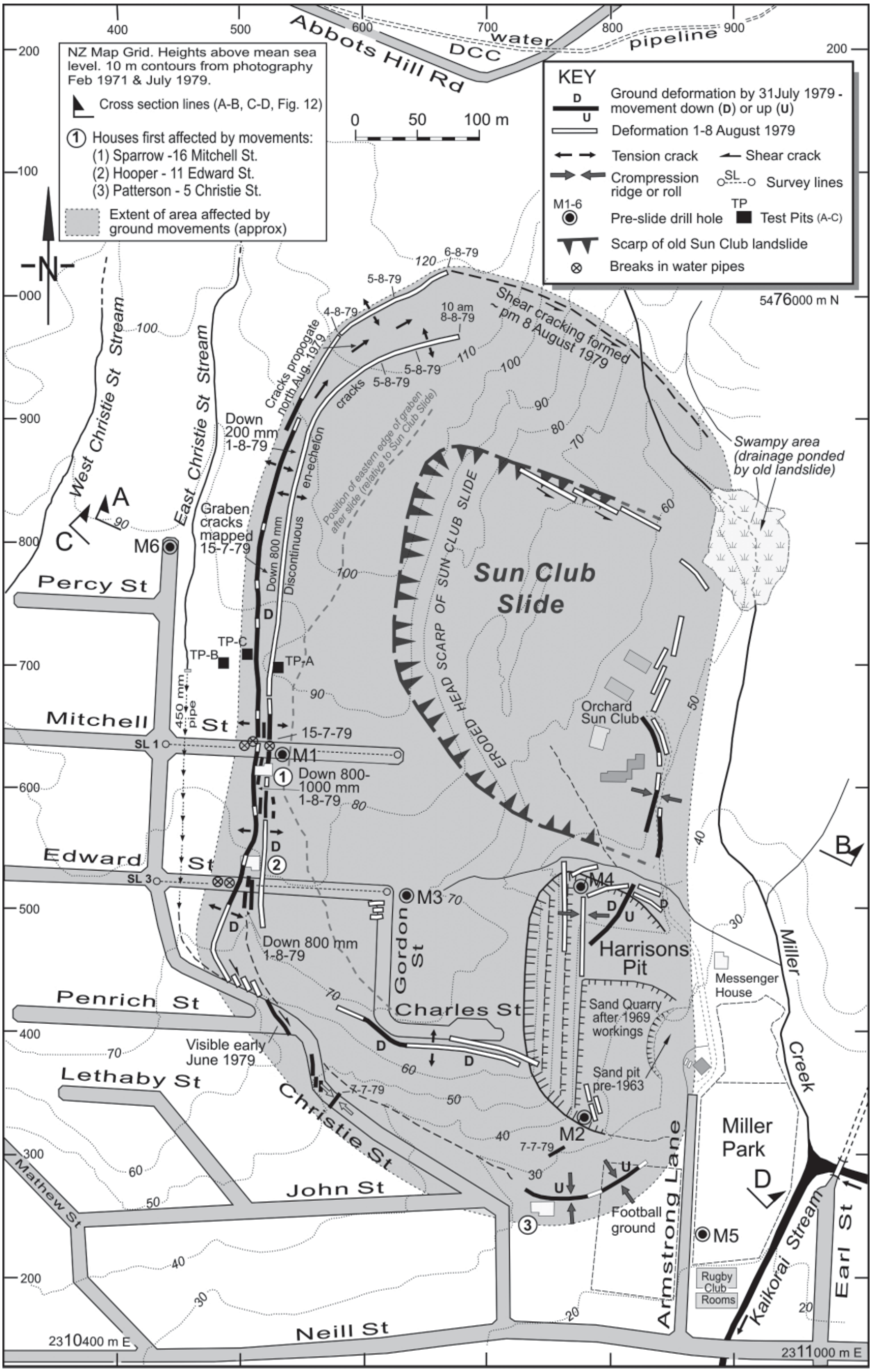


Figure 5: Map of the Abbotsford Landslide area showing the pre-slide topography, ground cracking and deformation features, houses damaged by the initial slide movements (1, 2, 3), and investigation drill holes, test pits, and survey lines (*after Hancox et al. 1980*).

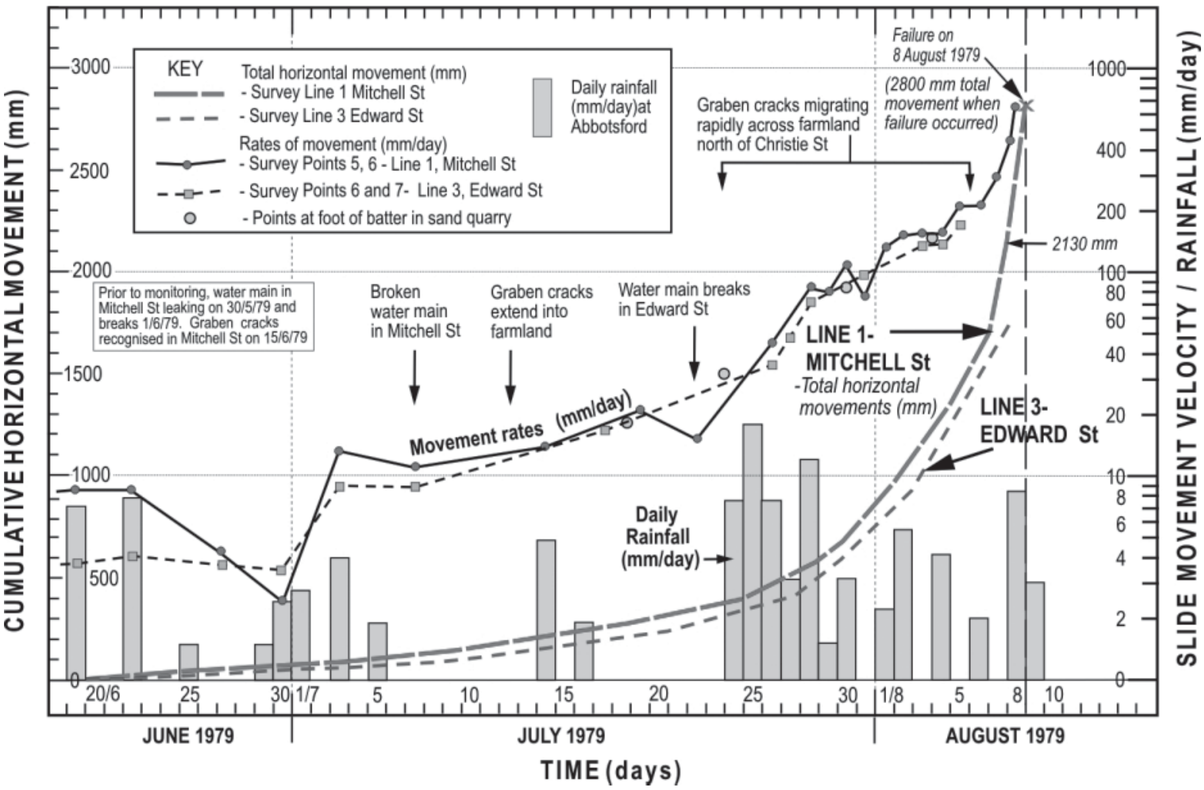


Figure 6: Graphs showing movements of the Abbotsford Landslide from 18 June to 8 August 1979 (after Hancox et al. 1980). Ground movement rates of survey points and lines are shown in relation to crack development, damage to water mains, and rainfall. The plots show both cumulative displacements (mm) and rates of movement (mm/day) across the widening graben cracks in Mitchell Street and Edwards Street. Movements of survey points at the slide toe are also shown. Response to the risk posed by the developing large landslide was planned on 5 August when movement accelerated from 150-200 mm/day. Plans were made to evacuate the landslide hazard zone by 12 August (a rate of 450mm/day predicted by then), but movements accelerated to 670 mm/day by 7 August. The increased rate of movement was surveyed, but the data were not plotted up and evaluated before the final movement on 8 August (*Survey data from Garden and Partners, and rainfall data provided by M Telfer*).

Drill Hole Number	Date Drilled	Depth (m)	Groundwater depth (m)	Inclinometer Data		
				Installed	**Failed**	**Failure position and time**
M1	22/6/79 – 29/6/79	15.9	9.3	31/7/79 [4th]*	1/8/79	Depth 17.6 m after 10 hrs* (0.9 m below GI Sand / Abbotsford Formation contact).
M2	30/6/79 – 9/7/79	21.2	0.8	10/7/79 [1st]*	12/7/79	Depth 12.8 m after 2 days* (0.4 m below GI Sand / Abbotsford Formation contact).
M3	10/7/79 – 26/7/79	36.8	14.2	27/7/79 [3rd]*	27/7/79	Depth 29.5 m after 17 hrs* (0.7 m above GI Sand / Abbotsford Formation contact).
M4	21/7/79 – 26/7/79	18	2.2	26/7/79 [2nd]*	27/7/79	Depth 13 m after 26 hrs* (0.85 m below GI Sand / Abbotsford Formation contact).
M5	c. 2/7/79 – 2/8/79	29	5.3	NA	-	-
M6	4/8/79 – 8/8/79	17.5	2.3	NA	-	-

Notes: Drill hole locations are shown on Figure 5. Groundwater levels are average water depths in drill holes left overnight. Piezometer tubes installed in drillholes M2, 3, 4, and 6 became blocked or sheared off by ground movements (no reliable readings). * Note: the time to inclinometer failure after installation becomes progressively shorter, showing that the landslide was accelerating (as shown in Figure 6).

Table 1: Details of pre-slide drill holes, inclinometers and groundwater levels.

3. Development and investigation of the landslide

3.1 Early landslide movements

The Abbotsford Landslide did not occur without warning. Minor cracking damage to the Patterson's house 60 m south of the sand pit (Figure 3) was reported as early as 1968–72, which coincided with removal of a large volume of sand from the pit. That house may have been damaged by initial creep movements of the landslide (CoI 1980), but there was not general agreement on this point. Damage to the house between 1968 and 1972 was thought by the authorities to be unrelated to the sand pit excavations or any major land movement. However, that house was one of the first to be seriously affected by early movement of the landslide in September 1978, when ground movements in the wider area first became obvious. Between September 1978 and June 1979, several water main breakages and damage to houses in Christie and Mitchell streets were caused by growth of ground cracks, which became more apparent as they spread northwards across adjacent farmland (Figure 3). These cracks eventually defined the head scarp and graben of a developing large landslide (Figures 4 and 5).

3.2 Pre-slide geotechnical investigations

Geotechnical investigations to determine the nature of the developing Abbotsford Landslide and depth of movement began in June 1979 and continued until the final movement on 8 August. The pre-slide investigations included:

(a) Establishment and monitoring of survey marks and two survey lines.
(b) Recording of damage to houses and essential services (water mains and storm water pipes, overhead wires).
(c) Mapping of ground deformation features (tension cracks, shear cracks, compression ridges, and areas of ground subsidence and collapse).
(d) Drilling of 6 drill holes (N-size, triconed and cored), with tube inclinometers installed in 4 of these holes.
(e) Several shallow (4-6 m) test pits and push-tube samples.

Details of the pre-slide drill holes, inclinometers, and recorded water levels are summarised in Table 1. Figure 5 shows locations of damage to houses, breaks in water mains, ground deformation features, survey lines, drill holes and test pits before the final movement on 8 August 1979. Plots of slide movement rates and widening of cracks determined from ground surveys are shown in Figure 6.

3.3 Pre-slide monitoring of ground movements

Although there was evidence of slope creep in the Abbotsford Landslide area between 1958 and 1978, the first unequivocal proof of ground movements associated with the 8 August failure were confirmed on 30 May 1979 when a broken water main joint was discovered in Mitchell Street (CoI 1980; Hancox et al. 1980). Fresh ground cracks and movements in several adjacent houses were also found at the same time, so a full-scale investigation was launched on 1 June to determine the extent and cause of this damage. This began with installation of ground survey marks around the affected area.

Over the next 11 days the established survey marks showed movement rates of 3–23 mm/day. Survey monitoring lines were then set out across the widening area of ground cracks in Mitchell Street and Edward Street. The ground surface deformation features suggested that deep-seated landslide movement was occurring. To determine the extent and depth of the movements, drilling investigations began in Mitchell Street on 22 June 1979 (drill hole M1). Over the next six weeks, 6 drill holes and several test pits provided information on the thickness of the landslide, the depth of the slide plane, soil properties, groundwater levels, and near-surface geology (Figure 5). Tube inclinometers were installed in four drill holes (M1–M4), but these were all sheared off by ground movements 1–2 days after they were inserted, about a week before the final movement. Notably, the length of time between installation and failure of the inclinometers became progressively shorter, indicating that the landslide was accelerating (Table 1).

3.4 Results of drilling investigations and monitoring

The mapping of ground deformation features, damage to houses and services, drillholes and inclinometers showed that the landslide was large and deep-seated. The failure zone was located near the top of the Abbotsford Formation between ~15–36 m below the ground surface (Table 1). Repeat surveys of monitoring marks and lines during June and July 1979 showed increased ground movements, with rates of 10–20 mm/day from June to mid July. The movement rate increased to about 100 mm/day at the end of July after 55 mm rain over 7 days, clearly showing that the ground movements were responding to rainfall (Figure 6).

During June and July 1979 the ground cracks in Mitchell Street began to define a graben. The movement accelerated after 26 July, when the graben cracks propagated quickly northwards across farmland behind the Sun Club Slide (Figures 4 and 5). Ground deformation was also recorded at the bottom of the slope in Harrison's Pit, and at the toe of the Sun Club Slide, where 30 m long cracks and turf compression rolls developed. By 4 August the graben cracks had extended more than 100 m north of the Sun Club Slide.

On the afternoon of 8 August shear cracks were found extending from the northern graben crack down to Miller Creek, defining the full extent of the overall area affected by ground movements, and clearly outlining the considerable size of the slowly developing large landslide (Figure 5).

3.5 Assessment of landslide risk and response

In late June 1979 a geotechnical committee of local engineers and geologists was formed to investigate the landslide. Throughout July the focus of the committee was mainly on defining the depth and mechanism of the landslide, measuring groundwater levels, and monitoring rates of ground movement. Because of the size of the affected area and accelerating rate of movements, remedial measures to stabilise the area were judged to be impractical (CoI 1980). Groundwater levels in the affected area were found to be relatively high (Table 1), so deep drainage was considered to be a possible solution (especially in the permeable Green Island Sand), but this was never initiated, mainly because of time constraints rather than cost.

Towards the end of July there was growing concern among members of the geotechnical committee about the increasing rates of ground movement. Therefore, after the last inclinometer failed at the beginning of August the emphasis shifted towards using movement rates to assess the risk posed by the landslide if a sudden, rapid failure and break up of the slope occurred. On 5 August, when ground movements had accelerated to 150–200 mm/day (Figure 6), the committee predicted that movements of 450 mm/day were likely by 12 August (CoI 1980). Because of this possibility, and increasing concern and uncertainty about when and how slope collapse might occur, the Mayor of Green Island Borough (GIB) declared a state of local civil defence emergency at 8 a.m. on Monday 6 August 1979. Under the wide provisions of a state of civil emergency, all residents in the hazard zone defined by ground movements (Figure 5) were advised by the GIB Council of the increasing landslide risk, and that they should make arrangements to vacate their homes (CoI 1980).

At midday on 6 August a progressive evacuation plan was adopted by the civil defence authorities. This plan required evacuation of the houses most at risk first (those in Christie, Mitchell, Edward, and Charles streets), and continuing until all of the houses at risk were vacated by 12 August. Completion of the evacuation was planned to coincide with a movement rate of 450 mm/day and 4 m total displacement predicated by 12 August, beyond which the essential water and power services to houses on the sliding mass could not be maintained.

The evacuation proceeded slowly on 6 and 7 August. Despite concern about ongoing slide movements, it was not appreciated over those days that the slide was in fact moving faster than had been predicted. Survey data compiled several days later showed a movement rate of around 430 mm/day on 6 August, which increased to 670 mm/day on 7 August (Figure 6). Although these precise rates of ground movement were not known at the time (prior to the final movement), anecdotal reports of steady movement of pegs and widening of cracks in Mitchell and Edward streets suggested that the state of the landslide was worsening sooner than expected (CoI 1980). However, because there was still uncertainty about how rapidly the slide would develop, some residents remained reluctant to leave their homes. By the evening 7 August most of the landslide-affected area had been evacuated, but six houses were still occupied when the final rapid movement occurred late the following day.

3.6 Final landslide movement

The final movement of the Abbotsford Landslide began at

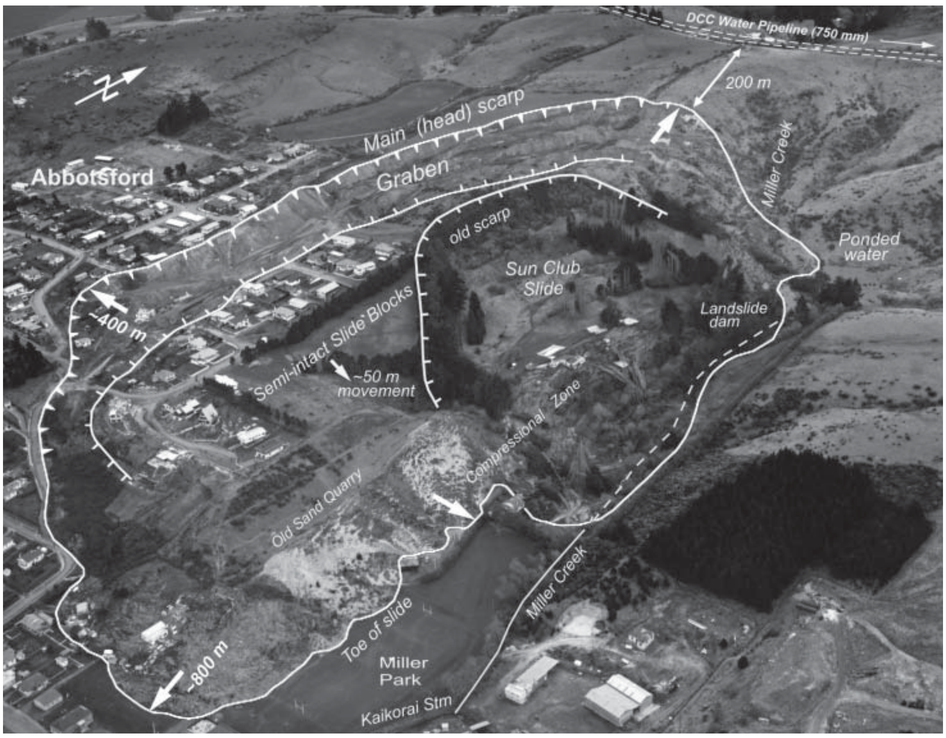


Figure 7: Aerial photo of the Abbotsford Landslide on 9 August, showing its main geomorphic features, which include: the head scarp, graben, houses on the sliding block, and the compression zone, landslide dam and ponded water at the slide toe. Other features shown that may have contributed to the failure include the old sand quarry at the toe and leaking DCC water pipeline 200 m to the north. The old Sun Club Slide on the west side of Miller Creek shows the inherent instability of the area (*GNS Science Photo CT947*).

Figure 8: Photo of the landslide disaster scene on the morning of 9 August showing the chaotic ground subsidence and some of the houses destroyed in the graben when it opened. The tilted and damaged house (H) moved about 50 m southeast and down about 12 m by the collapse (*photo by Don Bird, EQC*).

Figure 9: Aerial view of the Abbotsford Landslide looking south and showing the water ponded (p) in Miller Creek by slide debris, the graben (g) with its extensive concentric ridges and deep rents, earthworks (ew) to regrade the head scarp (hs) and protect nearby houses, the former sand pit (sp), Sun Club Slide (sc), and largely undamaged houses (h) on the slide blocks (*GNS Science Photo CT948*).

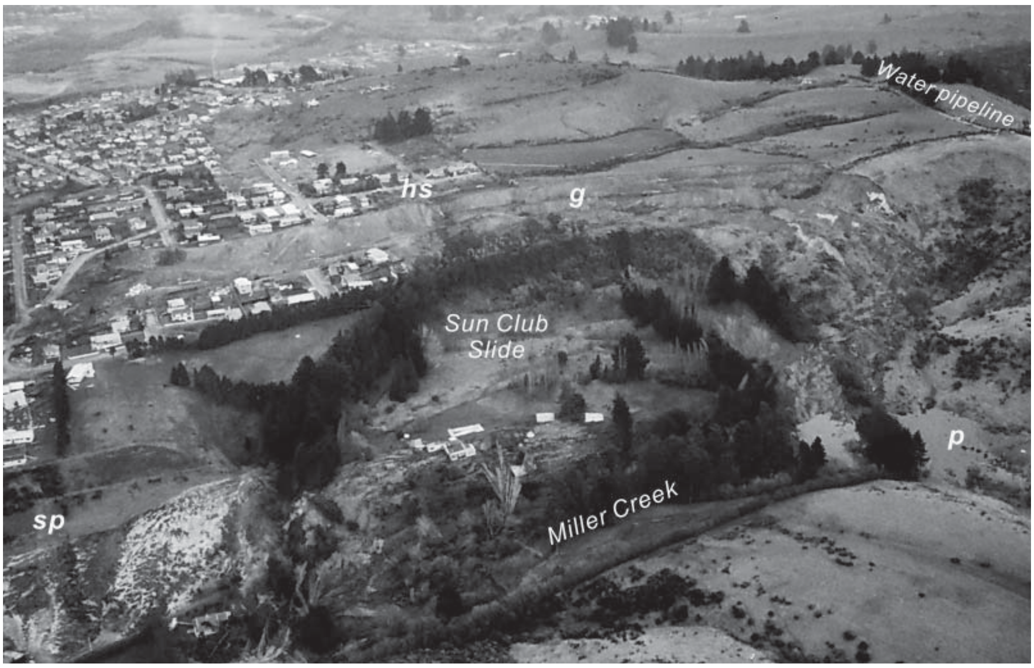


Figure 10: Another aerial view of the landslide showing the graben (g), head scarp (hs), sand pit batters (sp), the Sun Club Slide, and water ponded (p) in Miller Creek by landslide debris (*photo by G Hancox*).

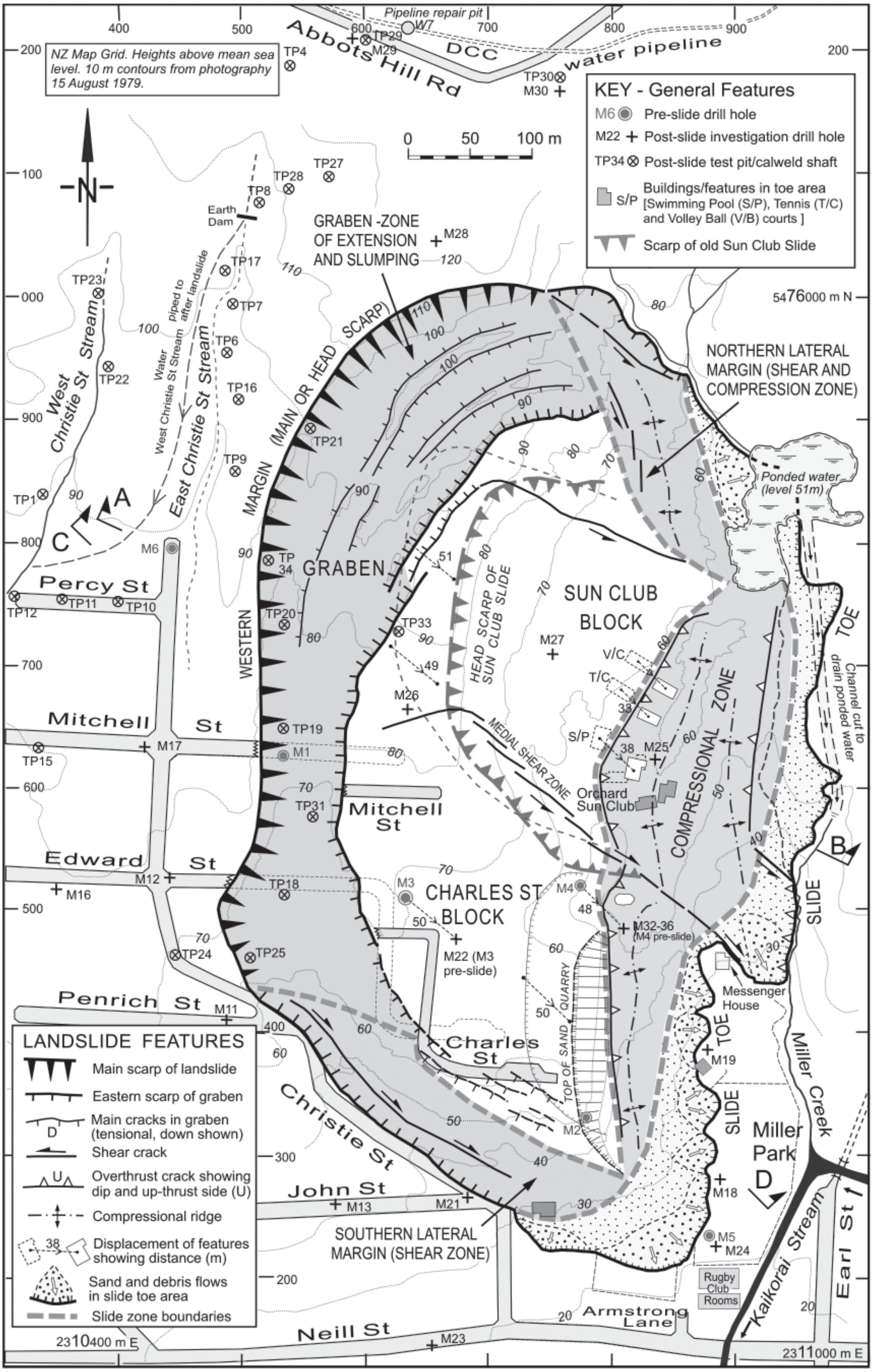


Figure 11: Map of the Abbotsford Landslide showing the post-slide topography, geomorphic features and zones of the landslide, and investigation drill holes and test pits. The amounts and directions of displacements of features on both slide blocks are also shown (after Hancox et al. 1980). The cross sections (A-B, C-D) are shown in Figure 12.

9.07 p.m. on 8 August 1979. Seventeen people were carried down the slope while still in their homes, and marooned on the slide mass when access was cut off by a gaping chasm, into which houses had tumbled and broken up. All of the stranded people were rescued by 11 p.m. Because of concerns about further slope collapses, areas to the west and south of the landslide were evacuated by 1 a.m. on 9 August. A total of 640 people were displaced from their homes as a result of the landslide (CoI 1980).

The final rapid movement of the landslide lasted up to 50 minutes. A large block of land containing 28 houses slid southeast towards Miller Creek. The sliding mass left behind a large chasm (graben) up to 30 m deep, into which 15 houses collapsed and were destroyed (see Figures 7 to 10).

The main landslide mass moved 50 m downslope towards Miller Creek over ~30 minutes, forming a graben 70–150 m wide and 15–30 m deep at the slide head. Much of the sliding block remained intact. Miller Creek was temporarily blocked by landslide debris, which included remnants of the relict Sun Club Slide (Figures 9 and 10). A trench was quickly dug to drain the ponded water and prevent a dam-break flood. Because the movement was relatively slow, and people had been evacuated from areas of greatest damage (in the graben and slide toe), no lives were lost and no one was injured by the slope failure. The landslide resulted in 69 houses being either destroyed or rendered inaccessible by the graben (Figures 8 and 9). Many houses on the sliding block survived the landslide

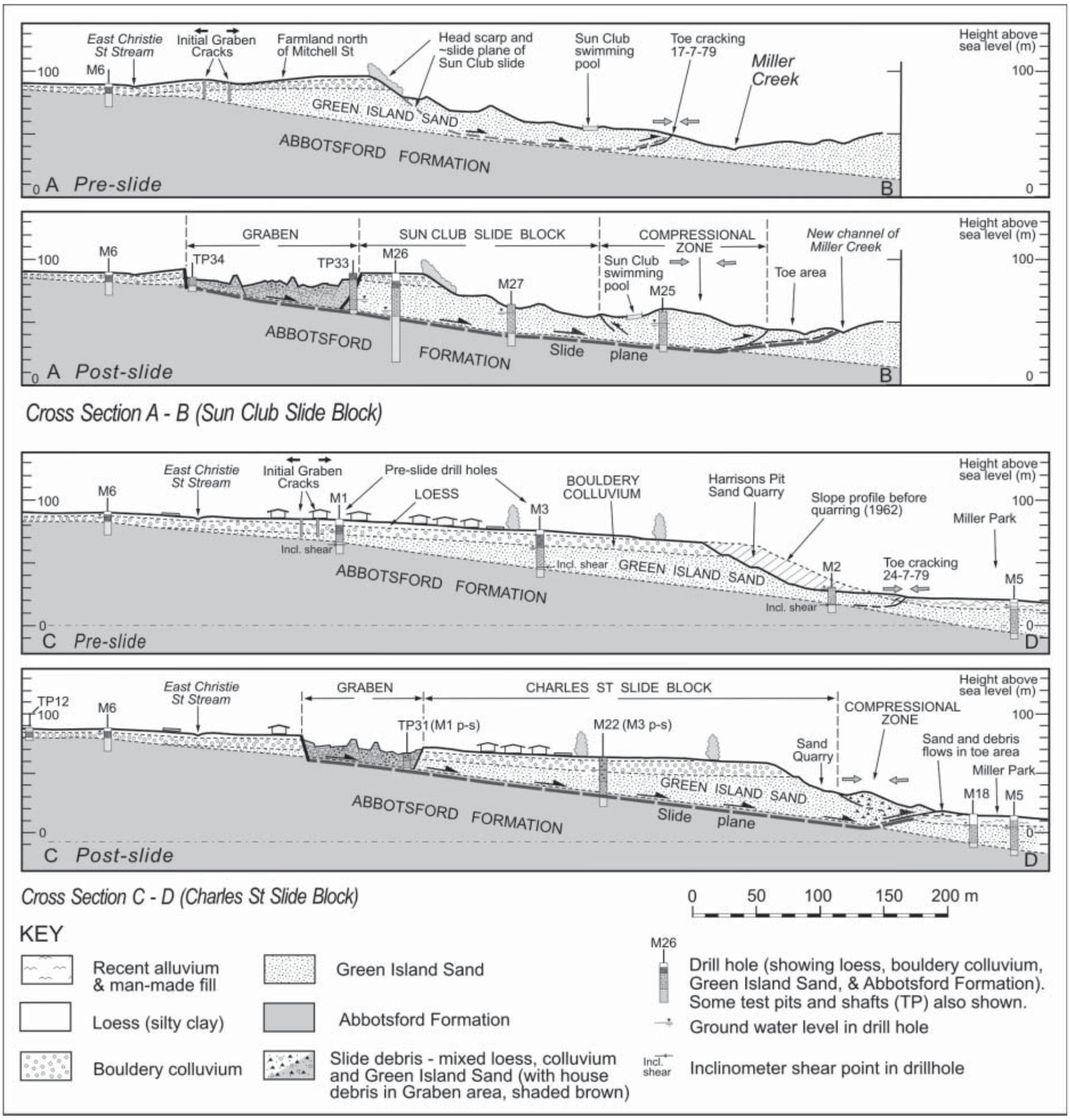


Figure 12: Cross sections of the Abbotsford Landslide showing the pre and post-slide topography, landslide features, and some investigation drill holes, test pits and shafts (*after Hancox et al., 1980*).

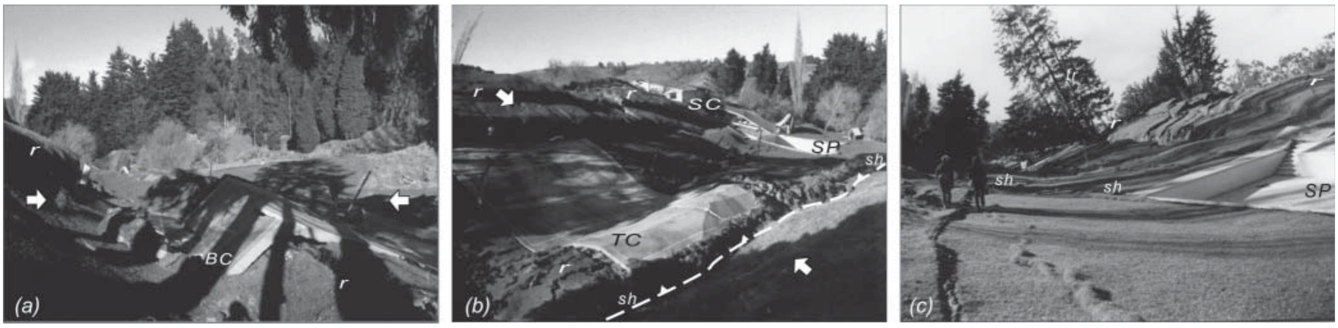


Figure 13: Three photos showing features in the Sun Club (*SC*) at the toe of the landside. Compressional ground deformation in this area formed compression ridges (*r*) which destroyed the badminton court (*BC*) and tennis court (*TC*) (photos a and b), tilted large pine trees, and deformed the swimming pool (*SP*) and moved it 38 m to the southeast (photo c). A low-angle shear plane (*sh*), which under-thrust the tennis court from right to left (*photo b*), defines the western (upslope) boundary of the zone of compression. [*Photos (a) and (b) by G Hancox; Photo (c) by D Coombs.*]

and were later transported away from the area to new sites elsewhere. The Commission of Inquiry found that the declaration of the state of emergency by the Mayor of Green Island was a timely and courageous decision, which was made despite opposition due to liability concerns, and the council could not be faulted for the part it played as the landslide movements increased (CoI 1980).

4. Description of the Abbotsford Landslide

Investigations carried out after the final movement on 8 August showed that the Abbotsford Landslide was a deep-seated block slide (as defined by Cruden and Varnes 1996). The landslide affected 18 ha, and had an estimated volume of about 5 million m^3 (an area 800 m x 400 m, and up to 40 m thick, Hancox et al. 1980). The main blocks of the landslide mass moved 50 m down a 7° bedding-controlled slip surface in 30 minutes (average speed 1.7 m/min). A graben formed at the head, with compressional ridges and sand flows at the toe. Several photos (Figures 7–10) and a geomorphic map (Figure 11) show this morphology and other features of the landslide. The subsurface geology of the landslide is shown by pre and post failure cross sections (Figure 12).

The landslide mass contained well defined geomorphic features and deformation zones. The chaotic graben at the slide head was the most obvious feature (Figure 9). Most of the slide mass was undeformed, but it was separated by a medial shear zone into two slide blocks. The Charles Street Block (with sand pit) moved ~12 m more than the northern block and ran out across the western side of Miller Park. The Sun Club Block, formed below the widest part of the graben, contained the largely intact Sun Club Slide, the toe of which dammed Miller Creek (Figures 10 and 11). Both slide blocks had wide zones of compression ridges and long over-thrust shears at the toe (Figures 11, 12, and 13).

There were also complex lateral zones of shear deformation at the north and south ends of the slide mass (Figure 11). The toe of the landslide consisted mainly of Green Island Sand (Figure 12). This material was the source for sand and debris flows seen at the toe after movement of the main slide blocks had ceased. Green Island Sand in the slide toe area appeared to have flowed (rather than liquefied) as groundwater drained from the slope after the failure. Investigation drill holes and shafts completed before and after the failure showed that the slide plane was located in a thin layer of weak clay at the top of the Abbotsford Formation (Figure 12).

5. Post-slide geotechnical investigations

Geotechnical investigations of the Abbotsford Landslide continued after the final movement on 8 August 1979. The investigations began on 11 August, primarily to assess the stability of the landslide area and the risk of further slope movements. When the New Zealand Government's Royal Commission of Inquiry into the Abbotsford Landslip Disaster was announced on 20 August the scope of the investigations widened to include establishing the cause of the landslide. The post-slide geotechnical investigations (see Figures 11 and 12) and activities included:

(a) Drilling of 18 small-diameter (N-size) fully cored drill holes, 11-59 m deep (M11–28).
(b) Inclinometers in 8 drill holes (M11–17, and 26), with standpipe groundwater monitoring.
(c) Sealed piezometers in 6 holes (M12–22, 27, 28).
(d) Six back-hoe test pits, 3–6 m deep (TP 12–15, 22, and 23).
(e) Seventeen Calweld shafts (900 mm), 10–30 m deep (TP 16, 18–21, and 24–35).
(f) Soil testing (Atterberg limits, densities, grading, consolidation, mineralogy, permeability, and triaxial (sand) and shear box (clay layers) strength).
(g) Geomorphic and geological mapping (see Figure 11).
(h) Establishing the geological and historical record of landslides and ground movements in the area, rainfall, earthquakes, pre and post-slide groundwater conditions, natural erosion, and

the activities of man (removal of vegetation, sand pit excavation, leakage of water pipes, and urbanization of the area).

(i) Limit equilibrium slope stability analyses.

(j) Evaluation of the conditions that could have contributed to the cause of the landslide, and the main factor(s) that triggered movements leading to the final slope collapse.

The main focus of the post-slide geotechnical investigations discussed above was to determine the cause or causes of the landslide and its long-term stability. Those investigations were spread over several months after the failure. The investigation programme was designed to be completed, analysed and reported on by the start of the Commission of Inquiry in March 1980.

The information obtained from the investigations was shared by all organisations involved, who prepared their own submissions on the cause of the landslide. The results of the geological and geotechnical investigations by DSIR were presented in several reports (Bishop et al. 1979, Hancox et al. 1980, and Salt et al. 1980). The laboratory soils testing was carried out and reported by the DSIR Soil Bureau (Northey and Barker 1980) and the Ministry of Works and Development (Millar and Turnbull 1979). All of these reports were submitted to the Commission of Inquiry and taken into account in their report on the disaster (CoI 1980).

6. The cause of the landslide

6.1 Information from geotechnical investigations

During the pre-slide investigations, inclinometer shear points in four drill holes showed that the slide plane was located 10–30 m below the ground surface, close to the top of the Abbotsford Formation (Table 1). This was confirmed after the 8 August movement when the failure surface was located in a 25 mm thick clay layer, about 1 m below the interface between the Green Island Sand and the Abbotsford Formation (in TP 34, Figures 11, 12, and 17). The clay layer, which showed slickensides indicating that movements had occurred along it, contained up to 25% montmorillonite clay (Northey and Barker 1980) and had very low residual shear strength (ϕ', internal friction angle) ranging from 5–10° (Millar and Turnbull 1979).

Montmorillonite is a highly active swelling clay mineral that is well known for causing slope instability problems in Dunedin and other parts of New Zealand (Crozier et al. 1992; Stout 1971). Because the clay layers were on the slide plane, they were foremost among a number of factors

Table 2: Summary of factors that may have contributed to the Abbotsford Landslide.

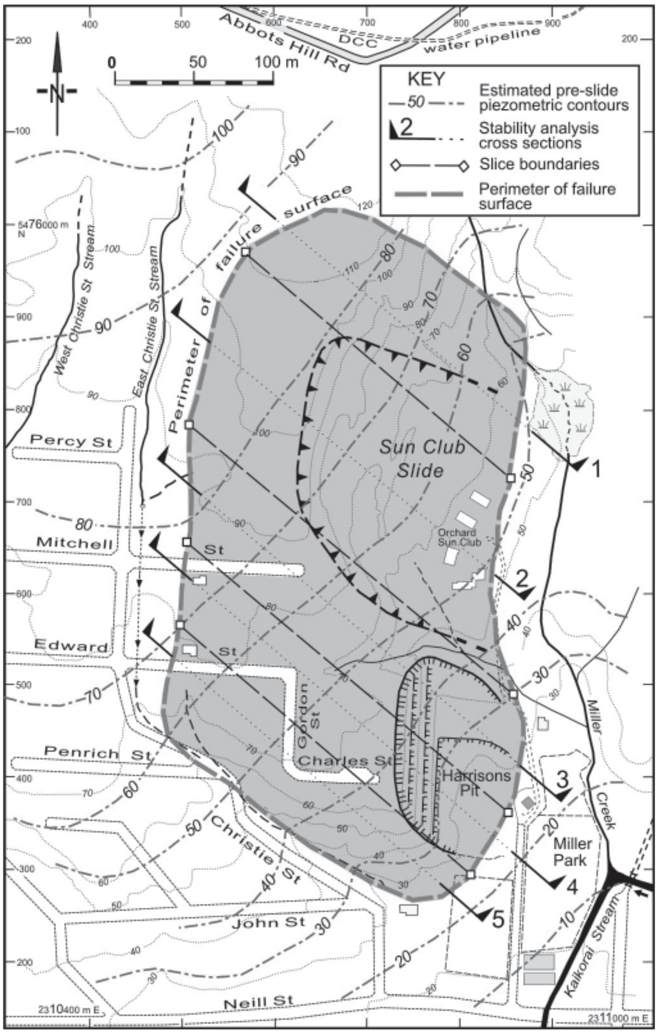
SIGNIFICANT
1.Unfavourable Geology and Topography (a) Landslide formed on a dip slope -Tertiary sediments dip ~7–10° southeast towards the incised valley of Miller Creek (Figure 2). (b) Very weak clay layers at near residual strength (ϕ' 5–10°) aligned with bedding at top of Abbotsford Formation. (c) Progressive failure over several thousand years reduced strength of clay layers to close to residual strength. This process probably began after downcutting by Miller Creek more than 20 000 years ago. (d) The prehistoric Sun Club Slide (at least 10,000 years BP) illustrates the inherent instability of the slope on the west side of Miller Creek. Slope stability was further reduced by stream erosion at the toe of the old slide.
2. Increased pore water pressure Limit equilibrium analyses showed that the slope that failed was sensitive to changes in the water table. A uniform rise of 1 m in groundwater level in the area was found to reduce slope stability by 2-3%. A long-term rise in the water table in the landslide area is likely to have occurred because of two important factors: (b) Increased rainfall over the last 10 years prior to the failure (following a 20-year drier period, Figure 18). (a) Leakage from the *Dunedin City Council (DCC) 750 mm water pipeline* at the top of the slope (Figures 5 and 19).
3. Quarrying of sand at the toe of the slope (a) Excavation of c. 300,000 m^3 (6% of the slide mass) from Harrison's Pit from1964-69 reduced stability of the overall slope by c. 1% (locally the reduction would have been higher). This was not sufficient to bring about immediate failure of the slope. Unequivocal signs of widespread, deep seated slope instability in the landslide area (broken water mains along the graben cracks) only became apparent in September 1978. Minor house damage at toe in 1968 initially interpreted as local instability. (b) The overall effect of sand excavation was to ensure that groundwater levels needed to rise 0.3 m less to achieve the same stability conditions as existed prior to the landslide, and so to that extent could have influenced the timing of failure.
LESS SIGNIFICANT
4. Removal of natural vegetation Possibly minor effect due to slightly increased runoff and infiltration of rainfall, and hence a rise in the water table. The lack of data about previous vegetation and runoff characteristics makes it impossible to quantify these factors.
5. Urban development (excluding DCC water pipeline) The addition of houses, roads, and services resulted in very small increased surcharge (weight) on the slope (affected 15% of slope area). Increased interception of rain (houses, roads) possibly offset deficiencies in storm water reticulation. Local breaks in water mains (1978 and 1979) resulted from early slide movement, and were detected and repaired. Vibrations from motorway too small to have affected slide area.
6. Earthquakes Not relevant. No significant earthquakes (MM 5 or greater) in the area in the 3 years prior to the slide. The strongest historical earthquake (M 5, MM VII in April 1974) did not affect the slope. The effects of prehistoric earthquakes are unknown.

examined to determine the cause of the landslide.

The rapid displacement on 8 August was the final movement of the Abbotsford Landslide. A number of slump scarps up to 2 m high developed across the end of Charles Street a week after the collapse (Figure 11), but no further ground displacements were detected in the area from August 1979 to March 1980 (Hancox et al. 1980), and no movements have occurred subsequently. Following the rapid 8 August movement the landslide area was found to be stable, with a factor of safety (FS) of 1.10 without the need for any stabilization measures (Salt et al. 1980). The built-up areas adjacent to the landslide area south and west of Christie Street were also judged to be stable (FS 2.2).

After substantial earthworks were carried out to fill in the graben and regrade the headscarp and toe areas, the landslide area was re-zoned as a reserve in which any future building and development has been prohibited.

Figure 14: Map of the Abbotsford Landslide area showing pre-slide topography and piezometric contours at the time of failure, the perimeter of the failure surface, and locations of critical slices and cross section lines used for stability analysis (*from Salt et al. 1980*).



6.2 Factors that contributed to the landslide

A range of natural and human factors were examined by the organisations involved in the investigations (DSIR, Ministry of Works and Development, Otago University, local authorities) to determine the cause of the landslide. The principal factors considered included: unfavourable geology and topography, increased groundwater levels (due to higher rainfall and leakage of water pipes), quarrying of sand at the toe of the slope, removal of the natural vegetation, urban development of the area, and seismic activity (CoI 1980; Hancox et al. 1980; Salt et al. 1980). These factors and their effects are summarised in Table 2.

6.3 Stability Analysis

In order to determine the significance and relative contributions of factors that may have contributed to the cause of the landslide, especially the sand excavation at the base of the slope and changes in groundwater level within the slope, slope stability analyses were carried out by several independent organisations. All analyses adopted the 'block slide' concept of the failure, and used two-dimensional computer programmes. The analysis method used was Janbu's theory of generalised procedure of slices and limit equilibrium (Janbu 1973), which allows interslice forces to be controlled (CoI 1980).

Investigations by DSIR into the cause of the landslide (Salt et al. 1980) included estimation of pre-slide piezometric contours across the landslide area (Figure 14) and limit equilibrium stability analyses using the Janbu method. For analysis purposes the landslide was divided into a number of slices. Locations of the five critical sections analysed are shown in Figure 15. The input geometry (ground surface, piezometric surfaces, and shear surfaces) of these cross sections are shown in Figure 15, while details of the critical cross section through

Harrison's Pit (Section 4) are shown in Figure 16. This shows the locations of the clay bedding plane failure surface, the critical pre-slide shear surface, the active shear zone at the slide head (graben) and passive shear zone at the slide toe conditions, and the estimated pre-slide piezometric surface. The soil parameters used in the analysis, based on the results of soils testing by Miller and Turnbull (1979) and Northey and Barker (1980), are as follows:

(a) Average bulk density: $= 2.0$ t/m^3

(b) Strength of undisturbed Green Island Sand:
– $c' = 0$, $\phi' = 33°$

(c) Strength of disturbed Green Island Sand:
– $c' = 0$, $\phi' = 28°$

(d) Strength of clay layer (failure surface – see Figure 17): average from residual shear tests – $c' = 0$, ϕ ~9.5°

(e) Strength of clay layer (failure surface): back-analysis shear strength at failure – ϕ' ~8° (for $c' = 0$)

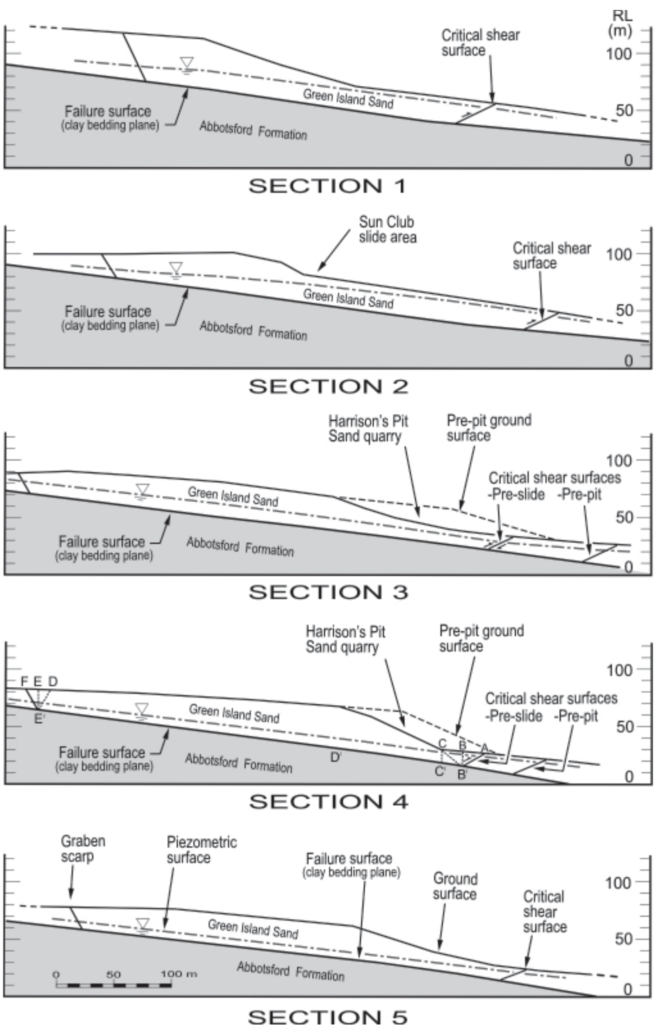


Figure 15: Cross sections of the Abbotsford Landslide used for stability analysis (*from Salt et al. 1980*). Piezometric data, shear surfaces, and geology based on pre-slide drill holes (Figures 11 and 12). Overlying colluvium and loess not shown.

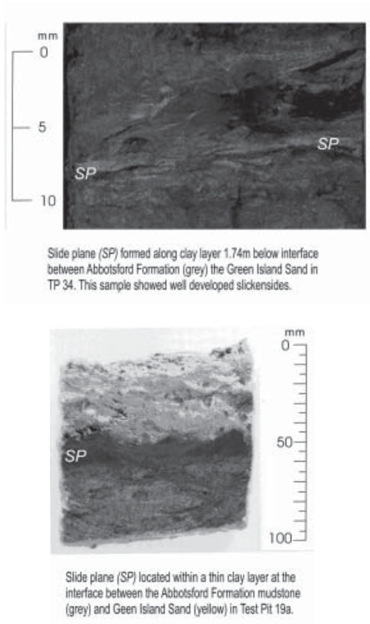


Figure 17: Photos of the slide plane showing the thin clay layers that were tested for shear strength

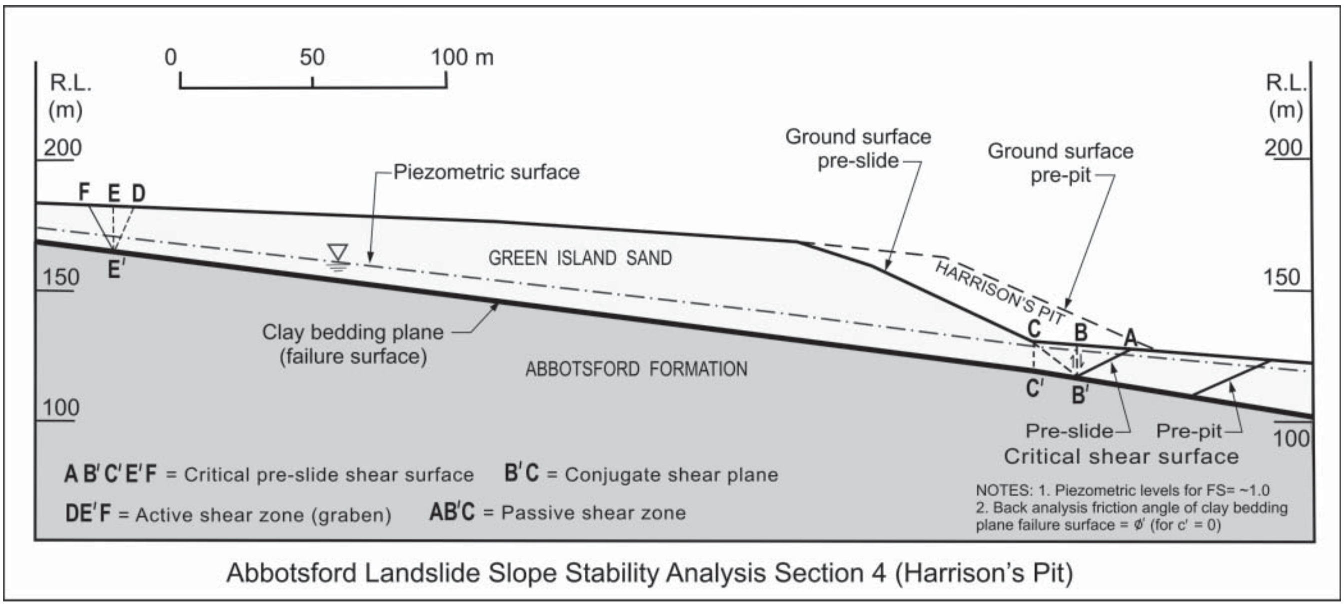


Figure 16: Cross Section 4 through Harrison's Pit showing details of the model used for the stability analysis.

Pre-slide	**Sun Club Block**		**Charles Street Block**			
Section	1	2	3	4	5	Margins
Available shear force (MN)	2857	3452	2272	1779	1681	740
Mobilised shear force (MN)	2962	3607	2145	1741	1586	740
2-D Factor of Safety (FS)	0.964	0.957	1.059	1.022	1.060	
Overall Factor of Safety (= total available/total mobilised forces) = 12800/12800 MN FS = 1.000						
Pre-Harrison's Pit	**Sun Club Block**		**Charles Street Block**			
Section	1	2	3	4	5	Margins
Available shear force (MN)	2859	3455	2948	2190	1683	740
Mobilised shear force (MN)	2963	3608	2781	2115	1597	735
2-D Factor of Safety (FS)	0.965	0.958	1.060	1.035	1.054	
Difference in FS post-pit (%)	- 0.1	- 0.1	- 0.1	- 1.3	+ 0.6	
Overall Factor of Safety (= total available/total mobilised forces) = 13900/13800 MN FS = 1.007 Difference in FS post-pit: = - 0.7 %						
Notes: All forces in Mega Newtons (MN). Data from Salt et al. 1980. Section locations shown in Figure 14, input geometry in Figures 15 and 16.						

Table 3: Summary of Abbotsford Landslide slope stability analysis data for critical sections.

A summary of the analysis results data for the critical sections analysed is given in Table 3.

6.4 Results of stability analysis

The main conclusions reached by DSIR (Salt et al. 1980) from the stability analysis results summarised in Table 3 and Figures 15 and 16 can be summarised as follows:

(a) Back analysis suggested that a friction angle of about 8° (for c′ = 0) was available on the clay bedding plane at the time of failure (for a factor of safety = 1.000).

(b) The Harrison's Pit sand excavation reduced stability of the overall slope by 0.7 % (FS post pit = 1.007, Table 3), based on the piezometric levels shown in Figures 14–16. A similar (0.7 %) reduction in stability of the pre-pit model also resulted from a 0.3 m rise in groundwater levels across the slide area. This equates to a 2–3% decrease in stability per metre rise in long term ground- water levels in the area. Similar results were obtained by the Ministry of Works and Development (Ramsay 1980).

(c) Removal of sand from Harrison's Pit had two opposing effects. A stability decrease resulted from the reduced length of the critical shear surface through the Green Island Sand at the slide toe. To a realistic level of accuracy it was concluded that the pit decreased stability of the slope by about 1%. However, this was probably offset by a small decrease in the long term pore-water pressures in the toe area (Figure 16).

The major excavation in Harrison's Pit from 1964 to 1969 occurred during a succession of years when rainfall was less than average. However, an increase in the long-term groundwater levels in the area is likely to have occurred because of increased long term rainfall over the ten years prior to the failure (Figure 18).

6.5 Findings on the cause of the landslide

Several submissions to the Commission of Inquiry in 1980 (e.g. Coombs et al. 1980, Hancox et al. 1980, Ramsay 1980, Salt et al. 1980) concluded that the thin clay layers in the Tertiary strata dipping 7–10° southeast towards Miler Creek were of fundamental importance in the development of the landslide. However, other factors also contributed to the failure, notably the quarry at the base of the slope, and the probability that groundwater levels in the landslide area were in an elevated state due to a wetter than average last decade (Figure 18), leakage from the Dunedin City Water Pipeline, and water from breaks in local water mains during the earliest pre-cursory movements. The main findings on cause of the Abbotsford Landslide may summarised as follows:

(1) Geology and topography

The topography and underlying geology of the area created the basic pre-conditions for instability of the slope formed in weak sedimentary rocks. The prehistoric Sun Club Slide is evidence of the long-term instability of the slope that failed.

The Abbotsford Landslide resulted from failure on thin, very weak (c' = 0–20 kPa, ϕ' = 5°–10°), clay layers near the top of the Abbotsford Formation dipping downslope ~7°. Progressive failure on these clay layers began several thousand years ago with the erosion of

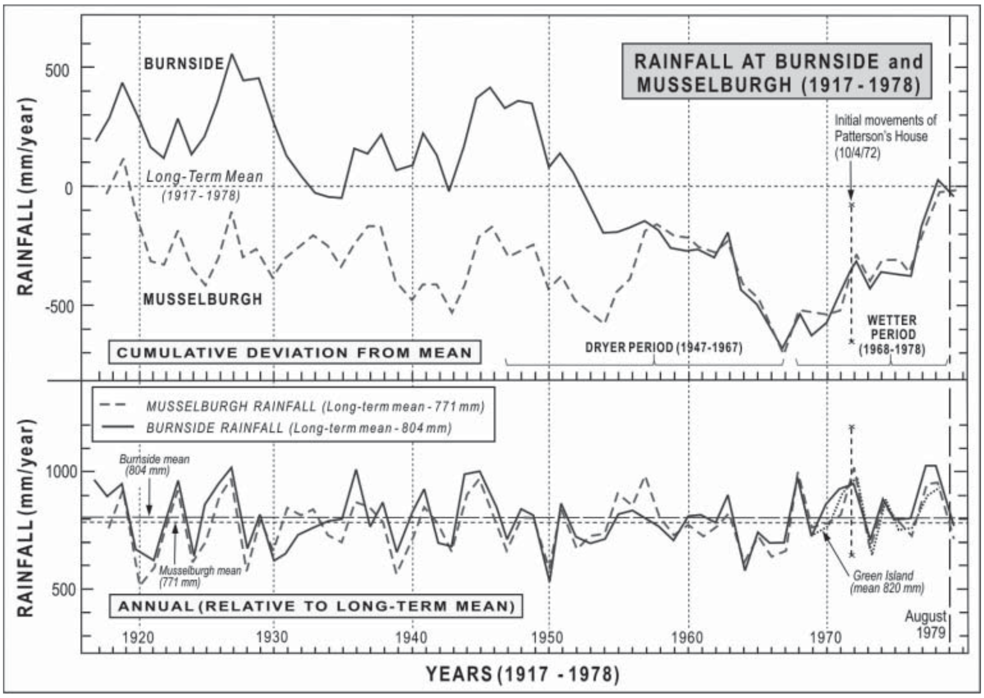


Figure 18: Graphs showing the annual rainfall from 1917 to 1978 in the Dunedin suburbs of Burnside (~1.5 km east of the landslide) and Musselburgh (~7 km east). The plots show long-term mean annual rainfall and cumulative deviation from the mean in both these areas. Mean rainfall at Green Island is also shown for 1969-1978. (*Based on data from the NZ Metrological Service*).

Miller Creek, and was important in the development of the slide (Salt et al. 1980).

(2) Harrison's Pit sand excavation

Limit equilibrium analysis showed that the excavation of c. 300,000 m^3 of sand from the base of the slope reduced stability of the overall slope (slide area) by about 1 %, but that was insufficient to immediately trigger the landslide (Salt et al. 1980, Ramsay 1980). One house on the side of a gully 60 m south of the pit developed cracks in 1968–72 which may have been caused by the initial movements of the landslide, but this was the only house affected until 1978. Unequivocal evidence of ground movement that eventually led to the Abbotsford Landslide first became apparent in September 1978, when water mains ruptured in Christie Street, about 250 m upslope from the sand pit (Figure 3).

Although the excavation at the base of the slope did not cause it to fail immediately, it reduced stability of the slope, and in combination with other factors it advanced the onset and timing of the failure process. From the evidence presented, the Commission of Inquiry found that the excavation of Harrison's Pit was a causative factor in the landslide. However, because there was very little reliable information about the history of groundwater levels in the landslide area before the slope failed, the Inquiry concluded that it would never be possible to determine the proportionate effect of the sand pit excavation (CoI 1980).

(3) Groundwater levels

As demonstrated by the slope stability analyses, changes in groundwater level across the slide area are likely to have had a significant effect on stability of the slope. A uniform 1 m rise in groundwater level over the landslide area resulted in about a 2–3% reduction in stability of the slope. A rise in the long-term groundwater levels in the area was considered likely because of (a) increased long-term rainfall, and (b) leakage from the DCC water pipeline, as explained below.

(a) Long-term rainfall: Graphs of the long-term rainfall in the Abbotsford Landslide area (Figure 18) show that the major excavation of Harrison's Pit (1964–1969) occurred during a succession of years (1958–1967) when rainfall was generally less than average. However, from 1968–1978 the rainfall was generally higher than average. The initial slope movements appear to have begun during the latter period, probably causing the early movement and damage to Patterson's house at the foot of the slope in April 1972. The ground movements and final failure in August 1979 occurred after a succession of years which were generally wetter than average. No similar prolonged trend of this magnitude occurs in the Dunedin rainfall records dating back to 1918 (Figure 18). The long-term rainfall pattern is thus seen as the prime factor in the timing of the Abbotsford Landslide events (Salt et al. 1980).

(b) DCC water pipeline leakage: Dunedin City Council (1980) told the Inquiry that prior to the landslide water had been leaking (up to 14 litres/min) from the DCC pipeline (a segmented 750 mm concrete pipe with rubber ring joints) upslope of the landslide slide area (Figure 19). Although the volume of water that leaked from the pipeline was not accurately established, Green Island Borough Council estimated it was about 5 million litres per year prior to the landslide (Garden and Partners 1979). This represented an average leakage rate of 6–9 litres per minute, with 4–5 litres per minute likely to infiltrate the ground in the slide area (CoI 1980). Based on information presented at the Inquiry, the average annual infiltration rate of rainfall into the landslide area was about 10 litres per minute. Hence the water leaking from the pipeline, possibly since 1976 or earlier, was roughly equivalent to a 40–50% increase in rainfall for several years prior to the final slope movement.

The Commission of Inquiry found that, although the exact amount of water reaching the landslide area from the leaking pipeline could not be precisely determined its effect on the landslide mass could have been significant (CoI 1980).

It is possible, therefore, that the combined effects of greater than average rainfall over the past decade, together with water from the leaking DCC water pipeline, raised the groundwater levels in the landslide area and advanced the onset of slope failure. However, because insufficient groundwater level data were collected during the 1979 pre-slide investigations, it was not possible to precisely determine the effects of water leakage on the landslide area. However, notes on driller's records and extensive sand flows at the toe of the slide after the final movement (Figure 11), suggested that groundwater levels in the landslide area prior to and during the final movement were very high.

(4) Other factors

Other effects such as the removal of vegetation, urban development in the area, and earthquakes, were generally thought to be insignificant. The lack of any significant earthquakes in the Dunedin area during the three months prior to the movement on 8 August indicated that the landslide was not triggered by an earthquake. Recent studies of historical earthquake-induced landsliding in New Zealand (Hancox et al. 1997, 2002) suggest that large creeping, low-angle landslides in Tertiary mudstone are not triggered or obviously affected by earthquakes. The strongest historical earthquake (M 5, MM VII in April 1974) did not affect the slope. The effects of prehistoric earthquakes are unknown.

7. Discussion

Effectiveness of investigations: The pre-slide geotechnical investigations accurately established the depth and rates of movement of the developing landslide, and the location of the slide plane within very weak bedding plane clay layers. However, the slide mechanism and (triggering) cause of the landslide could not be conclusively determined because groundwater levels in the area were not accurately defined before the 8 August movement.

Figure 19: Dunedin City Council 750 mm water pipeline under repair (*Pit W7*) 200 m north of the landslide area (*Fig.11*) on 13 August 1979. (*photo by courtesy of ER Garden and Partners*).

This critical information was not obtained mainly because there was insufficient time to properly install and prove the standpipes and piezometers or because they were sheared off or blocked by ground movements. There were also difficulties because there was limited practical experience in investigating a large moving landslide, and a lack of equipment to measure groundwater levels in drillholes that were continuously being deformed and blocked by ground movements.

With a wide range of new techniques and geotechnical equipment now available for monitoring rates of landslide movements and collecting piezometric data, better results would be expected in similar circumstances today. For example, much improved equipment is currently being used by GNS Science at Taihape in the central North Island to investigate a very large (45ha), deep-seated, creeping landslide with over 200 homes. A laser survey network and data-loggers are being used there to monitor near real-time surface movement, rainfall, and groundwater pressures in the landslide (Massey and Palmer 2007).

Assessment of landslide risk and response: Investigations before the final movement of the Abbotsford Landslide on 8 August accurately defined the rates and depths of the developing ground movements, and the extent of the areas likely to be affected by a sudden collapse of the entire slope. Survey monitoring across the developing ground cracks showed that the rates of movement were increasing exponentially the week before the final movement, but it was difficult to predict the exact acceleration of the landslide, or how and when the slope would break up. The movement rate predicted on 5 August (450 mm/day and 4 m total by 12 August) proved to be lower than what actually occurred, with much faster movements (430–670 mm/day) recorded by 6–7 August. However, the high risk presented by the landslide was appreciated, as was shown when the state of emergency and evacuation plans were announced on 6 August. When the final rapid movement occurred on 8 August, much earlier than anticipated, 17 people were still in residence on the sliding mass.

Although those people were unharmed, their survival was more a matter of chance rather than management. The author recalls that in 1979 it was a widely held view that the people on the landslide should have been evacuated earlier. Unfortunately, over the two days prior to the final movement on 8 August, the survey monitoring data were not plotted up and analysed quickly enough for the rapid increase in movements on 6–7 July to be recognised. This meant that the area was not completely evacuated before the final movement. In spite of anecdotal reports from some residents of increased ground movements over that critical pre-slide period, those reports were not taken seriously and acted on by the authorities in charge during the state of emergency.

The Commission of Inquiry found that the local council could not be faulted for the part it played as the landslide developed, and that the declaration of the state of emergency was a timely and responsible action. However, the CoI also considered that the consequences of rapid slope collapse would have justified the authorities (civil defence, police) requiring complete evacuation of the landslide area earlier than what actually occurred. Despite the possibility of an over-cautions prediction about how the landslide might develop, the Commission found that in such cases human safety should be the overriding consideration, and that decisions should be made assuming the "worst possible" rather than the "most probable" eventuality (CoI 1980). That approach to managing landslide hazards and risk remains equally valid today.

The cause of the landslide: The Abbotsford Landslide occurred in an area with a well known history of slope instability (Benson 1940, Gordon 1960). By present-day standards that association would call for greater vigilance in development of the area, particularly expansion of the sand pit adjacent to the prehistoric Sun Club Slide at the base of the slope. That landslide had been identified when the DCC pipeline route was evaluated (Benson 1945), and was again noted during investigations for the motorway (Gordon 1960). It was ironic that most of the sand removed from the toe of the slope, thereby reducing its stability, was used to stabilise a nearby landslide reactivated by motorway construction.

The Commission of Inquiry supported the DSIR's conclusion that the presence of very weak clay layers containing montmorillonite within a 7–10º dip slope was the basic precondition for the landslide, with increased rainfall and two human actions – the sand quarry and the leaking DCC water pipeline – identified as contributing factors. Although groundwater levels and the volume of water leakage could not be precisely determined before the slide, the overall effect of the sand excavation was that the water table in the area had to rise about 0.3 m less in order to reach the critical stability condition at the time the slope failed on 8 August. The sand excavation reduced stability of the overall slope by about 1%, and except for small local movements in one area at the toe, the slope remained essentially stable for ten years after the quarry was closed.

This suggests that the sand excavation did not trigger the onset of slope movements in September 1978 which culminated in the final movement on 8 August 1979. Raised groundwater levels are inferred in the slide area due to significantly increased long term rainfall, rupture of pipes within the slide area, and leakage from the DCC water pipeline at the top of the slope. Leakage from the pipeline alone was possibly equivalent to about a 40–50% increase in rainfall over several years prior to the landslide.

Therefore, in the author's opinion, a long term rise in groundwater levels in the landslide area controlled the timing of the failure, and in this sense is believed to have triggered the final movement of the Abbotsford Landslide on 8 August.

Leakage from a segmented water pipeline laid in unstable ground was also recently recognised as a prime cause of the 1997 Thredbo landslide disaster in Australia (Hand 2000). Both the Abbotsford and Thredbo landslides are graphic reminders of the potential destabilising role of groundwater in slope failures. Through increased awareness of the effects and dangers illustrated by both of these disasters it is hoped that they may serve some useful purpose in preventing similar events in the future.

The Commission of Inquiry believed the very weak, montmorillonite-rich clay layers that controlled development of the Abbotsford Landslide to be unique in terms of New Zealand experience. However, it is now well established that similar weak clay layers are relatively common in Tertiary sediments in many parts of New Zealand, and they are well known for causing slope stability problems. For example, large stratigraphically-controlled earth flows and block slides are known in Otago (Benson 1940, 1946; Crozier et al. 1992) and Canterbury (McSaveney and Griffiths 1987).

Similar large block slides and earth flows are also known in the central North Island, for example, at Utiku where the NIMT railway line and State Highway 1 were for many years affected by a very large block slide until it was stabilised by drainage works (Ker 1970, Stout 1971). Further to the north a large part of Taihape township in the Central North Island is located on an old block slide (Thompson 1982). Parts of this landslide are today actively creeping, and the area is currently being monitored to measure the ongoing slope movements (Massey and Palmer 2007).

Development of deep-seated landslides in Tertiary rocks has generally been attributed to the presence of weak thin layers of montmorillonite clay, in association with high groundwater levels caused by above average long-term rainfall. The Abbotsford disaster therefore draws attention to the potential slope instability hazard caused by unfavourably oriented bedding in weak sedimentary rocks containing clay layers, even in gently sloping areas with moderate relief.

The Abbotsford Landslide is the most damaging single landslide to have occurred in an urban area in New Zealand. After 30 years it still provides a valuable reminder of the potential dangers of large excavations at the base of slopes, and leakage from water mains in urban areas. More generally, the Abbotsford Landslide points to the need for geotechnical investigations to precede urban development. Following extensive earthworks to regrade the graben and toe areas in 1979 after the landslide occurred, the slide area was judged to be stable, with little potential for reactivation. It remains so today.

8. Conclusions

The Abbotsford Landslide is the most damaging single landslide to have occurred in an urban area in New Zealand. After 30 years it still provides a valuable and timely reminder of the potential dangers of excavations at the base of slopes, leaking water pipes in urban areas, the need for investigations to assess landslide hazard and risk, and appropriate responses by the authorities to manage and reduce the risk of property damage and loss of life.

Geotechnical investigations enabled the extent, and nature of the risk presented by the Abbotsford Landslide to be determined as it developed over the two months prior to the final movement on 8 August 1979. However, because ground movement data were overlooked in the two days prior to the failure, the area was not completely evacuated when the rapid failure occurred. Undeniably, the people who remained on the slide mass were fortunate to survive.

In retrospect, it must be concluded that the assessment and management of the Abbotsford Landslide event was far from perfect. This case history underlines the value of plotting and analysing landslide movement data as soon as it is collected. It also shows that human safety should be the prime consideration in landslide risk assessment, and it is generally better to make decisions based on the "worst case" rather than the most likely outcome.

Unfavourable geology and topography (weak clay layers dipping 7° downslope towards an incised valley) was the main underlying cause of the landslide. A former sand quarry at the foot of the slope and a leaking DCC water pipeline above the slide area were found to be man-made factors that contributed to the cause of the failure. Because the quarry closed 10 years before the landslide occurred, it is concluded that a long-term rise in groundwater levels, due to increased rainfall over the previous decade and leakage from the pipeline, controlled the timing of the failure, and in this sense is considered to have triggered the landslide.

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An Engineering Geologists Observations

Nick Rogers is a Principal Engineering Geologist at Tonkin & Taylor Ltd. In 1979 he was a Geologist working for Brickell, Moss & Partners, in Auckland when he was asked to go to Abbotsford. He arrived the afternoon before the main failure. Nick talks to editor Paul Salter.

When did you first hear the name Abbotsford?
I was aware of the suburb, and also the Abbottsford Formation mudstone, from previous project work in Dunedin, and also large block slides from working on the landslides from Puketeraki to Evansdale for NZ Railways.

How did you end up working in Abbotsford?
When it became obvious that significant movement was occurring on the Abbotsford landslide, the Lower Hutt office of Brickell, Moss & Partners, which later became Connell Wagner, was engaged by the Earthquake and War Damage Commission to provide technical advice. Investigations, including mapping and drilling were being undertaken, but I was sent down to see if Council should have been aware of instability in the area.

What was the level of understanding of the landslide slide mechanisms when you arrived?
I arrived in Dunedin at about 4:00pm on August the 8th, 5 hours before the main failure initiated, and undertook a brief visit of the toe area just before dark. The toe rolls and compression were clearly evident. The geology of the landslide was fairly well understood. Monitoring of the landslide showed that movement was continuing with periods of acceleration but the general consensus was that the displacements would continue to be relatively small. An imminent catastrophic failure was not expected.

So, when did you hear the landslide had "gone"?
It was pre-mobile phone days and I didn't hear any radio or TV reports that night. I got a phone call in the morning, was informed about the landslide and headed up there.

What did you find when you got to site?
The movement had stopped by then and all the residents had been taken off the slide blocks. The locals were shocked and stunned. The area was under Civil Defence control and there was a briefing for an aerial reconnaissance to check immediate risks in the slide area. I accompanied Ian McKellar, the DSIR District Geologist, and Errol Chaves from the MoW on a helicopter fly over. The most obvious features were the massive grabens at the head of the landslide and extensive zone of compression at the toe in Miller Park. The landslide had dammed Miller Creek which presented a potential dam burst scenario, and a digger was instructed to make a controlled release. Later that day we were allowed into the slide area on foot and rescued a few pets.

Was landslide assessment, in the 1970's much different to today? e.g. the current Saito methodology for predicting failure timing. And, given the landslide was a deep translational slide, would we have recognised a potential slide at this location any better today?
The technical understanding of landslides in the late 1970's was reasonably strong - probably just as strong as it is now. We understand a few things better, but the quality of work back then was as good or better than nowadays. There was prior evidence that the Abbotsford area was unstable with the old Sun Club landslide identified and movement elsewhere associated with the State Highway 1 realignment. We would probably have a similar amount of evidence to suggest instability at this location as we did back then.

What was the geotechnical legacy of Abbotsford?
The Abbotsford landslide itself didn't significantly change the technical approach to investigating landslides – the industry best practice was strong back then too. However, the subsequent Commission of Inquiry did result in changes to the Earthquake and War Damage Commission (now EQC) Act. Prior to Abbotsford they didn't have the ability to act before damage occurred. EQC can now protect property against "Imminent Loss". The Local Government Act 1974 was also changed after Abbotsford to prevent development on unstable land. However, this caused some significant problems for Councils who had already permitted developments on unstable land and by 1981 these changes had been repealed and Councils could allow development on unstable land and not be under any civil liability.

Do you have any other comments?
The Abbotsford landslide gained a lot of attention because it was photogenic and people were surprised this kind of thing could happen, but only a month earlier during the Telethon weekend of June 30/July 1 there had been a severe rainstorm in Auckland that caused landslides which damaged hundreds of homes. We shouldn't have been surprised this kind of thing could happen. Nowhere is risk free and, geologically, New Zealand is a risky place to live. We need to make sure the wider public understand the geotechnical risks and accept that this is just part of living in NZ. Just make sure that you are properly insured.

Living through a Major Urban Landslide – Media and residents recollections

Above: "Road Works". Photo courtesy of The Otago Daily Times

Paul Salter reviews accounts from the 1980 Commission of Inquiry report, media reports of the time, and talks to some of the residents who lived through the landslide.

By 1978 some residents of the hillside suburb of Abbotsford had got used to filling minor cracks in their houses. But in early June significant cracking started to be reported in numerous homes. When a water main broke in Mitchell Street, in May 1979, Council called in consulting engineers to report on the problem. They found fresh cracks in the ground and in nearby basements and concrete paths. A full scale investigation was launched and within a few days a large landslide was suspected to be the cause. The water mains continued breaking, were repaired, then broke again.

By the end of June, newspapers were reporting on the issue and Civil Defence and the Earthquake and War Damage Commission were involved. Drilling investigations were started to assess the situation and by 27 June one family had to be relocated due to their house buckling off its footings.

Heading into the middle of winter, the movement continued to increase; water mains became unserviceable, large cracks propagated across paddocks near the head scarp and on Friday 13 July the local council gave residents a newsletter about the situation, including the steps to take when moving out and details of insurance cover under the Earthquake and War Damage Act.

By the end of July an Emergency Services Co-ordinating Committee had been established and most residents were now moving possessions out after another newsletter indicated "...movement has increased significantly to about 2 inches per day….". Demolition orders were issued on 3 more houses near the head scarp graben, including Graeme and Nancy Hooper's home at 11 Edward Street. "My husband had only finished the house 18 months earlier and it was very upsetting" recalls Mrs Hooper. "We would be sleeping and creaks in the wall would wake us up. The doors wouldn't close and the corners of the walls were bent.

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I felt like a drunk walking on the uneven floorboards". She recalls surveyors arriving everyday at 3:30pm to measure pegs outside and various engineers and valuers coming through the house, but feels there was too little information given to the affected residents. "My husband refused to accept the demolition order and ended up battling the Earthquake and War Damage Commission. Our house was very solidly built and could have been moved".

On 1 August, a week before the final failure, the landslide dominated the front page of The Otago Daily Times. Under "Loan Extension Offer for Landslip Victims" the paper detailed how residents who had lost homes may qualify for a second Housing Corporation loan. By this time Police were restricting access and residents were given windscreen stickers – to join their Carless Day sticker (which came into force on July 30, as part of the Government's response to the second Oil Shock).

In Abbotsford, rainfall and ground movement increased in early August. The landslide was moving about 100mm day, and considerable amounts of fill were needed to maintain road access. The head scarp crack was also opening at a rate that made maintaining services untenable. There were predictions the graben would be 2m wide within a week, and 4m shortly after, but the general consensus between technical experts was that no rapid "avalanche" was imminent. Despite this, the Mayor of Green Island strongly advocated declaring a state of local civil emergency, and after some debate, this was declared on the 6 August - requiring all residents to vacate the designated landslide area by Sunday 12 August.

The day of the main failure dawned cold and wet, with hail and snow forecast for Dunedin. The ODT headline, "Assistance for Slip Area", outlined a government offer of financial help and noted separate Rotary Club and Mayoral appeals were underway. The Herald's front page covered the aftermath of gang battles in Moerewa, Northland and news that the Wellington Seismological Observatory had recorded tremors from French nuclear testing at Mururoa atoll, with Dr W. D. Smith reporting a recent blast equivalent to 6.3 on the Richter scale had been detected. Inside the Herald, under "Home Insurance: Don't Let it Slip Too", home owners worried about landslide cover were reminded to check they have realistic fire insurance cover.

On the evening of 8 August a handful of Abbotsford families were still in their homes. The Emslie's lived at 9 Mitchell Street and were rolling up their lounge carpet when the slide started. Colin Emslie still lives in Dunedin and vividly recalls having to grab his walkie talkie and torch and call authorities as he, his wife, and two young daughters were carried downhill by the main slide. He described the experience in a 1996 NZ School Journal article – they had just finished rolling the carpet when a neighbour arrived at the backdoor shouting the ground was moving. At the same time Civil Defence phoned and told the family to get out. They reached the top of their driveway in time to see lampposts starting to lean and the power lines break, and spark across the road, putting the area into darkness, as a deep rumbling started. A large yellow cliff appeared in the moonlight, which seemed to be moving before the Emslie's realised they were the ones moving amid a tremendous noise of houses shattering and falling into the graben. They were joined by several other families and retreated to a paddock behind the house and Mr Emslie used his radio to let Civil Defence know where the group of 17 were stranded. After 2 hours a group of fireman made their way across the landslide and lead the group to safety with ropes strung between some remaining lampposts. The family joined with other residents at the Green Island Town Hall where the Red Cross has set up an evacuation centre.

The New Zealand Herald

ABBOTSFORD GOES DOWN

17 Caught As Chasm Grabs Homes

Press Assn Dunedin

The Dunedin suburb of Abbotsford was split apart last night.

Shortage Creates Babysit Teachers

Above: Herald front page 9 August 1970. Courtesy of The NZ Herald

"Slip Tears Abbotsford Apart" is the banner headline in the ODT the day after the disaster, along with "Chasm 75m wide opened in hours", "Thunderous roars echoed throughout area" and "Block of houses swept downhill". The reports describe a combination of good planning and good luck resulting in no major injuries or deaths, but by the end of the night landslide had destroyed 69 houses.

With the Prime Minister, Robert Muldoon on a visit to Africa, it was up to Acting-PM, Brian Talboys to monitor the aftermath. Offers of support came from around the country with several appeals launched. Auckland waterside workers donated 1-hours pay, and Radio Station 1ZM and Air New Zealand offered free flights to Auckland for, now homeless, Abbotsford children for the holidays.

The ODT headline for 10 August was "Abbotsford Slip now Stable" and the paper included a front page photo of Colin Emslie recounting his story. The Herald's lead story, running under the headline "Day of Anguish for Abbotsford" describes how 200 residents were briefly allowed home under Police escort to collect what they could carry in paper sacks. The paper reports army troops

Above: Houses in the landslide graben. Photo courtesy of The Otago Daily Times

had been sent to the area and the Minister of Civil Defence describes the landslide as New Zealand's worst natural disaster since the 1953 Tangiwai disaster, as well as praising Colin Emslie's efforts leading residents to safety. The papers include various commentators' opinions on what may have triggered the slide, some even theorizing a connection with nuclear testing at Mururoa atoll.

A week after the landslide, Abbotsford is not the main headline in the papers. The ODT and Herald both lead with the story of deer hunter surviving a helicopter crash that killed the pilot. Other front page news is the Prime Minister's and MP's proposed salary increases (to $52,000 and $21,000, respectively) and news of several yachts lost and crew dead in a violent storm during the Fastnet yacht race. The papers mention the government's announcement of a full investigation into the landslide and in a sideline story in the ODT the Ministry of Works postulates "water lubrication" rather than toe excavation may have been the trigger. An interesting story in the Herald discusses the legal situation now that some houses are on other peoples land. The article indicates home owners titles effectively extend from the ground surface to the centre of the earth, and are not altered when surface layers, including buildings, move. The report notes property boundaries will need to be re-surveyed after the current earthworks contouring are completed in order to re-establish property boundaries even though the topography of many owners land will be completely different.

For Nancy Hopper the experience was very difficult. "We had to move to a friend's house in St Kilda and felt like refugees". She recalls being financially worse off following the insurance settlement and thinks her husband who passed away 7 years ago "never really got over it". "I'll never forget it and even hearing the word Abbotsford today is very painful".

The Governor-General, Keith Holyoake, ordered the Commission of Inquiry 12 days after the disaster with terms of reference that included determining the cause, any human influences, the adequacy of the responses and any changes to legislation needed. The Commission sat for 58 days and heard 103 witnesses. The Inquiry report noted the landslide was a traumatic experience that caused a great deal of distress, but had a positive side, showing New Zealand communities had the ability and resources to cope. The Commission hoped "that the Abbotsford experience will lead to decisions being taken which will lessen the impact of such disasters when they arise in the future as they assuredly will".

Recollections from the Commission of Inquiry

Trevor Roberts was Executive Director of the Insurance Council of New Zealand at the time of the Abbotsford landslide, and was a member of the subsequent Commission of Inquiry into the disaster. Trevor is currently a consultant to the Wellington law firm Kiely Thompson Caisley, and is active on the boards of various organisations. Trevor discussed his recollections of Abbotsford with Geomechanics News editor Paul Salter.

The Disaster and the Appointment of the Commissioner

The actual landslip occurred on 8 August 1979. There had been indications for some weeks that something was afoot and the local authorities, Civil Defence, the Earthquake and War Damage Commission (as it then was) and others including a few insurers were involved. There were cracks in roads and in the foundations of some houses, notably a house at the bottom of the eventual landslide belonging to a Mr Patterson, and these seemed to parallel some difficulties that had been experienced on the other side of the valley in which Abbotsford was situated, though as it transpired that that was a completely different sort of landslide.

The situation got gradually worse and whilst it was not anticipated that a catastrophic event such as that which occurred was likely, the area was heavily monitored by Civil Defence, the Police and Fire Services and properties were evacuated. On the day of the disaster there was a sudden and spectacular landslip with very little warning, and photographs taken from the air some of which appear in the report of the Commission and which were widely distributed throughout the media and overseas indicated the spectacular nature of what happened.

Fortuitously, there were no fatalities but it was very evident indeed that the landslip scared the living daylights out of the people who were in the area at the time of the slip and when the photographs are examined and one of the classic photographs appears at page 136 of the report, it is easily understood why there were emotional reactions to what happened. There are a group of a dozen or so houses intact in the middle of the photograph. It is only when you match the roads serving these houses with the roads that terminate abruptly at the edge of the landslip area that you see just what the extent of the catastrophe was in real terms.

After the slip as the shock began to subside, the usual accompaniment to all such disasters manifested itself and public outcry began, and a search for blame started to emerge, led as might be expected, by the news media who as it emerged in the course of the inquiry were not in the slightest help to anybody.

The Government led this instance by the Minister of Justice, The Honourable Jim McLay acted commendably quickly and effectively. The first very bright thing that it did was to appoint Rodney Gallen QC (now Sir Rodney Gallen) and a highly regarded, respected and retired High Court Judge as Chairman of the Commission of Inquiry under the provisions of the Commissions of Inquiry Act. A very quick search was made for other Commission Members and George Beca, a leading Civil Engineer, Professor John McCraw, Professor of Soil Science at Waikato University and a much respected scientist, and the writer who is a Lawyer but was then Executive Director of the Insurance Council of New Zealand were quickly appointed as the other members of the Commission.

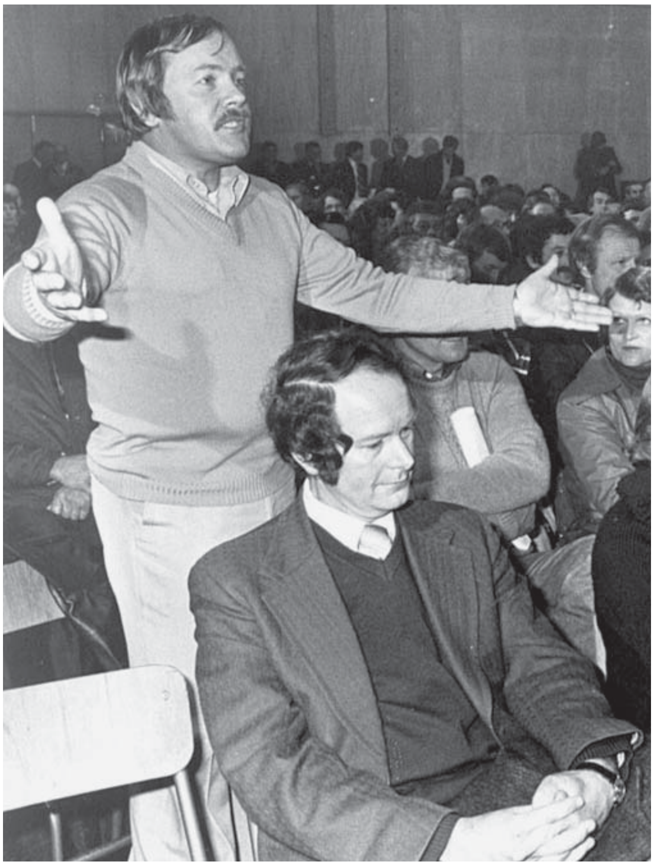
Unusually, all of the Commission Members were consulted with regard to the terms of reference and Jim McLay made it clear that he wanted a full and comprehensive inquiry with no holds barred covering every aspect of the landslip dealing with the cause, the effectiveness of the legislation that was relevant to the event, and the performance of all of the people and the institutions involved. Accordingly, the Commission had very wide terms of reference.

Sir Rodney Gallen's leadership and the way in which he set the tone of the inquiry were evident from day one. The Commission agreed that it had to move as quickly as possible, but that it also needed to leave no stone unturned and address itself all of the issues. In particular the residents and those who had been subject to criticism and in some instances quite unreasonable attack had to have the opportunity to be heard and the rules of natural justice had to be observed without however descent into an unreasonably legalistic and bureaucratic approach.

Ground rules were set which were designed to make life as easy for everybody involved as is possible. The Commission was provided with generous resources to fulfil its responsibilities. John Wild now a High Court Judge was appointed Counsel to assist the Commission and Mr Jim D'Ath provided administrative support in a very effective fashion and ensured that we had very good secretarial and logistical support.

We all became firm friends from day one, and those friendships remained for many years. We sat in Auckland, Wellington and of course primarily in Dunedin where we spent about eighteen full weeks hearing evidence and deliberating, with 58 sitting days. It wasn't always easy. My own children referred to me as "the Abbotsford Disaster".

I had the support of the Insurance Council, who were extraordinarily generous in releasing me. I spent eighteen weeks away from my family with the exception of the weekends and my normal work was heavily interrupted.



Above: Residents seek answers, courtesy of Otago Daily Times

The matter was compounded and my children's impression that I was the Abbotsford Disaster was reinforced when in the middle of it all the Taieri Plains, which included the Dunedin Airport, suffered the heaviest flood in its history, so our too-ing and fro-ing to Dunedin for some months consisted of an airline trip to either Invercargill or Timaru and car or bus journeys from there. A very interesting early helicopter flight over the Abbotsford Disaster area, which was later repeated courtesy of Civil Defence, was followed by another trip over the flooded Taieri Plains and I was able to report to the General Manager for New Zealand of one of the Insurance Council Member insurance companies that I had identified his car at the Dunedin Airport, sitting in the carpark with water up to the top of the windows and a very large goose sitting on the roof.

The Hearing

It was very evident from day one that there was a great deal of emotion surrounding the incident and not a small amount of anger. The Abbotsford residents were not at all happy and one could well understand that emotion. The problem was compounded by the fact that there was at least the possibility that some of the organisations involved before, during and after the disaster had performed badly and were in some degree culpable.

Allegations were certainly made. The Ministry of Works was in the firing line. It had designed the Green Island Motorway and there was a strong suggestion that its engineering research was inadequate and that it unloaded the toe of the slip and the work in connection with what was known as Harrison's Pit had compounded the difficulty. The Dunedin City Council were in the firing line. Their water supply ran across the top of the slip area and there were strong allegations of leakage that it poured water into the tip which had been the cause of the slip. The Borough Council were under criticism for not having done enough to warn and protect the residents and for approving subdivisions which should never have been approved in view of the geological nature of the area and Civil Defence and the Police and Fire Service were accused by some residents and others of incompetence and ultimately heavy handedness. Gradually the criticism subsided and the general air of grumpiness, to put the matter mildly at least initially, was ameliorated by a very real desire on everybody's part to get to the bottom of the matter.

Results of the Inquiry

It is instructive to read the summary of conclusions that appear at the end of the Commission's report. With regret the Commission had to ask for extensions of time. We were absolutely determined to do the job properly and in particular that we would make considerable efforts to address any gaps in the legislation. It was necessary to look at the relevant legislation:

- The Civil Defence Act
- The Earthquake and War Damage Act and Regulations
- Local Government legislation
- Legislation regarding the management of water courses

In the end, we waded through the legislation and with all of the Commissioners making a contribution, we produced a report of 196 pages which was presented to the Governor General in November 1980.

I think there was a nice little piece of history in the actual production of the report. 1980 was before the days of Microsoft, Word and the ability to produce and edit lengthy reports adequately with computer software. Besides writing sections of the report, I had the responsibility of collating the report and drawing the various bits of it together before it was very capably and finely edited by the Chairman. I think we would still be labouring our way through the review of the enormous pile of evidence and collating and matching various pieces of input were it not for a fortunate occurrence which had a nice New Zealand feel about it. One of my Insurance Council Members, Norwich Union Fire Insurance Company had just installed the best and most modern word processing system in Wellington, and through the generosity of its then General Manager who was also President of the Insurance Council and my employer, though he hadn't seen much of me for months, I was able to use his secretary and his then state of the art word processor to produce the initial drafts and not only that, it didn't cost the New Zealand Government and the taxpayer a cent. I think this is probably the first public acknowledgement of a very generous act of citizenship pretty typical the sort of things I saw much of the time around the insurance industry in my very happy 11 years as Executive Director of the Insurance Council. Finally, after a little over a year we completed the report presented it to Sir David Beattie and sat back to wait for the result.

The reaction was actually surprisingly muted, and over a period of years, many of the things that we identified and criticised were quietly dealt with.

- There were extensive amendments to what was then The Earthquake and War Damage Act and the first steps in the modernisation of the Commission were undertaken leading to the very effective and world leading organisation that it now is. Another of the little ironies, was that I was subsequently appointed to the Commission and served as its Deputy Chairman for 9 years and was able to play some part in the modernisation of the organisation. In particular, it now administers one of the largest catastrophe reinsurance programmes in the world, its reserves have dramatically increased. It has been able to a greater extent to match its investment profile with the risks that it insures and it has made considerable contributions to earthquake and geological research in New Zealand and into research and development of building standards. The nature of the cover it provides has changed in part to meet the recommendations of the Commission of Inquiry and in particular there is now cover for loss of use of land in a landslip or similar disaster. The Commission of Inquiry was I believe a useful catalyst to change.
- Civil Defence and local authorities were mostly but not entirely exonerated and in some cases deservedly congratulated. I think the Commission made a contribution towards improvement of Civil Defence, but we still have a distance to go in the hope that it will not take another disaster and another inquiry to concentrate with the public mind and produce some of the reforms and legislation that are still needed.

I have never been entirely convinced that we got the "blame" issue relating to the cause of the disaster entirely right but we simply did not have enough evidence and research available to reach any firm conclusion. One of the main issues for the Commission was the significance of a series of reports and papers produced by Professor Benson of Otago University covering the geology that was relevant at Abbotsford, and we struggled with the significance and application of those reports, and the availability of a wide range of scientific and engineering evidence. In particular, the Ministry of Works was in the firing line in respect of motorway construction and its work around Harrisons Pit. The question was, did it investigate far enough and even if it had would Professor Benson's work have served as a warning. Did it proceed carefully, or did it, in the words of one of its own engineers, just "have a go".

We came to the conclusion that we didn't know enough to point a finger. On the face of it, what the Ministry of Works did or didn't do was not unreasonable, but I still have a slightly uncomfortable feeling which I think may have been shared by other members of the Commission. Unfortunately what we knew or guessed or were told didn't enable us to reach even the civil standard of proof of culpability "upon the balance of probabilities". I think by and large we did pretty well. What we did do made a difference.

Other Lessons to be Learned

With the benefit of 30 years hindsight I think that I can draw other useful conclusions.

- Commissions of Inquiry under the Commissions of Inquiry Act can be an extremely useful tool in appropriate circumstances, but it is absolutely essential that the terms of reference of the Commission be carefully constructed to deal with the actual issues and not in any way influenced by collateral issues such as being a mechanism to avoid criticism of institutions or persons. We were lucky in the brief that the Government and in particular Jim McLay gave us.
- Careful choice of commissioners but particularly the Chairperson is crucial to success. We were lucky and in particular the wisdom and humanity of our Chairman made all the difference.
- I use that word humanity deliberately. Rod Gallen was a very caring and careful Chairman and took very careful steps to deal with some of the anger that surrounded the incident as it always does in any national disaster, and to ensure that everybody who wanted to be heard was properly decently and compassionately heard, and that the Rules of Natural Justice and the necessity for fairness was uppermost in everybody's mind.
- We won't ever be entirely free of the possibility of natural disaster. When lives are lost anger and emotions rise in a predictable fashion. There is some useful work done by for instance the University

of Queensland in this area and any organisation dealing with natural disaster needs to understand that the reaction to natural disaster particularly when lives are lost (fortunately in the case of Abbotsford they weren't) will be strong and in some respects predictably irrational.

- I have to say that the news media doesn't always fulfil its claimed function as the guardian of transparency and the public interest. The media, as I recall, was highly critical of lack of response particularly on the part of Civil Defence to requests for information which at times verged on the irresponsible and certainly were intrusive unhelpful and in some cases circumstantially unable to be met. I might say that I personally suffered similar criticism in latter years when as Executive Director of the Insurance Council I had to deal with news media enquiries arising from major floods. The Commission gently pointed out that Civil Defence had found it difficult in Otago and Southland to recruit on a voluntary basis a news media person who could act as an information officer in the event of a natural disaster. Apparently, members of the news media do not volunteer as citizens to make their knowledge and expertise available. In the circumstances of natural disaster, the general public interest makes access to those skills useful and necessary. I note with interest that the Australian news media is currently coming under similar criticism in respect of a lack of professional ethics and responsibility in dealing with bush fire events and their aftermath. We saw the predictable reaction that "they" should fix it or "they" didn't help us, or "they" were to blame. That is entirely to be expected as part of the human condition, but it has to be said that in circumstances where there is a considerable degree of public pain and anguish and a resulting search for "they", often accompanied by a degree of crass media conduct, doesn't contribute much to the public good. Again with 30 years hindsight I think it might have been useful if our reference to the media and the terms of our criticism of it had been a little less gentle.

Hindsight

Finally, by way of general comment, I think we gained a useful insight into the way communities work and to the good as well as the bad. We have made considerable strides in planning disaster litigation and disaster management over the last 30 years, but we still have a way to go. The Commission was privileged to share with the Dunedin community its pain and its anguish and there weren't too many light moments. One however did stand out.

The technical nature of the various sorts of landslips was a key element in endeavouring to identify whether one or more of the public bodies involved had a degree of culpability. The Dunedin City Council was in the firing line because of the allegations of poor maintenance of its Silverstream pipeline. The Ministry of Works was in the firing line because of its failure or alleged failure to adequately identify the geological fraility of the Abbotsford area. There was much discussion as to whether the landslip was a creep, slump, a bedding plane slide, a progressive failure or one of a number of other geological phenomena. Senior Dunedin Counsel, was cross examining a Ministry of Works Engineer and endeavouring to obtain his opinion as to the type of landslip involved. His method of cross examination was largely to examine crossly. Finally, he fixed the hapless engineer with a very beady eye and asked in somewhat aggressive and sarcastic tones – "and pray what is the difference between a creep and a progressive failure." A little voice from the back of the courtroom piped up – "University Entrance". The Commission found it necessary to adjourn hurriedly for afternoon tea.

Report by:
Trevor Roberts
November 2009

MEMBER'S PAST CONTRIBUTIONS

Reprinted from NZ Geomechanics News – Issue 22, June 1981

2.2 The East Abbotsford Landslide

At a very successful joint meeting of the Wellington NZIE Branch, the Geological Society, the Geology Section of the Royal Society, and the Geomechanics Society was held on Thursday, 4 June. The technical aspects of the East Abbotsford landslide were discussed. Over 70 people attended this meeting, the subject of which was recently addressed by a Commission of Inquiry.

Guest speakers at the meeting were Graham Hancox from the New Zealand Geological Survey and Graham Ramsay from the Ministry of Works and Development. The address was divided into two sections: Graham Hancox described the development of the slide, its physical extent and geology; and Graham Ramsay discussed the stability of the slide and its possible causes.

INTRODUCTION

The East Abbotsford landslide occurred on 8 August 1979 in the residential area of Abbotsford, Dunedin. It was a translational block slide covering some 18 hectares and involving approximately 5 million cubic metres of material which moved about 50 metres. There had been evidence of the development of the slide for several months prior to the final movement. No lives were lost in the slide but 69 houses were either destroyed or rendered inaccessible.

GEOLOGY

In the Abbotsford area, very weak Tertiary sediments dip 8^{o} south east towards the Kaikorai Stream. Over the past several thousand years natural processes of erosion in Miller Creek and the Kaikorai Stream have removed material down dip and formed the slope on which the landslide is situated.

The stratigraphic sequence basically consists of a minor surficial loess deposit which rests on bouldery colluvium, a solifluction sheet some 12 metres thick in some places. This in turn rests on the Green Island Sand, which is a fine sand with minor clay content, and this sand is laid conformably on the Abbotsford Formation. The top 30 m of the Abbotsford Formation at East Abbotsford consists of grey sand with thin clay layers, below which is grey mudstone. Sand in the Abbotsford Formation is of similar grading to the Green Island Sand. There is a sharp colour change between the light yellow coloured Green Island Sand and the dark grey coloured Abbotsford Formation sand.

POST SLIDE INVESTIGATIONS

Initial post slide investigations, using small diameter drill holes, did not reveal any materials which were sufficiently weak to explain the slope failure. Typically ground water levels were found to be some 10-20 metres below the ground surface. For failure to occur on a slope of 8^{o} with this order of ground water it was considered that some very weak material must be present in the slope.

With this objective 1 metre diameter shafts were sunk to enable a more thorough examination of the stratigraphic sequence. This was not an easy operation and many problems were encountered, particularly in penetrating the layer of boulder colluvium. The effort was rewarding as several thin layers of clay were found in the upper region of the Abbotsford Formation and typically within 1 metre of the upper contact with the Green Island Sand. These clay layers varied in thickness up to 50 millimetres and most significantly they contained slickensided surfaces indicative of movement.

Samples of the clay layers were recovered and subjected to laboratory tests. These tests indicated residual strength parameters of the order $c' = 0\text{–}20$ kPa and $\phi' = 5^{\circ} - 10^{\circ}$ which are amongst the lowest reported in the literature.

STABILITY AND CAUSES

A number of possible natural and man-induced causes of the slide were investigated including the effects of rainfall, changed land use, a leaking water main up slope, the excavation of a sand quarry at the toe, and seismic activity. It was found that any one of the above appeared to result in only a very small change in stability of the order of 0-2 per cent. Depending on the input assumptions the effect in some cases could be an improvement or reduction in stability.

Although it is open to debate Dr Ramsay said that for the purposes of analysis it had been assumed that one or more of the clay layers was continuous. This was considered reasonable because of the continuous nature of the Green Island and Abbotsford Formations. Back analysis of the slide has shown that clay strength parameters of the order $c' = 0$, $\phi' = 8^{\circ}$ would produce failure.

Dr Ramsay said he considers that the basic cause of the landslide was geological and arose because of the existence of a thin weak montmorillonite rich clay layer within the top few metres of the Abbotsford Formation. The ancient Sun Club Slide which forms part of the Abbotsford slide mass, is evidence of the marginal stability of the slope to the west of Miller Creek.

COMMENT

The spontaneous discussion which followed the speakers' presentation was an indication of the level of interest which this significant event has generated in the geotechnical world. It is to be hoped that the various people involved with the technical aspects of the slide will make a contribution to the literature in the near future. In the meantime for those seeking more information, the Commission of Inquiry's report is very good reading.

D.N. Jennings

BOOK REVIEWS

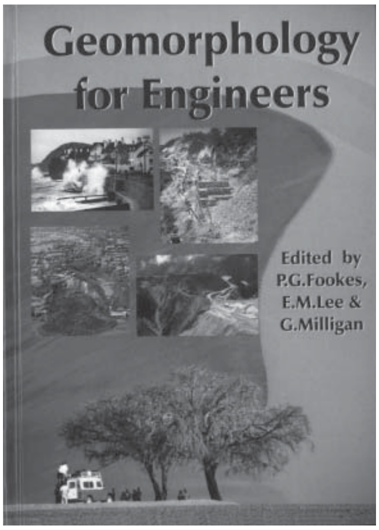
Geomorphology for Engineers – Edited by P.G Fookes, E.M. Lee and G. Milligan

As a practicing geotechnical engineer who occasionally gets let outside, a decent chunk of my time is spent considering how the features of a site came to be, what the causes of them were and whether the events that occurred in the past are going to happen again in the foreseeable future. Understanding what's going on is a fundamental part of providing a safe and cost effective solution. We typically need geological input and guideance to assist in providing sensible answers. Some of our greatest cock ups are based on an omission to consider or misinterpretation of the geomorphology.

Put simply, geomorphology is the study of why land forms look that way they do and how they change. This is typically something geologists know quite a bit about and geotechnical engineers need to pick up as they go. My recollection of geology from university comprised trying to identify Basalt for Andesite based on little samples in little boxes, not the most inspiring introduction. Nothing relating to geomorphology seemed to stick around long enough to be useful (maybe it was me). This book has provided an opportunity to catch up in this area.

Geomorphology for Engineers is set out in three parts. Part one describes the underlying controls that influence geomorphology, including weathering, sediment deposition, tectonic activity and highlights of the most recent two million years or so that have strongly influenced what we see around us today. Part two covers geomorphological processes, including landslides, erosion, earthquakes and rivers. Part three discusses the characteristics of a number of distinct landform types around the world from glaciers to deserts, karstic terrain to urban areas.

I've found Geomorphology for Engineers to be a great reference that I have happily sat down to read, jumping around the chapters to suit what grabs my interest. Parts one and two in particular give a good basic understanding about what is going on and to assist in effectively communicating with geologists (I've noted on occasion that we don't always see things the same way). Part three covers specific landforms, some of which are either absent or not common in New Zealand. I can't say that I've read all about savannahs just yet, but am reassured that when the dream job on the plains of Africa comes up I've got a reference there to get up to speed quickly. After completing a few projects involving deep peat deposits, I found the Holme Post to be a good story about the magnitude of settlements in such areas.



Consistent with its title, Geomorphology for Engineers is orientated towards the engineer and may therefore lack some of the more advanced geologically focused information or techniques that you would find in more specific texts, say the latest landslide hazard assessment methodology for instance. However, given that most engineers formal training in geomorphology is limited or non-existent, this book offers a great leap forward in our understanding of this area of high importance to a geotechnical engineer.

I happily recommend this book to all practicing geotechnical professionals, whether coming from a geological or engineering background, though as noted above I suspect it provides more of an opportunity to step up for the engineer. At $275, its not cheap but well worth the investment. Better yet, get work to buy it for you...

Reviewed by:
Nathan McKenzie
nathan.mckenzie@beca.com

Author	Edited by P.G Fookes, E.M. Lee and G. Milligan
Publisher	Whittles Publishing
Year Published	2005 (reprinted 2009)
Hardback	874 pp
ISBN	1-870325-03-6
Web shopping	http://moo.whittlespublishing.com/whittles/item/2533
Price	GBP £100

Fundamentals of Soil Mechanics for Sedimentary and Residual Soils
– Laurence D Wesley

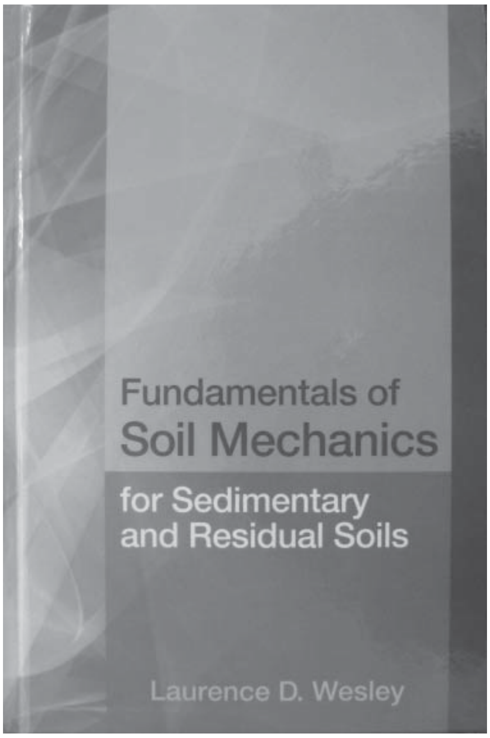
The publication of this new text is so recent that the copyright date is 2010.

One might observe that the market for soil mechanics text books is well served; it would be easy to name more than half a dozen texts in print. However, this book is unique in having an emphasis on the properties of both residual and sedimentary soils. Thus it should have a good market in NZ and also in the many other places around the world where residual soils are a significant feature of the landscape. It is this emphasis that sets Laurie's book apart from the many other "standard" texts which emphasise the properties of soils of sedimentary origin but offer negligible or no insight into residual soils. The big difference is that the formation of sedimentary soils is dominated by stress history whilst stress history has little to do with the formation of residual soils. The problem arises when the terminology for sedimentary soils, such as preconsolidation pressure, is carried over to residual soil. I can say from personal experience that it is not easy to expunge this jargon from one's technical subconscious when considering residual soils!

On the basis of my experience, I welcome this book as it is directed at introducing the distinction between sedimentary and residual soils to students right at the start of their journey along the soil mechanics path.

The book is a good size both in terms of the number of pages and physical dimensions – not the cumbersome size of some other Wiley soil mechanics publications. The accompanying photograph shows that the designer came up with a pleasing cover. Opening the volume reveals pages with an attractive layout adorned with Laurie's superb diagrams. So first impressions are of an inviting text.

Looking through the table of contents one sees that there is an orderly sequence of chapter titles such as one would expect – basic material about the particulate nature of soil, stresses in the ground, effective stresses, permeability and seepage, compressibility, shear strength, site investigation. It then goes onto applications – bearing strength, slope stability, earth pressures, and compaction. Interestingly there is no chapter devoted specifically to residual soils. Reference to the index shows that there are about the same number of page entries under sedimentary soil and residual soil. The reason for no special chapter is explained in the preface which emphasizes that nearly all the concepts of soil behaviour are common to sedimentary and residual soils – the subtleties about origins are most clearly apparent when it comes to compressibility. Incidentally the second paragraph of the preface is nicely provocative and aimed at teachers of soil mechanics the world over.



Many of the readers of this review will have been beneficiaries of Laurie's teaching whilst the University of Auckland was so fortunate to have him as a staff member; you will know what to expect in the way of clear explanations so I don't need to recommend the book to you. For the rest, I can recommend this as an essential volume not on your book shelf but right by the computer keyboard on your desk.

Reviewed by:
Mick Pender
m.pender@auckland.ac.nz

Author	Laurence D Wesley
Publisher	Wiley
Year Published	2009
Hardback	464 pp
ISBN	978-0-470-37626-3
Web shopping	http://au.wiley.com/WileyCDA/WileyTitle/productCd-0470376260,descCd-description.htm
Price	NZD $220

Life on the Edge – New Zealand's Natural Hazards and Disasters

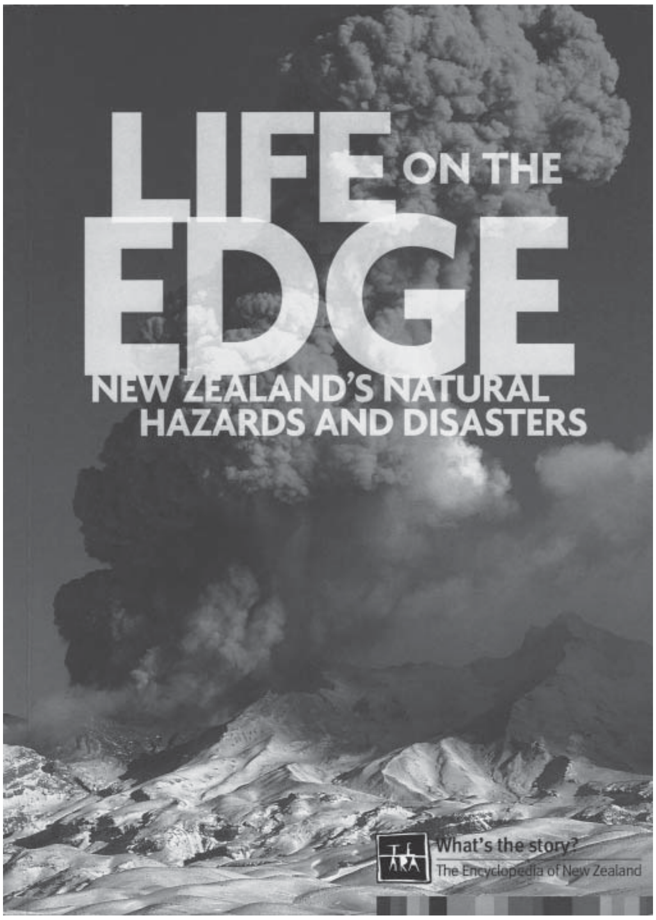
Life on the Edge is packed full of so many good graphics, quotes and photographs of our amazing country and its people – a somewhat 'dangerous' New Zealand. Who would live here? The awe of the natural hazards themselves, the fault lines, the cracks, the slips, the volcanoes, earthquakes and floods are in places overwhelmed by the personal accounts of the residents, eye witnesses and professionals whose lives are constantly being challenged by our geology and geography. Each time I randomly explore the book (it would read well cover to cover too) more is revealed. Historical facts sit alongside anecdotes; it feels like a well researched and beautifully presented history lesson. Those two plates crashing together have certainly had a lot of fun – yes, the Pacific and Australian ones.

The chapters are distinct, with nifty titles. For example, Landslides: Gravity Always Wins – designed to stick in your mind or incite fear? There is plenty of science here – detailed descriptions, and of course many famous natural disasters take centre stage (Napier, Cyclone Bola, Tarawera, Tangiwai) but the book also moves past the devastation caused. For example, in the chapter on earthquakes there is an explanation on how we can now monitor (Geonet) and make predictions.

The layout is superb. There is also a detailed index (Abbotsford has three entries, landslides - 29) and a chapter on preparedness completes the picture (a photo of a relief camp in Hastings following the 1931 Hawke's Bay earthquake is fascinating).

The book is meant to be a resource – it has been extracted from the online Encyclopedia of New Zealand, a 10 year project being prepared by the "Te Ara" team at the Ministry for Culture and Heritage. Te Ara means 'the pathway', and this book offers an easily digestible pathway to understanding New Zealand's natural hazards and disasters. I would urge anyone with children to explore additional material at www.TeAra.govt.nz.

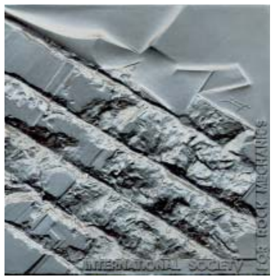
Reviewed by:
Amanda Blakey
nzgeotechnicalsociety@xtra.co.nz



Author	Te Ara
Publisher	David Bateman Ltd
Year Published	2007
Paperback	160 pp
ISBN	1869536908
Web shopping	http://www.fishpond.co.nz/Books/Childrens/Fiction/Reference/General/9781869536909/ ?cf=3&rid=1042024473&i=20&keywords=Life%20on%20the%20Edge
Price	NZD $39

International Society for Rock Mechanics

ROCHA MEDAL 2011



Since 1982 a bronze medal and a cash prize have been awarded annually by the ISRM for an outstanding doctoral thesis in rock mechanics or rock engineering, to honour the memory of Past President Manuel Rocha while stimulating young researchers.

Starting with the Rocha Medal 2010, one or 2 runner-up certificates may also be awarded.

An invitation is now extended to the rock mechanics community, and especially to Faculty members, for nominations for the Rocha Medal 2011.

Full details on the Rocha Medal are provided in ISRM By-law No. 7.

Application

To be considered for an award the candidate must be nominated within two years of the date of the official doctorate degree certification.

Nominations shall be by the nominee, or by the nominee's National Group, or by some other person or organization acquainted with the nominee's work.

Nominations shall be addressed to the Secretary General and shall contain:

- a one page curriculum vitae;
- a written confirmation by the candidate's National Group that he/she is a member of the ISRM;
- a thesis summary in paper and digital formats, written in English, with between 5,000 and 10,000 words, detailed enough to convey the full impact of the thesis and accompanied by selected tables and figures;
- one copy of the complete thesis and one copy of the doctorate degree certificate;
- a letter of copyright release, allowing the ISRM to copy the thesis for purposes of review and selection only;
- an undertaking by the nominee to submit an article describing the work, for publication in the ISRM News Journal.

Application Deadline

The nomination must reach the ISRM Secretary General by 31 December 2009.

Past Recipients

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1983	S. Bandis	GREECE
1984	B. Amadei	FRANCE
1985	P.M. Dight	AUSTRALIA
1986	W. Purrer	AUSTRIA
1987	D. Elsworth	UK
1988	S. Gentier	FRANCE
1989	B. Fröhlich	GERMANY
1990	R.K. Brummer	SOUTH AFRICA
1991	T.H. Kleine	AUSTRALIA
1992	A. Ghosh	INDIA
1993	O. Reyes W.	PHILIPPINES
1994	S. Akutagawa	JAPAN
1995	C. Derek Martin	CANADA
1996	M.P. Board	USA
1997	M. Brudy	GERMANY
1998	F. Mac Gregor	AUSTRALIA
1999	A. Daehnke	SOUTH AFRICA
2000	P. Cosenza	FRANCE
2001	D.F. Malan	SOUTH AFRICA
2002	M.S. Diederichs	CANADA
2003	L. M. Andersen	SOUTH AFRICA
2004	G. Grasselli	ITALY
2005	M. Hildyard	UK
2006	D. Ask	SWEDEN
2007	H. Yasuhara	JAPAN
2008	Z.Z. Liang	CHINA
2009	G. Li	CHINA
2010	J.C. Andersson	SWEDEN

All relevant information can be obtained from the ISRM website, at http://www.isrm.net.

STANDARDS, LAW AND INDUSTRY NEWS

Review of NZ Standards 4404 and 3604

NZGS has provided comments on two New Zealand Standards currently under review. Management Committee member Simon Woodward has coordinated NZGS's input on proposed changes to NZS4404:2004 the Standard for Land Development and Subdivision Engineering and Section 3 of NZS3604:1999 the Standard for Timber-framed Buildings.

The reviews are being undertaken by Standards New Zealand – the public face of the Standards Council, an autonomous Crown entity comprising an appointed body and various sector representatives, which is self-funded to promote independence.

The Standards are agreed specifications developed by expert committees using a consensus-based approach and public input. They aim to improve safety and quality and meet industry best practice. The Standards covering natural disasters e.g. earthquakes aim to minimize the impact of such events and typically require a level of detail sufficient to support legislation.

Many consultants will be familiar with NZS4404 – the Standard for Land Development and Subdivision Engineering – which covers stability, earthworks, roads, stormwater, wastewater and other aspects of developing land under the RMA. Council's often invoke this Standard, in part or in full, in their District Plans. The current review is not a complete revision but seeks to encourage "sustainable and modern design that emphasises liveability and environmental quality in the development and subdivision of land". The intent seems to be for a Standard Council's can apply that will allow for more innovative and low-impact solutions rather than a prescriptive set of rules that can discourage alternative design. A good summary of the background to this revision can be found in an article by the Local Government New Zealand (LGNZ) representative on the Standards Committee, at www.ingenium.org.nz/pages/36/publications.htm.

The ACENZ representative on the NZS4404 review committee sought NZGS's comment on improvements to this Standard. Following Management Committee input, an NZGS response, focusing primarily on the sections relating to Land Stability, Foundations and Earthworks, and Road design and CBR testing was compiled by Simon Woodward and forwarded to the review committee for their consideration. The new draft Standard has now been released for public comment and is available (at www.standards.co.nz) for comparison with the current Standard. The closing date for submissions on this draft is 5 February 2010.

Section 3 of NZS3604 relates to minimum Site Requirements for Timber-framed Buildings and includes the definitions surrounding "good ground", which define whether foundation provisions in the Standard can be applied or specific engineering design is needed. The Standard covers such topics as soil type, tests for bearing capacity, site preparation, etc.

As for NZS4404, a limited technical review of this Standard is currently underway. The Department of Building and Housing and the Earthquake Commission are sponsoring the revision and a draft Standard is due for release in late 2009 for public comment (still pending as this edition goes to print), with the goal of implementing the updated Standard by late 2010 or early 2011.

The revision is being lead by a leadership group, but the task of producing the draft Standard is being undertaken by a technical committee with 22 industry representatives, including two IPENZ representatives, which has meet 6 times this year. The IPENZ members sought NZGS comment on possible revisions to Section 3 and Simon Woodward and Philip Robins canvassed the Management Committee and industry colleagues for comments which were incorporated into a detailed response to the review committee. We are yet to see how these suggestions have been incorporated in a revised draft Standard.

Progress on the revision of NZS3604 can be followed on the Standards NZ web-site (www.standards.co.nz).

As mentioned in Philip Robins Chairman's report, a subcommittee of the NZGS has also completed the "NZ Geotechnical Seismic Design Guidelines" this year. The background on these design guidelines, including where they sit in relation to Standards such as NZS1170.5, can be found on the NZGS web-site under Pending Publications.

Reported by:
Paul Salter
Co-editor: paul_salter@urscorp.com

IAEG2010 Congress Report

We have now received and reviewed over 900 abstracts submitted to the 11th Congress of the International Association for Engineering Geology and the Environment (IAEG), to be held in Auckland in September, 2010. IAEG2010 is hosted and underwritten by the New Zealand Geotechnical Society Inc. (NZGS) and will be a five-day congress including a day of field trips mid-week, addressing the range of geotechnical issues. The Congress will be preceded by a full day of activities aimed at Young Professionals, including a mentoring breakfast hosted by Jamie Fitzgerald, competition for the Richard Wolter's Prize and match racing in America's Cup yachts.

Key-note speakers include international experts in the fields of liquefaction, disaster risk management, landslides and tunnelling, and mining engineering.

The themes of IAEG2010 extend beyond engineering geology and the congress aims, at least in part, to bridge the gap between science and practise.

Check out our registration brochure on-line www.iaeg2010.com and register now!

Sponsorship and Exhibition

IAEG2010 gratefully acknowledges the support of the following organisations:

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If you are interested in receiving a sponsorship brochure or investigating sponsorship opportunities, please contact Clare Wilton of The Conference Company,+64 9 623 8088 or clw@tcc.co.nz. We look forward to having you join our team!

Ann Williams
Co-convenor IAEG2010

Uncertain Geotechnical Truth and Cost Effective High Rise Foundation Design – 45th Terzaghi Lecture, ASCE, 2009. By Clyde N. Baker, Jr., P.E., S.E

The Auckland NZGS branch were privileged to host Clyde Baker deliver the 45th Terzaghi Lecture on 2 November 2009 at Auckland University. A brief biography follows this summary. Mr. Baker's lecture was delivered to share experiences gained during his 57 year career (so far!), focussing on the development of cost effective foundations for high rise buildings, bearing in mind the risks and use of judgement. The speaker developed his career in Chicago, working with Peck, Casagrande and a host of other well known geotechnical names. A brief summary of the geology of the Chicago area was followed by discussion of the major high rise developments including the Audotorium, which is still in use despite having settled 400mm in 80 years. Old load test information was presented and the predominant construction methodology of belled caissons was discussed. Various design methods for the Menard Pressuremeter were presented and discussed, including the reliability of the predictions.

Mr. Baker then moved on to explore the use of judgement and experience to develop more cost effective foundations. The conclusions essentially comprised (a) know your local geology and building history, and (b) we may not know as much as we think we do. The importance of geotechnical and structural engineers working as a team was emphasised, with the examples of the Burj Dubai and Petronas Towers illustrating the point. Mr. Baker emphasised the importance of having a fallback position when pushing the design envelopes. The lecture concluded with a review of case histories and a revisit to the Petronas Towers design. The key point to come out of the examples was that very large skyscrapers were designed with settlement mitigation in mind rather than structural capacity of the piled foundations. Piles supporting the Petronas Towers, for instance, would be around 30m long for capacity but were installed up to 150m long for settlement mitigation.

The talk was a fascinating insight into the design of foundations for large structures from someone who has been involved for over 50 years. The wealth of experience and enthusiasm shone through and ensured that the lecture was well worth attending.

Above: Clyde Baker

Short biography

Mr. Baker has developed an international reputation in the design and construction of deep foundations. He has been a leader in using in-situ testing techniques correlated with past building performance to develop more efficient foundation designs. Over the past 55 years Mr Baker has served as the geotechnical engineer on the major portion of high rise construction built in Chicago during that time frame. He has also served as geotechnical engineer or consultant on eight of the twenty tallest buildings in the world including the four tallest in Chicago (Sears, Trump, Hancock, and Amoco) and the current four tallest buildings in the world, the Petronas Towers in Kuala Lumpur, 101 Financial Center inTaipei, and Burj Dubai.

He is currently working as a consultant on several super tall buildings currently under construction including the Spire in Chicago, Doha Convention Center and Tower in Qatar and Incheon 151 in Incheon, Korea. Mr. Baker is a past Chairman of STS Consultants Ltd, which is now part of AECOM and currently serves as Senior Principal Engineer.

He is an Honorary Member of ASCE, a past President of SEAOI and the Chicago Chapter of ISPE, past Chairman of the Geotechnical Engineering Division of ASCE, past Editor of the Geotechnical Engineering Journal, past Chairman of ACI Committee 336 on Footings, Mats and Drilled Piers. He is a member of the National Academy of Engineering. He is the recipient of many awards including the Deep Foundation's Institute Distinguished Service Award, the ADSC Outstanding Service Award, ASCE's Thomas A. Middlebrooks and Martin S. Kapp awards, the ASCE Ralph B. Peck Award, the 2007 Engineering News Record Award of Excellence, the ASCE Opal Lifetime Achievement Design Award 2008, and the 2009 Washington Award.

Reported by:
Pierre Malan

NZGS Young Geotechnical Professionals

The last six months have been very busy for the YGP team with the promotion and organisation of the Student Award presentations for 2009 (discussed further on page 73) and finalising of the Student Award for 2008.

NZGS have awarded the 2008 Student Awards to the following:

Southern Region: "Assessment of the performance of pile foundations in liquefying soils" by Jennifer Haskell

Northern Region (jointly awarded):

1. "Variations in index and shear strength parameters of sensitive soils in the Tauranga Harbour area" by James Arthurs

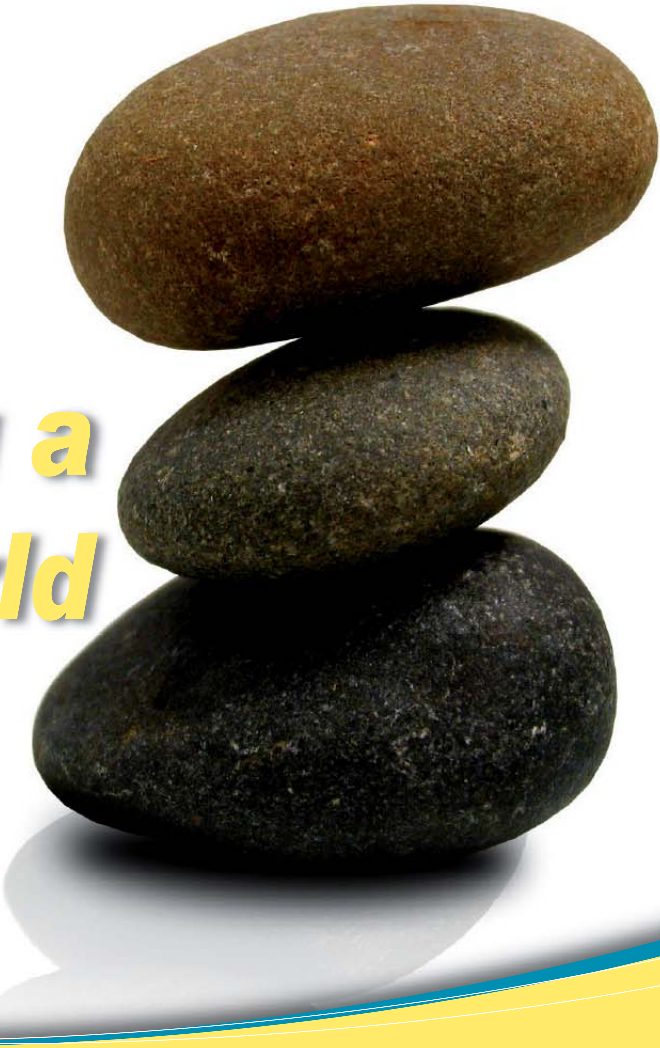
2. "The Sensitivity of Some Weathered Pyroclastic Material in the Tauranga Region" by Justin Wyatt

The next six to eight months will be another busy period for the YGP team with finalising details and arrangements for the IAEG 2010 Young Professionals component of the Congress. We have a full day planned for 5th September starting out with a mentoring breakfast, the Richard Wolters Prize presentations followed by Americas Cup Style Match Racing on the Waitemata Harbour.

This will be the premier event for NZGS next year including the YGP's so come along yourself and /or support one of your fellow budding engineering geologists.

We welcome support and inspiration from the young geotechnical professional's community. If you have any ideas or activities you would like to see happen please contact me.

Reported by:
Kate Williams
YGP Representative
Email: kwilliams@tonkin.co.nz



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CONFERENCE REPORT

The Fourth International Young Geotechnical Engineers Conference

The Fourth International Young Geotechnical Engineers' Conference (4iYGEC) was held on the 3rd to 6th October 2009 in Alexandra, Egypt. Hayden Bowen and Lucy Coe were given the fantastic opportunity with support from NZGS to attend this conference held on the coast of the Mediterranean Sea, and present their papers written for the Australian and New Zealand Young Geotechnical Professionals Conferences (2006 and 2008).

In the words of Hayden "the conference was the United Nations of geotechnical engineering". There were approximately 80 delegates representing 45 different countries, spread over 6 continents. Both research based and practitioners were represented. The conference gave participants the opportunity to share their ideas and experiences in geotechnical engineering.

Professor Ahmed W Elgamal of the University of San Diego (USA) presented a keynote lecture on "Scenario-focused 3-D computational modelling in geomechnics". The themes of the conference focused on:

- Soil Behaviour and Properties, New Concepts and Correlations
- Ground Improvement: Chemical Mechanical and Reinforcement
- Seepage Flow, Contaminated Soil Treatment and Response
- Landslide and Slope Stability, Case Studies
- Deep Foundation Design and Practice
- Performance of Different Types of Earth Retaining Structures
- Soil Structure Interaction, Risk Management
- Underground Construction

There was a particular emphasis on ground improvement techniques and underground construction due to marginal land areas now being developed. Lucy presented her paper "Geotechnical Aspects of the Upper Harbour Bridge Duplication and Causeway Widening" as part of the "Landslide and Slope Stability, Case Study" session. Hayden presented his paper "Pile Foundations in Liquefiable Soil – A Case Study of a Bridge Foundation" as part of "Deep Foundation Design and Practice" session.

Following the two-day iYGEC program, participants joined the 17th ICSMGE for the first two days of state-of-the-art lectures. This gave a taste of an international conference and the chance to listen to world renowned geotechnical professionals. The thoughts expressed during the 4iYGEC were summarised and presented in a special session of the 17th ICSMGE. These thoughts included encouraging regular iYGE conferences and giving young professionals the opportunity to participate in the ICSMGE.

Hayden and Lucy would like to thank the NZGS for their support, and providing this exceptional opportunity to attend the 4iYGEC and meet a worldwide network of other young geotechnical professionals.

Reported by:
Lucy Coe
Beca Infrastructure Ltd

> Full Technical papers prepared for the conference by Hayden and Lucy are reprinted in this issue of Geomechanics News

AWARDS

STUDENT AWARDS 2009

After a 3 – 4 year hiatus, this year has seen the redevelopment of the NZGS Student Award with more encouragement to students to participate in applying for the Award and submitting an abstract.

Two awards are presented, the Southern Region and the Northern Region.

SOUTHERN REGION

We had two applicants from the South Island with the following topics:

1. "Ground Response Evaluation for Enhanced Seismic Hazard Assessment" by Jawad Arefi
2. "Collapse Mechanisms of Chimney Caving" by Matt Fitzmaurice

Matt and Jawad presented their topics to a moderate sized gathering (approx. 20) at the University of Canterbury on 20 October. The judges had a very difficult decision as both topics were presented with excellent clarity. The visual presentations were both clear and concise and found both presenters explanations to be excellent with their handling of questions very competent. The judges overall could not distinguish between the two presenters and decided to jointly award the Southern Region Student Award for 2009.

NORTHERN REGION

We had five applicants from the North Island this year from which four were selected to present their topic at the University of Auckland on 27 October. The topics that were presented are outlined below:

1. "Small strain shear modulus of Auckland residual soils" by Anas Ibrahim
2. "Looking at the Clyde Power Project with the Benefit of Hindsight: Geotechnical Lessons to be learned from this Landmark New Zealand Infrastructure Project" by Daniel Scott
3. "Incorporating soil yielding into earthquake shallow foundation design" by Thomas Algie
4. "Dynamic response of single laterally loaded piles in Auckland residual soil" by Norazzlina M.Sa'don

The Northern Region presentations were made at the University of Auckland Engineering School on 27 October to a small audience. Again all presentations were of excellent quality and clarity with some curly questions asked of the presenters. The judges had an extremely hard task of deliberation with a final decision to award the Northern Region Student Award to Thomas Algie.

We congratulate all the students on their achievements and wish them well for their future studies.

Copies of the award winners submitted abstracts are included in this issue of the Geomechanics News.

Reported by:
Kate Williams
YGP Representative
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Southern Region Student Award 2009

M. Jawad Arefi - Ground Response Evaluation for Enhanced Seismic Hazard Assessment

Jawad has just started his PhD in geotechnical earthquake engineering under the supervision of Associate Prof. Misko Cubrinovski at the University of Canterbury.

His presentation explains the motivation for future research in evaluation of site response and basically covers his PhD thesis topic proposed about two months ago. He received his bachelor of civil engineering from Iran and his masters in earthquake engineering from ROSE school in Italy.

The dynamic response of soils beneath a site has a significant effect on the ground motion hazard of engineered structures (Boore 2004). To understand such a complex response and mitigate its consequences, one must have both a fundamental understanding of dynamic nonlinear soil behaviour as well as mathematical models capable of predicting soil response in future possible earthquakes.

There have been extensive studies on understating and evaluating the dynamic behaviour of soils during seismic excitation. It is known that under large strong ground motions, soil behaves nonlinearly and this nonlinearity has been incorporated into site response analyses in different forms depending to the problem in hand and computational limitations.

The shear modulus and damping properties of soils are critical in seismic response evaluation at both small- and large-strain response levels. As a result, considerable attention has been given to the characterization of shear modulus and damping properties of different soils, including the influence of mean effective confining pressure, soil type, plasticity, loading frequency, and number of cycles, among others (Darendeli, 2001; Seed & Idriss 1990; Vucetic & Dobry 1991). Previous studies however have not systematically examined the influence of soil fines content on modulus and damping characteristics at large strain, which is of particular relevance to Christchurch soils under strong ground motion excitation.

Different methods of soil modelling are used to simulate seismic site response. Equivalent linear (EL) modelling is the simplest, but most commonly utilized approach in practice. This type of modelling iteratively uses an equivalent shear modulus and damping as a function of soil shear strain, and represents a simple approximation of actual nonlinear behaviour of soil. As it is linear, the computed strain returns to zero following ground shaking and hence permanent displacements and soil failure cannot be predicted. The small computational effort and the few and physically-intuitive input parameters has lead to the EL approach being widely used for small-strain and amplification studies. Alternatively, nonlinear (NL) modelling of soil has the obvious advantage that actual (nonlinear) stress-strain path during the cyclic loading is explicitly accounted for. Nonlinear models can be formulated in terms of total or effective stresses, the latter allowing modelling of the excess pore water pressure and liquefaction during earthquake shaking. The ability to evaluate the development of permanent displacement and addressing large strain levels and failure are the key advantages of NL modelling over EL models. However, the numerous parameters which must be determined for conducting nonlinear analyses is a drawback for employing such methods.

Numerical tools to conduct seismic site response analysis have developed tremendously over the last 30 years. Following the development of the EL program SHAKE (Schanbel et al, 1972), a number of nonlinear total stress models have been proposed for considering the effects of soil conditions and nonlinearity on ground motion (Matasovic, 2006; Hashash and Park, 2001; Li, 1992; Pyke, 2000). However, because most nonlinear total stress numerical codes use a Masing Rule-based constitutive model, they cannot adequately account for both modulus and damping characteristics as a function of shear strain simultaneously. A range of sophisticated elastic-plastic constitutive models for effective stress analyses and liquefaction problems have been also proposed and extensively verified (Cubrinovski and Ishihara, 1998; Elgamal, 2003). These methods are considered the most appropriate for analysis of cases when significant nonlinearity and deformations are expected but they are far too sophisticated and hence difficult to implement.

Selection of which seismic response method is appropriate for a particular problem requires a thorough knowledge of each method's limitations and assumptions. Many studies have tried to identify those limitations to investigate the effectiveness of each method or document the benefits of taking into account the nonlinear modelling for complex circumstances. However, many past studies have each focused on some particular aspect of nonlinear soil response or particular input parameter, and therefore their outcomes alone do not provide a holistic picture on the appropriateness of the different methods for a particular problem.

Even more simplified than the EL approach, conventional seismic hazard analyses presently classify soils into several discrete groups, rather than explicitly modelling soil response. Based on the aforementioned site response studies and the composition of such discrete soil classes (e.g. A–E in NZS1170.5) it is clear these 'crude' soil class definitions are a significant source of uncertainty in the resulting seismic hazard at the ground surface. It is important to note that although probabilistic seismic hazard analysis is site specific, uncertainties regarding nonlinear soil behaviour have been ignored due to lack of data. Therefore, incorporation of uncertainties in soil characteristics into probabilistic seismic hazard assessment is expected to result in improvement of the estimated design ground motion.

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Southern Region Student Award 2009

Matt Fitzmaurice - Collapse Mechanisms of Chimney Caving

Matt has recently started his MSc thesis in Engineering Geology at Canterbury University after completing his BSc and Post Graduate DipEngGeol there.

His thesis topic is classifying overburden collapse above abandoned underground coal mines. He has spent two summers working in Australian mines and had a 7 month hiatus from University at the end of last year in which he worked as a consultant in NZ.

ABSTRACT

Subsidence is phenomenon often associated with mining activity, whether intentional, as with longwall mining for coal, or unintentional. Both unintentional and intentional subsidence present a major hazard to activities on the surface, particularly in urban areas where there are buildings such as houses, shops, schools and hospitals situated above the mine workings, or in areas where mining activities are also being carried out at the surface.

The predominant hazard comes from abandoned, often forgotten, mines. This forgetting of abandoned mines leads to construction above mine workings, with no knowledge of the voids below the surface. If the voids are close to the surface, or if the construction is a large enough project to warrant a drilling investigation, then the abandoned mine may be discovered, but if it is simply a residential development then the workings may not be discovered until the load induced by the buildings causes failure of the overburden and results in subsidence and structural damage.

Collapse of the overburden usually occurs when stress locally exceeds rock strength and loading progresses beyond the elastic limit (Pariseau, 2009). The presence of favourably oriented joints and fractures and also of water can encourage collapse of mine roofs or hanging walls.

Shallow underground mining is not common in recent times, with the advances in technology leading to more economical opencast mining methods, but before the technology was available, shallow underground mining was the most economical method of extracting ore. A good example of this is the West Coast coal mines. They were historically mined by underground methods as that was the most economically viable method at the time. Now, with advances in technology, we are able to return to previously mined areas to remove the coal that was left behind for roof support. This can present a safety hazard as you remove the overburden and remaining coal, because you may come across voids which are propagating towards the surface as the roof progressively caves in.

Chimney caving is one of the principle types of collapse associated with mining. There are several different mechanisms of chimney caving, which depend on the geology of the site. Chimney caving is characterised by the progressive collapse of the mine tunnel roof into the open mine workings. This type of failure is distinguished from other types of failure because the void tends to retain parallel sides rather than falling back at an angle of draw, although they often arch at the top if they do not daylight. According to Brady & Brown (1993), there are three main mechanisms of Chimney caving, which are each associated with different geological environments.

The first of these mechanisms is associated with weak or weathered rock or previously caved rock. This mechanism involves the rock mass weakening and effectively crumbling out of the roof. As the material falls from the enlarging void, it tends to bulk and fill the open cavity below. If the material falling from the void bulks enough it will fill the mine workings to such an extent that the loose, caved material will support the roof and collapse will cease. If bulking is not sufficient to fill the open space and support the roof, the void will continue to propagate until it reaches the surface or the roof forms a stable arch shape and ceases to propagate.

The second mechanism is associated with the unravelling of a discontinuous rock mass. This method involves failure of blocks along pre-existing fractures in the rock mass, despite the fact that the rock material itself may be very strong. This failure mechanism requires an open space beneath the propagating void as the previous mechanism did, and, as with the previous mechanism, bulking of the material released by the roof will affect void propagation if the void is filled to such an extent that it will support the roof. In a shallow mine, the thickness of overburden above the mine workings will also affect void propagation towards the surface. If there is only a relatively thin amount of overburden above the mine workings then there will not be sufficient material to fill the void enough to support the roof and it will propagate to the surface, assuming a stable arch is not formed.

The third mechanism involves the failure of large blocks controlled by major structural features, for instance local or regional fault lines, fault zones or shear zones. This mechanism is also termed 'plug subsidence' in much of the literature, as the rock fails as one large block, or 'plug', with the zones of low shear strength at its edges, along which it fails. As with the other two mechanisms, a void is required for the 'plug' to fail into. Unlike the others, however, the displacement at the surface is likely to be close or equal to that at the base of the 'plug'.

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Northern Region Student Award 2009

Thomas Algie - Incorporating Soil Yielding into Earthquake Shallow Foundation Design

Tom graduated with a Bachelor of Engineering from the University of Auckland in 2007 with first class honours. He went straight into a Ph.D. there studying under Professor Michael Pender researching non-linear earthquake behaviour of shallow foundations. In 2007 he received a Fulbright scholarship to the University of California, Davis where he conducted centrifuge tests of rocking bridge foundations. Currently Tom is carrying out large scale field tests of shallow foundations and expects to finish his Ph.D. by the end of 2010.

ABSTRACT

Soil yielding as an effective energy dissipation mechanism can be cost beneficial as an alternate earthquake design procedure. Current design standards in New Zealand minimise the work done by a foundation and channel all of the energy dissipation characteristics of a system into structural components. Traditionally, energy dissipation through soil yielding has been ignored because of the uncertainty involved with predicting soil behaviour; however research has shown that foundations can produce a well defined moment capacity. Finite element models of soilfoundation systems can satisfactorily predict behaviour however further validation is needed.

When an earthquake occurs, a foundation is typically designed so that even under extreme circumstances, the bearing capacity factor of safety of the part of the foundation still in contact with the soil is greater than one. However brief instances where the bearing capacity factor of safety falls below one have the possibility of being incorporated into geotechnical earthquake design. This type of mechanism is usually referred to as a 'rocking foundation'. This design philosophy is standard procedure in other areas of geotechnical earthquake engineering; such as earth dams, slopes and gravity retaining structures.

Research is currently being undertaken at the University of Auckland in the area of rocking shallow foundations. The research is two tracked; firstly finite element modelling is being carried out to accurately predict shallow foundation response during an earthquake, and secondly large scale field tests of rocking foundations are currently being undertaken. The motivation for this research is to incorporate a design guideline back into the loadings standard and have rocking foundations as a viable option for earthquake design in New Zealand.

In the first track of research, finite element modelling of rocking foundations, three finite element models have been developed.

The first model is a bearing strength surface model developed by Jeremy Toh at the University of Auckland. This model defines combinations of vertical load, horizontal shear and moment that produce failure for a shallow foundation. The surface is shown in Figure 1. The equation for it is taken from the Eurocode 8 equation for the yield surface of cohesive soils. The surface acts as a yield locus; all the paths within the surface are elastic and those on the surface perfectly plastic.

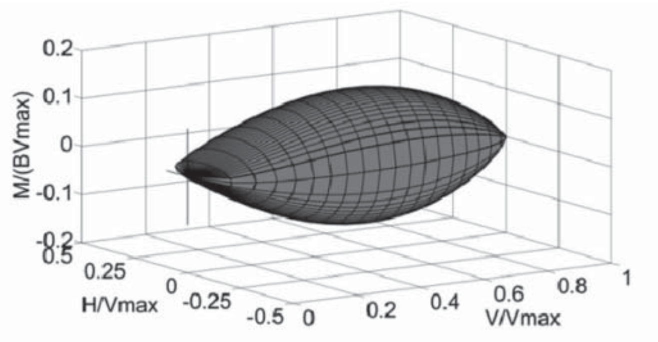

Figure 1: The yield surface as defined in the Eurocode 8 for cohesive soils.

The second model is a bed of spring's model developed at the University of Auckland by Liam Wotherspoon using the earthquake engineering software program Ruaumoko. Vertical and horizontal bi-linear springs were used to represent foundation behaviour. These springs are compressible only and can detach from the foundation so when a foundation rocks the reduction in bearing capacity is captured because of the detachable springs.

The final model is another bed of spring's model developed at the University of Auckland by Thomas Algie using the earthquake engineering software program OpenSees. The difference between this model and the last is that the OpenSees model uses non-linear vertical and horizontal springs. The springs follow a more hyperbolic function rather than the bi-linear function of the Ruaumoko model.

All three models were compared with centrifuge experiments that were performed at the University of California, Davis. These experiments were of rocking shear walls sitting on San Francisco Bay mud (clay). All

Figure 2: The test setup

three models showed roughly comparable results and all these values compare well with the centrifuge results. The OpenSees model produced results most comparable with the centrifuge data. However further development is still needed, as the sliding capacity of this model was insufficient.

The second track, large scale field tests of rocking foundations are currently being undertaken onsite in Pinehill, on Auckland's North Shore. Results from these experiments are expected to further validate the numerical models therefore allowing accurate prediction of rocking shallow foundations. Figure 2 shows what the test setup will look like. The foundations will be tested at various static factors of safety. The tests are expected to be taking place in September and October.

Once the results have been analysed and the finite element models validated, design guidelines on rocking foundations can be written. The old loadings standard (NZS 4203) had a provision for designers wishing to incorporate rocking foundations into their design. However this was taken out because there was very little experimental evidence to support the idea and there were no 'actual guidelines' included. The field testing proposed, running parallel to the numerical model development will provide an excellent base in which to write these guidelines and have soil yielding as a viable option for earthquake energy mitigation in New Zealand.

PROJECT NEWS

A Rock Dilatometer Lands in New Zealand

Figure 1: (left) Transportation to the drill site

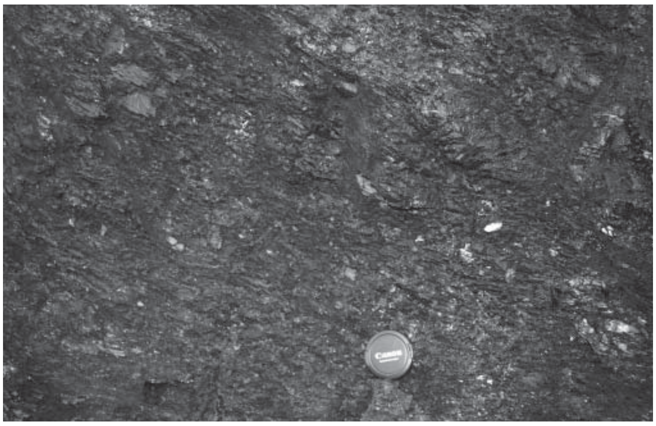

Figure 2: The weak, sheared nature of the coal seam

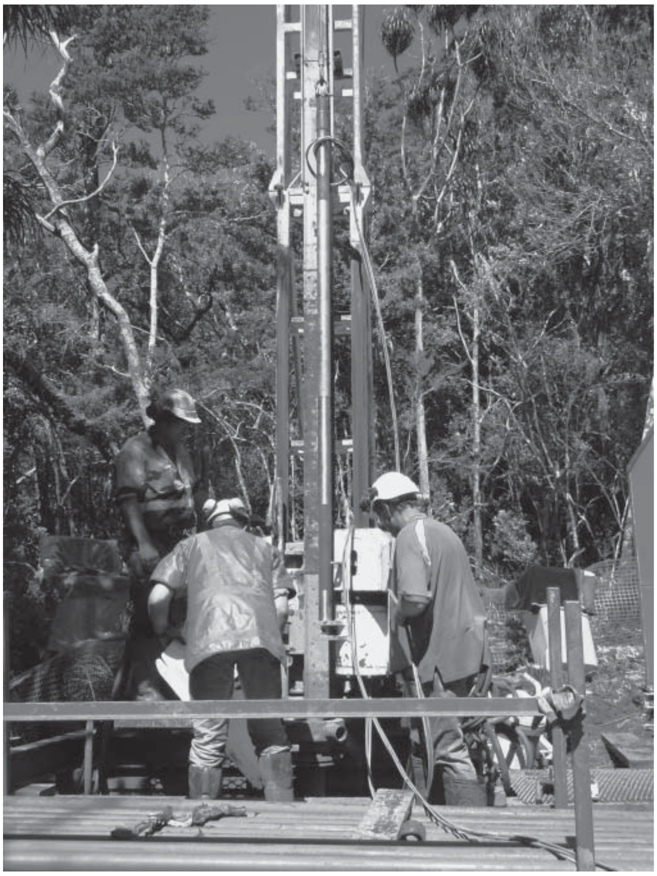

Figure 3a: The dilatometer as it's about to be lowered down the borehole.

URS New Zealand Limited (URS) acquired a rock dilatometer in late 2008 after coming up against a challenging slope stability problem at Solid Energy's Stockton Mine, where a weak, sheared coal seam (Figure 2) formed a critical portion of a slope. It has traditionally been difficult to sample and test this particular form of coal in the laboratory, and therefore shear strengths have historically been estimated via back analysis of existing slopes and previous slope failures. The values derived from back analyses were proven to be too conservative for this particular slope – it was known that the coal strength was higher, but it was not known how much higher. Unless that higher coal strength could be demonstrated, there would be no other choice but to carry on with the more conservative design. It was decided that in-situ testing using a rock dilatometer may be the way forward, and URS borrowed a rock dilatometer from one of its offices in the US to assess its effectiveness.

The rock dilatometer (Figures 3a and 3b) is a cylindrical, borehole expansion device containing a rubber membrane that is expanded against the borehole wall in order to measure the borehole deformation as a function of the applied pressure. It is essentially a high-capacity pressuremeter that is designed to be used in weak to strong rock. The dilatometer has a maximum working pressure of 30 MPa, whereas pressuremeters, designed for soil to soft rock, typically have a capacity of less than 10 MPa.

The rock deformation modulus is directly measured during the test. If the tests are carried out in relatively weak rock, where the pressure is increased past the onset of plastic deformation, then it is also possible to estimate a shear strength for the rock through back-analysis of the pressure-deformation curve (Figure 4).

Some useful results were derived from the testing programme carried out at Stockton, where the higher coal shear strength was demonstrated and was used in subsequent stability analyses. URS recognised the usefulness and potential future applications of the dilatometer for projects related to mining, underground excavations and foundation design, and have since invested in a dilatometer that is based in New Zealand.

Plans are in the works to use the dilatometer to estimate deformation and strength properties of weakly-cemented Waitemata Group sandstones and siltstones for a tunnelling project in Auckland and to estimate the deformation properties of greywacke for another tunnelling project in the South Island. URS' Australian offices have also expressed interest in using it on a number of their projects.

Dilatometer Statistics:

Maximum Test Pressure: **30 MPa**
Borehole Diameter: **76 mm (NQ)**
Effective Test Length: **460 mm**
Maximum Test Depth: **90 m (with current set-up)**

The dilatometer is available for use by other organisations on a commercial basis. For more information, please contact Rori Green in the URS Christchurch office.

Reported by:
Rori Green
Senior Associate Geotechnical Engineer
URS New Zealand, Ltd.

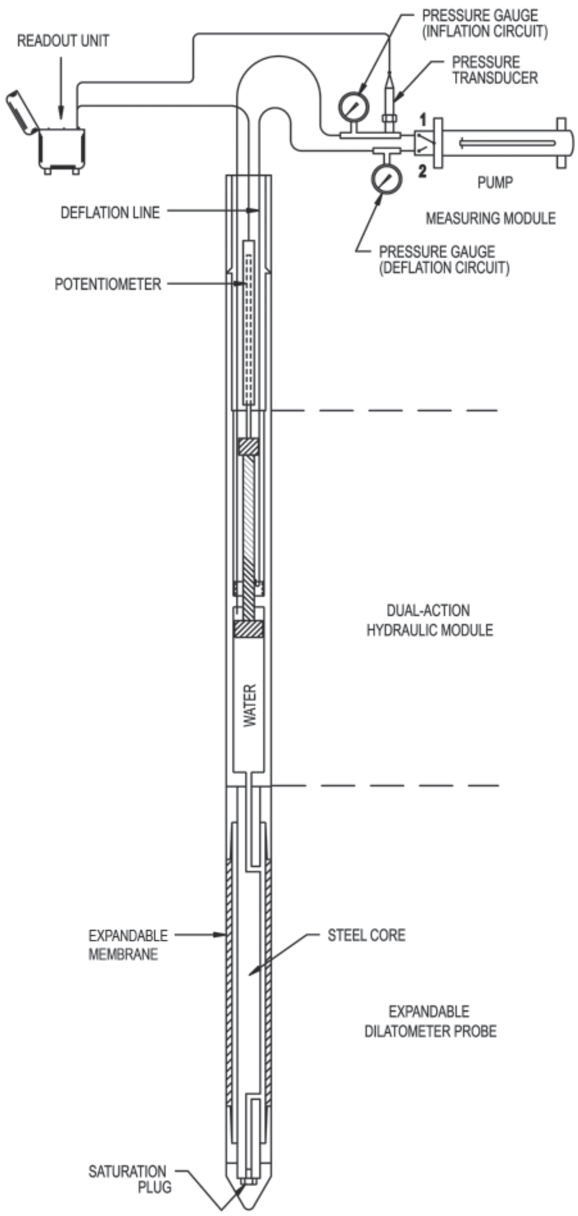

Figure 3b: (above) Schematic of dilatometer probe (image from Roctest PROBEX PROBEX-E50037-W fact sheet)

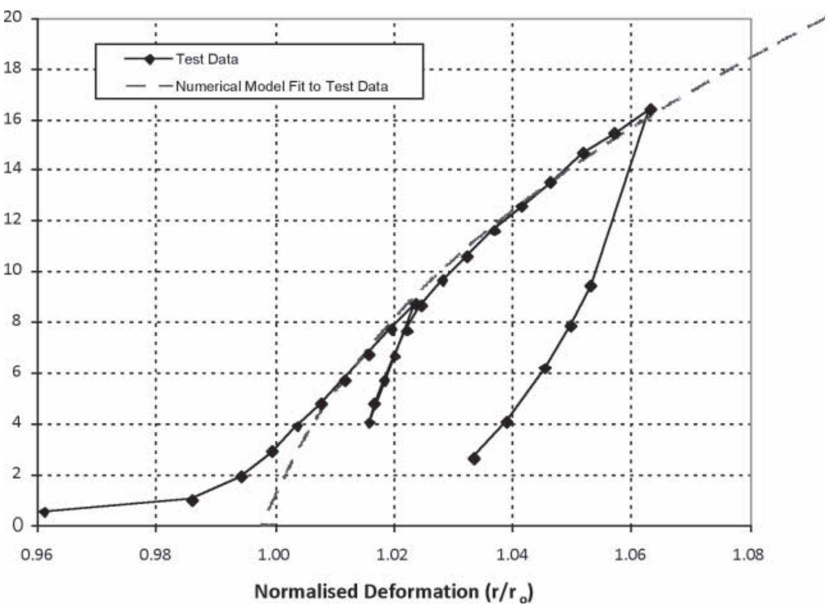

Figure 4: (left) Plot of test results

GeoNet landslide response - Waihi Landslide, Taupo (June/July 2009)

– Chris Massey, Dick Beetham, Charlotte Severne, Garth Archibald, Graham Hancox, William Power.

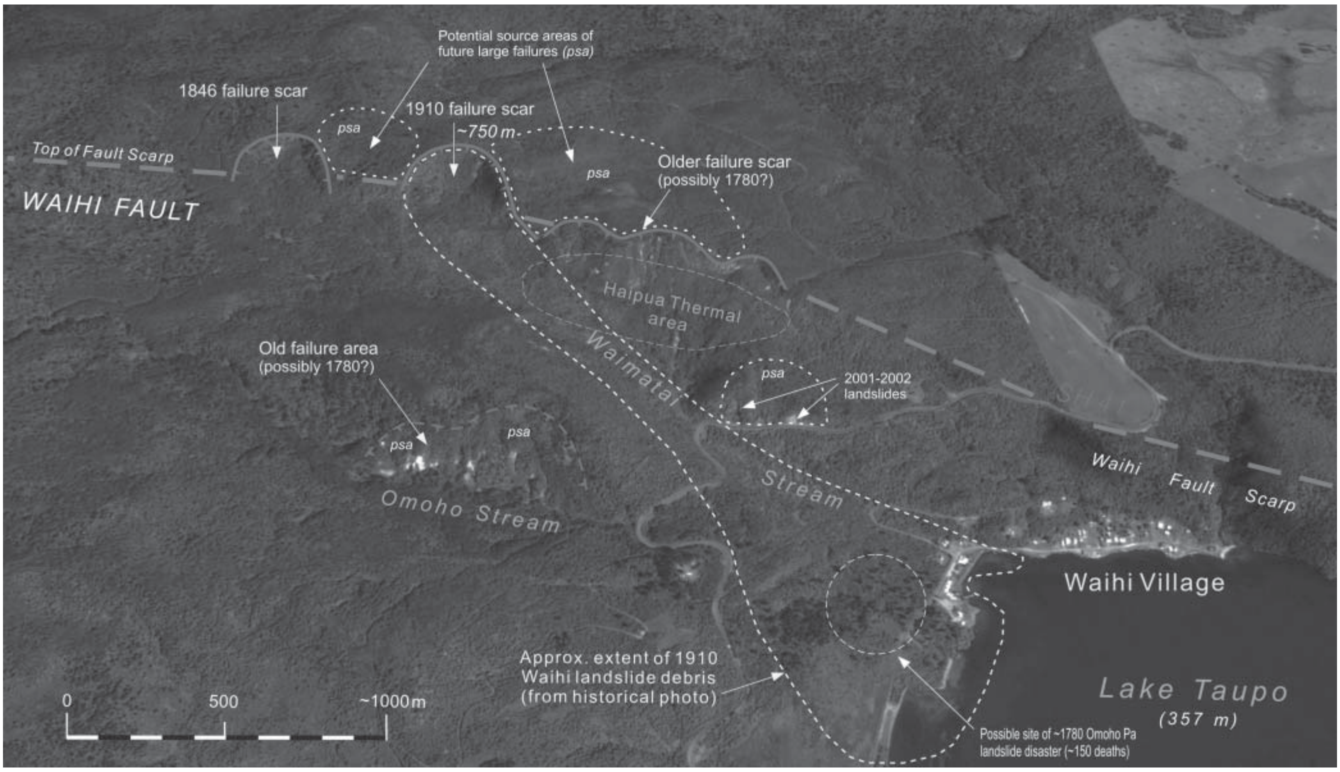

Figure 1: Annotated Google Earth Image (20 March 2007) showing the Waihi Landslide area above Waihi Village. The main features shown include the source areas of three historical failures (1780, 1846, and 1910), and the estimated extent of the 1910 landslide debris, based on a historical photo. Note that the main area of Waihi Village is located outside (north) of the 1910 landslide zone. Some potential source areas (psa) of future large landslides are also shown.

1.0 Introduction

On the 30 June 2009 The GeoNet Landslide response team attended a potential landslide situation developing at Waihi Village, located at the southern end of Lake Taupo. The response was initiated because national news reported that Waihi Village had been evacuated, SH41 closed and a civil-defence emergency had been declared by Taupo District Council, due to the threat of another large landslide at Waihi. The area around Waihi is synonymous with landslides, as three historic landslides have occurred in the area; two of which are well documented.

Earthquake swarms had been occurring in the area around Waihi Village from about April 2009 until about August 2009. Two shallow (4 km) magnitude 4 (M4) earthquakes occurred over the weekend of 27-28 June, very close to the village. These earthquakes and other environmental changes noted by the local community heightened their concern about the possibility of another large landslide from the Hipaua Geothermal Field.

2.0 Background to the Area

Several large landslides have occurred from the Waihi Fault scarp above the Hipaua thermal field at the southern end of Lake Taupo in the last c. 230 years. These have transformed into very large debris flows in Waimatai Stream (Figure 1). A well documented event occurred at night in May 1846 with the loss of 64 lives in Te Rapa (Little Waihi) village, including the paramount chief Te Heu Heu. Another occurred on the morning of 20 March 1910 with the loss of only one life as people were alerted and escaped the massive debris torrent. There is also evidence of an earlier failure in about 1780, which apparently buried a pa at the mouth of nearby Omoho Stream with the loss of possibly 150 lives. The source area of the 1780 landslide is uncertain, but it could have been in the Waihi Fault Scarp just north of the 1910 failure scar, or possibly in Omoho Stream ~500-700 m upstream of SH 41. Past failures on the steep fault scarp seem to have been related to ongoing tectonic and hydrothermal activity in the area, and also to periods of heavy rain. The 1846 failures initially formed

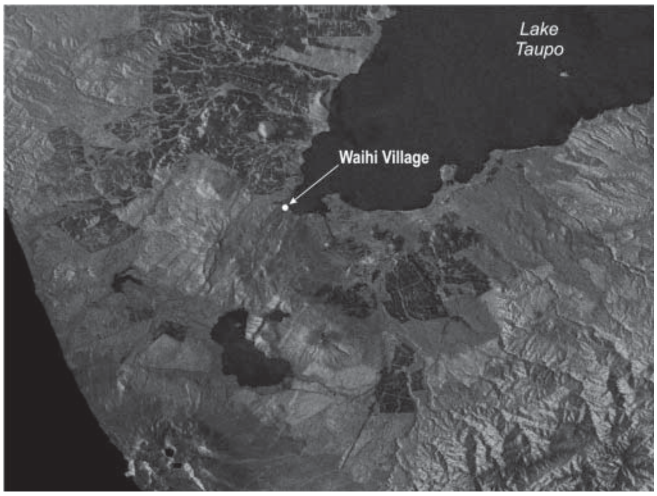

Figure 3: (left) Map of surface change for the Waihi area derived from Interferometric synthetic aperture radar (InSAR) images pre and post the M4 earthquakes in June. Note there is no change between the Waihi area and surrounding areas, indicating no obvious ground deformations.

a small landslide dam in Waimatai Stream, and the main failure occurred when the dam breached during heavy rain. The 1910 landslide also originated at the Waihi Fault scarp about 300 m to the north of the 1846 failure area. The 1910 failure began with a loud noise and a cloud of steam or dust, and may have been triggered by a geothermal eruption. It was larger than the 1846 landslide, and produced a debris flow of about 3 million m^3 which flowed 2 km to the southern shore of Lake Taupo (Figure 3), subsequently forming a wave on the lake about 3 m high.

3.0 The GeoNet Landslide Response

The GeoNet landslide response team drove to Taupo on the morning of 30 June 2009 and were briefed by Taupo District Council (TDC) along with MCDEM, Environment Waikato and OPUS (Land Transport NZ consultant for SH41). A brief summary as follows: The villagers of Waihi had self evacuated, SH41 had been closed, all residential property and the Braxmere Lodge had been evacuated; Evidence of imminent landslide activity (at this stage) were anecdotal, with no quantifiable data available.

The GeoNet response team comprised: Dick Beetham, Chris Massey and Garth Archibald of GNS Science; and Charlotte Severne of NIWA.

3.1 Response activities

The initial response comprised aerial inspection of the area, to look for signs of "recent" landslide activity, followed by field inspections of key locations around Waihi Village, the Hipaua geothermal area (the Steaming Cliffs) and the Waihi Fault escarpment.

During the field inspection, the response team along with local surveyor Russell Dick (Central Surveys, Taupo) installed 11 survey marks along the ridgeline of the Waihi Fault escarpment (Figure 2). The marks were located in and around potential future landslide sources (based on the results of previous work), which if failure were to occur would have potential to impact parts of the village.

Figure 2: GeoNet in association with Central Surveys Ltd have installed a series of survey marks to monitor the stability of the ground above the Hipaua thermal area. Nineteen galvanised tubes have been placed along the scarp above the true left bank of the Waimatai stream as close to the cliff edge as possible in areas with acceptable sky visibility. These marks together with LINZ marks outside the area of interest form the Waihi landslide monitoring network.

Above: 'Steaming Cliffs' at Haipua Thermal Field looking towards Lake Taupo

3.2 Conclusions from the site inspections

Based on the results of the initial field inspection and monitoring:

- The current reported damage in the village appeared consistent with a ground-shaking intensity of ~MM5, associated with a magnitude M4, localised shallow earthquake.
- Evidence of recent (over the last few months) large-scale landslide activity was not seen.
- There was evidence to suggest that historical instability of the ridge-line in the area north of the 1910 landslide source area, and between the 1846 and 1910 source areas, had occurred, possibly post 1910. It is difficult to assess the age of these features (also possibly pre-1910), however none appeared recent (i.e. in the last two years).
- Based on historical precedent, previous studies and the results from these recent observations, there is still a significant risk to parts of Waihi village and Braxmere Lodge from large landslides originating from the steep cliff formed along the Waihi Fault scarp. In addition, other hazards affecting the village include rock falls from the steep cliffs immediately above the village; and reactivation of the large (and assumed to be actively-creeping) landslide currently affecting SH41 (this assumption will be tested when the survey results become available).

4.0 Ongoing Activities

The GeoNet response team made a number of recommendations to help develop a better understanding of the slope behaviour in the Waihi Village area. Several of these are currently underway. 1) the acquisition of ground-based survey data including installation of additional ground survey marks to monitor ground movement (or lack of movement) over time; 2) acquisition of satellite-based radar images (InSAR) to ascertain if movement had occurred in the weeks preceding the M4 earthquakes (Figure 3); and 3) comparison of LIDAR data collected pre and post the M4 earthquakes.

5.0 Recommendations

The team recommended making a multi-disciplinary re-assessment of the geological hazards and associated risks affecting Waihi Village and acquiring data to underpin this assessment. The report and its recommendations are currently being evaluated by the Taupo District Council and other responsible agencies as they decide on an appropriate path forward.

The Waihi community continues to be vigilant and aware of potential landslide signs and their triggers. Landslide signs include reduced flows along drainage lines, tilting/deformed vegetation and deformation of SH41. Landslide triggers include: storm rainfalls (> 30 to 50 mm a day) and/or periods of prolonged rainfall (> 10~20 mm a day over days or weeks, and earthquake-induced ground-shaking typically associated with shallow earthquakes. Typically MM7 to MM8 (on the modified Mercalli scale and equivalent to a peak ground acceleration of ~ 0.2g) is required. This level of shaking usually requires localised, shallow earthquakes of magnitude ≥ 5.

Information relating to the landslides in the area and the other useful references are contained in the GNS Science Report 2009/34, Field investigations at Waihi Landslide, Taupo 30 June & 1 July 2009, which can be down loaded from here:

http://www.geonet.org.nz/resources/landslide/landslide-reports.html

6.0 Acknowledgements

The Landslide team would like to acknowledge: GeoNet for funding this work; TDC for their local knowledge and hospitality; support from the residents of Waihi Village, Russell Dick (Central Surveys) and our GNS Science colleagues: Grant Dellow, Rawiri Faulkner, Brad Scott, Nick Perrin, Biljana Lukovic, Sergey Sansomov and Mauri McSaveney.

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International Summer School on Rockslides and Related Phenomena

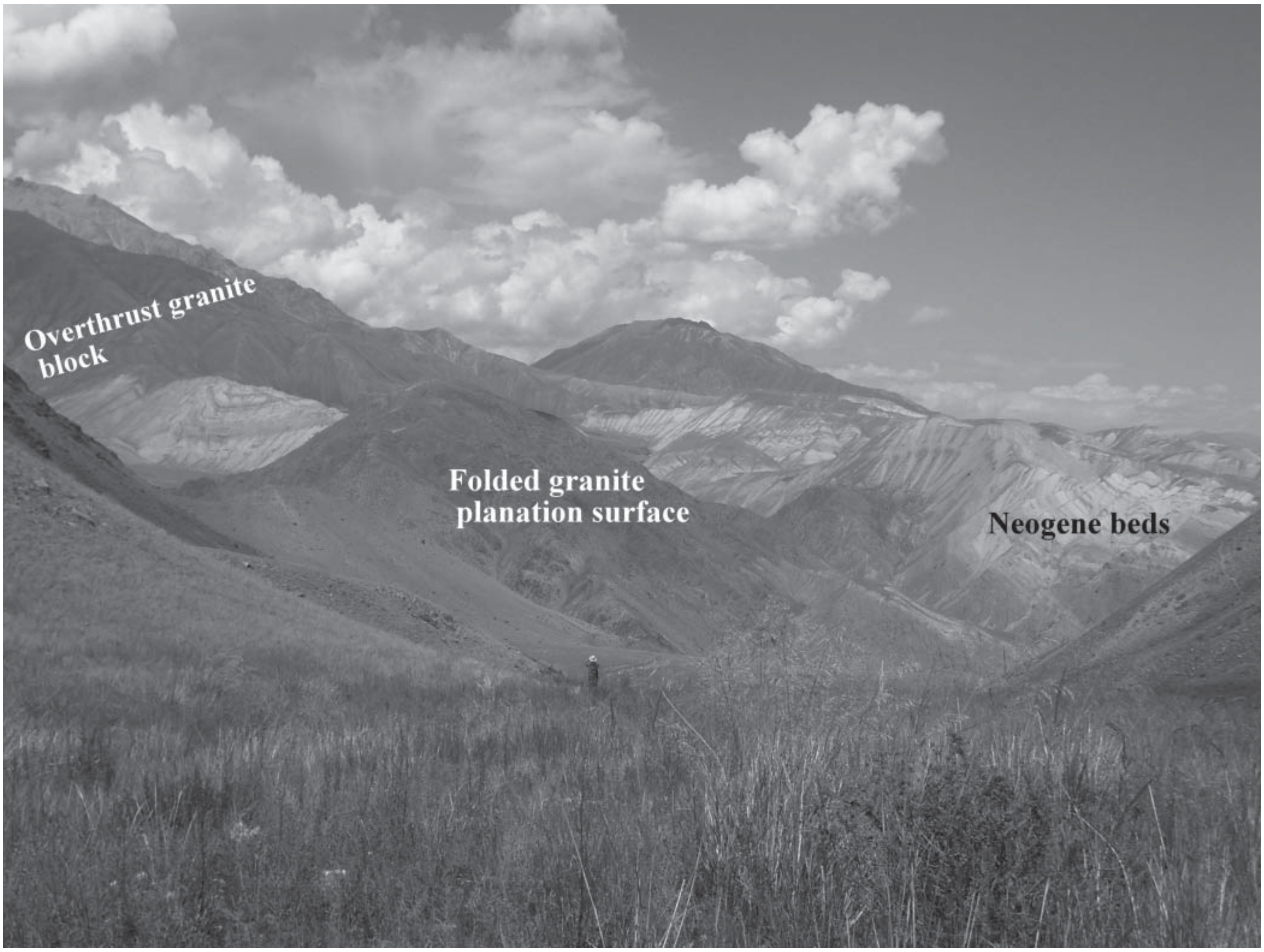


Figure 1: Neotectonic features are very expressive throughout the region. The red and white Neogene deposits overlay the dark red basement granites. Neogene sediments were originally deposited flat and are now significantly folded as seen in the large anticline structure in the centre of photograph where the granite and overlying red and white beds have been pushed up. The granite mass in the upper left of the photo is being overthrust on top of the whole sequence so that they now lay unconformably over much younger Neogene sediments.

During August 2009 I attended a 2-week field course to study large landslides phenomena and geology in the Central Tien Shan Mountains, Kyrgyzstan. Herein I describe the course and give a taste of just some of the fascinating phenomena observed.

Course information

The annual field-excursion/school is run by the Kyrgyz Institute of Seismology and Russian Academy of Science with support from the International Consortium on Landslides. This year there were eight participants, mostly from Europe. It was the first time that a New Zealand student has participated.

The school is held in Kyrgyzstan because of the high density of diverse landslide phenomena caused by the combination of the country's unique geology, topography and Neotectonics. Further, the arid climate makes fieldwork straightforward because of the way that landforms are well preserved and because of the lack of vegetation obstructing the view!

Aims of the school are to: demonstrate various types of large landslide and how they can be recognised by morphology, internal structure and indirect evidence; and give techniques for landslide investigation.

Geology and geomorphology in the Tien Shan

The Tien Shan are formed in basement rock of Pre-Cambrian to mid-Palaeozoic age, composed of heavily tectonised meta-sediments and intrusive granites that have undergone at least two mountain building phases during the Palaeozoic. For the duration of the Mesozoic, in a long period of tectonic quiescence, the mountains were eroded to form an extensive planation surface. Since then, the flat surface has been tilted, faulted and folded in the formation of the Tien Shan during the late Pliocene and Pleistocene, as a result of the collision of the Indian plate into the Asian plate - the same process that formed the Himalayan Mountains. Remnants of the surface are widespread and overlain by thick Neogene sedimentary sequences (figure 1). The region remains seismically active and the 1992 Susymmur earthquake which produced extensive surface rupture, a cluster of landslides, and building damage is testament to this. Relief of the Tien Shan is striking with some elevation differences between the basin and ranges of over 7000 metres. Some of the higher elevations are glaciated. Most of the Tien Shan however is semi-arid, with some parts receiving as little as 100 mm per year, and at most 2000 mm.

Some landslide observations

During the field course we examined a wide range of rockslide and rock avalanche phenomena, mostly large, earthquake triggered events. The course was structured so that we first examined landslide morphologies, then internal structures, and finally some of the indirect impacts of landslide events. I present here a sample of the key observations for each of these three parts.

Landslide movement style and the underlying topography strongly influence deposit morphology. For example, sometimes the entire mass of displaced material will travel many kilometres, while in other cases only a small portion of the mass developed a long runout phenomenon. It was shown that the initial fall height and trajectory of the mass can make a difference to the resultant morphology, and can control whether or not secondary failure motion occurs. Compare figures 2 and 3.

Internal structure provides insight to the movement dynamics of rock avalanche. One peculiar characteristic of most rock avalanches is that the material undergoes little mixing despite long runout. This was clearly seen in one outcrop where an original contact between two lithologies within the landslide debris was perfectly preserved (figure 4). Yet both rock masses were comminuted to the point that we could break the unweathered rock (granite and metasediment) with fingers. It is the high proportion of comminuted sub-micron particles that make rock avalanche deposits so impermeable and effective at damming rivers.

Figure 2: Seit Rock Avalanche. The majority of the deposit accumulated at the distal end, marked in white, some 2-3 km from the vacated source area marked in black. The reason for this may be that the steep source area grading into a more gentle but straight valley allowed for the entire mass to remain as a relatively 'intact' but comminuted mass.

Figure 3: Northern Karakungen rock avalanche showing initial emplaced mass (white) and secondary failure deposit (black). The secondary failure occurred moments after the initial failure, with material being 'squeezed' out from the basal part of the initial mass upon its emplacement. The lack of failure scar in the initial mass gives support for a basal failure, rather than a side collapse like that seen at other landslides in the area. While the initial failure didn't develop long-runout behaviour, the secondary failure did, travelling about 2 kilometres. A likely reason in this case is the initial moving mass, travelling very steeply, was decelerated abruptly as it reached the valley floor. Arrows indicate initial and secondary movement directions.

Figure 5: (below) Well sorted, stratified deposits with angular boulders 100 metres downstream from the remains of an old rock avalanche deposit. The material in this photo must have been deposited by water (dam breach) but has not travelled far since its initial deposition as a landslide. (Photograph by A. Strom.)

Figure 4: Kokomeren rock avalanche deposit (3-4 kilometres from source) showing perfectly preserved contact between two lithologies: pinkish granite on the left and grey metasediments on the right. Both rocks, unweathered, also retain their original structure but are completely comminuted and will break up in fingers. This part of the deposit is the remaining part of the crest of a 200 m high dam from the Kokomeren rock avalanche, which has since breached. Over 15 km of the valley upstream was inundated and the current river had to cut a new course through the valley to be in its present day position.

Indirect evidence of a landslide, often in the form of dam deposits, can be used to locate otherwise inconspicuous ancient landslides. Dam breach deposits also indicate landslide events. At one site, a well-sorted, stratified, angular gravel and boulder sequence was seen in a floodplain exposure just downstream of a possible landslide (figure 5). The angularity of the clasts, totally uncharacteristic of the rest of the fluvial deposits upstream of the inferred dam breach site, was the key to identifying the landslide breach deposit – the sediments must have had a very local source and began as very angular landslide material.

Other benefits of the course

The geology and landslide phenomena are world class but the region is also a place of beauty and cultural interest. The course provides an opportunity to compare practices of another country to that of New Zealand, meet researchers from around the world and it opens doors for further research in the region, which is strongly encouraged by the organisers. I highly recommend any students with an interest in landslide phenomena and geology to consider taking the course. Visit the ICL website for more information and contact details of the course organisers. Note the link below is for the 2009 course information only. Course details for 2010 will be advertised over next few months.
http://www.iclhq.org/Summer_School_announcement_2009.pdf

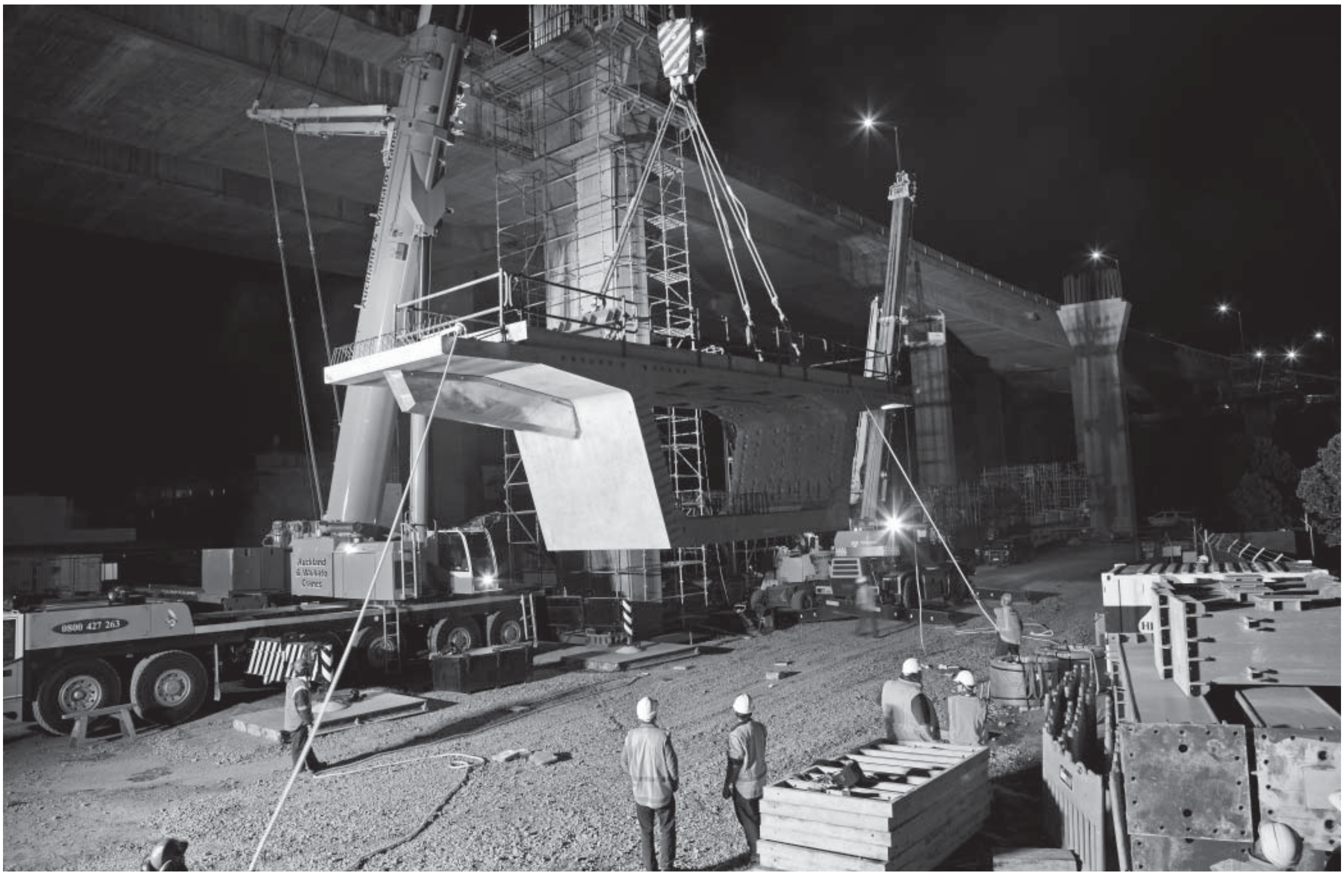
Acknowledgements:

I thank the EQC and GNS Science for supporting my travel and attendance at this course.

Reported by:
Samuel McColl
University of Canterbury

Upfront in Auckland – The Newmarket Viaduct Replacement Project

Robert Hillier and Michael Wulff – Tonkin & Taylor Ltd



Built in the mid 1960s, the Newmarket Viaduct conveys motorway traffic high above the busy Newmarket Broadway. Now, as it approaches its 45th anniversary, it is being replaced as part of an extensive suite of upgrades to Auckland's motorway network. The four-year replacement project is known as Newmarket Connection, a name which references both the motorway capacity and urban design benefits the new structure will provide.

Not only does the Newmarket Viaduct preside over one of Auckland's most fashionable, and retail-focused suburban centres, this landmark structure also accommodates the most highly trafficked section of road in New Zealand. The project therefore provides significant challenges to ensure the work is completed with least disruption and risk to both the motoring public and the surrounding community.

Since the original viaduct's construction, traffic patterns have changed markedly - and over the last 30 years the number of vehicles passing across the structure has more than trebled. To help mitigate the effects of this increased wear, tear and general ageing, overweight vehicles have actually been prohibited from driving across the structure. As the main access south of the city, the viaduct also serves as a life link. Therefore in addition to enhancing capacity, the new structure will be built to exacting modern design standards to address safety and seismic limitations inherent in the existing structure.

The contract for the project was awarded by the NZ Transport Agency (NZTA) to the Northern Gateway Alliance (NGA), recognising the high levels of design and construction skills that had been brought to bear in the successful delivery of the Northern Gateway Project, which extended SH1 north of Auckland. The highly successful team of NZTA, Tonkin & Taylor, Fulton Hogan, Leighton Contractors, URS & Boffa Miskel has been joined by Beca and VSL to provide additional design and construction skills, and is now operating as NGA Newmarket.

At the heart of the replacement project are the carefully orchestrated phases of construction and deconstruction that will see southbound capacity enhanced and delivered long before overall project completion.

Geotechnically the project has an interesting mix of challenges, reflecting the underlying influences of the basalt flows from the nearby Mount Eden (Maungawhau) and Mount Hobson (Remuwera) volcanoes. These flows converge beneath and towards the present alignment of Broadway.

Top: Placing the first pier head

Above: Birds-eye view of first pier head and segment construction

Left: Grout consolidation of fractured basalt within an anchored deep excavation

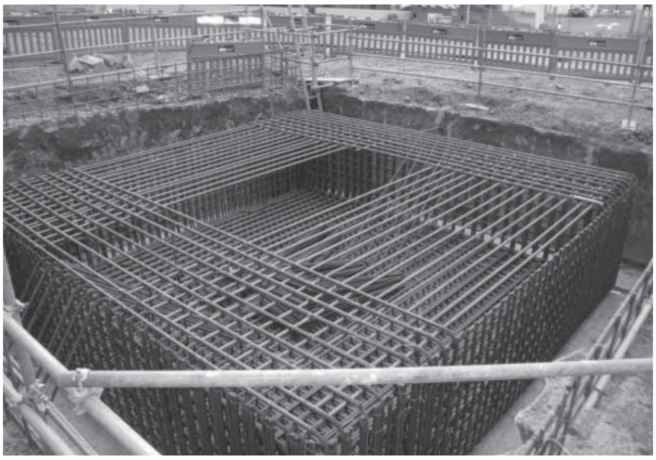
Opposite: Lifting of the first pier head

Top: Pile cap shuttering and pier stub reinforcement

Above: Deep excavation for a pad foundation within massive basalt

The basalt flows which provide the target for many of the shallow pad foundation bases have yielded many of the construction challenges. Locally the basalt has been found to be highly variable with thinner pockets and/or of poorer quality than had been originally anticipated. Conversely where deep excavations have been required (to allow pad foundations below potential future and adjacent basement developments) the basalt has proven both obstinately thick and massive, but also loose and rubbly. Deep excavations through the basalt have therefore required a variety of treatments from simple drape meshing through to rock bolting. Where these deep excavations have had the potential to undermine existing pier footings, pre-stressed anchors and intensive monitoring works have been implemented to ensure security of the existing viaduct.

All permanent pad foundations for the new structure have been subject to extensive grid drilling and grouting. These works have been undertaken to confirm adequacy of the basalt and to consolidate these materials where the basalt is found to be excessively fractured and/or voided. Grout takes have been highly variable, ranging from nominal volumes to fill the drill holes, up to $60m^3$ where major voids have been identified. In one case the same Tomo feature was identified in several adjacent drill holes. A CCTV survey confirmed an expansive cavity within



Clockwise from top left: Shallow pad, reinforcement cage (7m sq, 2m deep)

Construction of pile cap over piles

Bryden McKinnell, Resident Engineer on the original viaduct project, and his construction manager, John Built, are flanked by the present day Construction and Project managers, Patrick Arnold and Ian Harbeck (left to right)

Open Excavation and preparation of basalt surface by power jetting for a shallow pad foundation

the basalt at around 8m depth. This single feature alone required 30m^3 of concrete to fill.

Extensive temporary works, in the form of bracing, is also required to support the existing structure during the staged deconstruction as sections of the existing structure are removed. This requires an additional array of temporary foundation supports which will need to resist both the deconstruction loads and potential seismic actions. Individual pad foundations are being constructed with solid bar and multi-strand tie down anchors, where required, to resist seismic uplift and overturning loadings. Locally the challenges of installing anchors through basalt of variable quality has required the anchors to be installed with grout socks and/or be pre-drilled and pre-grouted.

At other locations where the basalt flows are thinner, piled foundations have been used, keyed into the underlying East Coast Bays Formation (ECBF) basement rock. The 1.2m diameter piles have been excavated through oversized casing to allow construction through the basalt, and have extended up to approximately 30m below ground level in to the underlying unweathered ECBF basement rock. All pile rock sockets have been grooved and inspected using CCTV cameras.

At the Gillies Avenue off-ramp and St Marks Road on-ramp, the approach embankments have been widened for the new carriageway alignments by a combination of mass gravity walls, MSE walls and anchored gravity walls.

At present approximately 60% of the foundation works have been completed and piers are springing up across the site. The first of the balanced cantilever segmental sections are now beginning to span out from the pier heads and shortly a highly visible blue framed truss will be erected to assist in the construction.

This is a landmark project on an elevated stage within a very public arena. As the new structure rises and stretches across Newmarket the engineering challenges and their solutions will inevitably become a fascinating visual focal point for both locals and the throngs of busy Christmas shoppers alike.

TECHNICAL NOTE

An Insight into the Performance of Flexible Debris Flow Barriers

Kevin Hind – Tonkin & Taylor Ltd

Introduction

Although traditionally used for the mitigation of rockfall hazards, flexible "ring net" barriers have more recently found an application as debris flow control structures. The first flexible barrier system was installed at Illgraben in 2005 by the Swiss Federal Institute WSL, although flume trials were undertaken by the US Geological Survey as early as 1996.

The author has recently undertaken the detailed numerical modelling and geotechnical design of a flexible barrier for the Awatarariki Stream in the eastern Bay of Plenty. Together with the Waitepuru Stream, the Awaratarki Stream was the source of several debris flows that impacted the township of Matata in May 2005. With a height of 15m and a width of more than 40m, the proposed barrier would be significantly larger than any previously constructed.

One of the fundamental decisions in flexible barrier design is the selection of the ring diameter. Field trials and flume experiments undertaken in Switzerland by WSL have shown that:

- A ring diameter equal to the 90th percentile grain size (D_{90}) results in almost complete debris retention (Wendeler et al, 2008); and
- A barrier with a ring diameter of $2xD_{90}$ allows the passage of some material, however it is still highly effective as a debris flow detention measure (Volkwein, pers comm).

A ring diameter of up to $2D_{90}$ is therefore considered to be a reasonable basis of design.

An obvious question to ask is "*how does a structure which is predominantly void space prevent the passage of a multi-phase fluid?*"

Although existing installations provide clear evidence that flexible barriers are highly effective at retaining debris flows, the author has encountered a general belief amongst people unfamiliar with flexible barriers that the structure would behave more as a sieve than a net i.e. debris material smaller than the ring diameter will readily pass through the barrier.

This article presents the results of informal research undertaken with the aim of gaining an understanding into the mechanics of debris flow retention by a flexible barrier. The assessment was undertaken using a physical model.

Model Structure

A flexible barrier is formed from interlocking steel rings, each consisting of several steel strands. The physical model used in the research presented here was constructed using key rings with outside diameter of 36.5mm and an internal diameter of 31.0mm. The model net is shown in Figure 1. The model rings were connected at their edges by wire rather than being interlocked. This arrangement was adopted as a means of simplifying the geometry of the net as well as maximising the relative area of void to steel. Although not strictly a scale model, the "net" is expected to give an insight into barrier-debris interaction. The net was connected to a wooden frame making it rigid.

"Debris flow" material was represented by glass and plastic spheres with diameters of 4.5mm, 7mm, 11mm, 15mm and 22mm respectively. The diameter of the smallest class of sphere was such that passage through the net could only occur via the centre of a ring.

Symmetry allows the net structure to be represented by a series of contiguous hexagons, each with a central circular void (Figure 2).

Single Sphere – Net Interaction

The first series of experiments consisted of the blind throwing of single spheres at the centre of the net. Three unique outcomes were possible (Figure 3):

1) The sphere passes through the net without any physical contact (Sphere "A" in Figure 3);
2) The sphere clips or glances off the side of a ring and continues through (Sphere "B");
3) The sphere squarely strikes the front of the net and bounces back (Sphere "C").

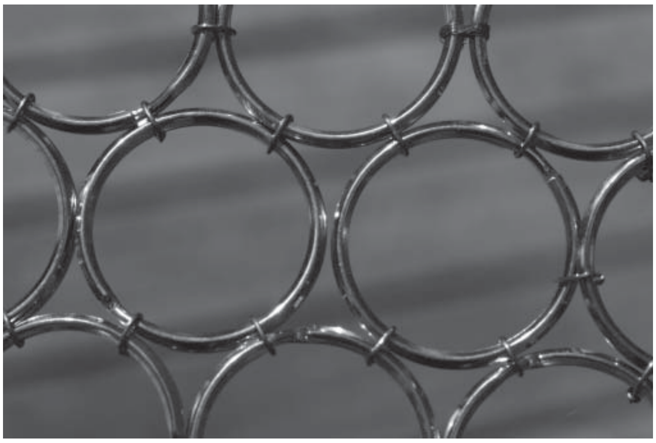


Figure 1 View of the "net" used in the physical model

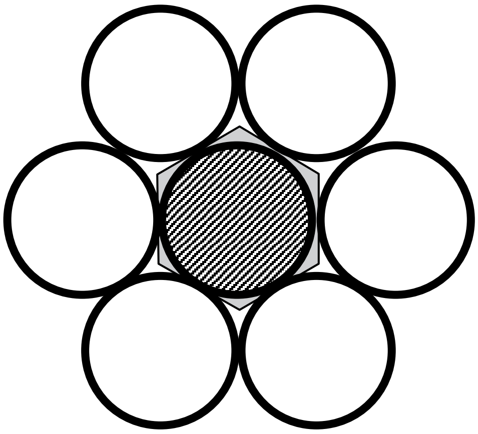


Figure 2: Geometry of the model net showing the relative location of the hexagon (grey) and central void (hatched)

Case 1: No Contact

In order for a sphere to pass through the internal void without striking the side of the ring, the centre of the sphere must be a minimum one radius away from the ring.

The probablity of this occurring (Pa) is:

$$Pa\ (\%) = A_c / A_h \times 100$$

Where:
A_c = area of dashed circle shown in Figure 3
A_h = area of enclosing hexagon

This can further be expressed as:

$$Pa\ (\%) = [\pi(R_i - r)^2]/(R_o.P/2)x100$$

Where:
R_i = inside radius of ring
r = radius of sphere
R_o = outside radius of ring and hexagon apothem
P = perimeter of hexagon

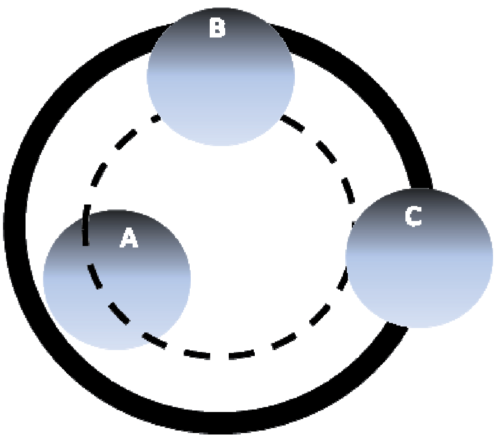


Figure 3: The three possible outcomes of sphere-ring interaction: A) no contact B) glancing off the side of the ring C) bouncing back off the net. The dashed line defines area of no contact (Ac)

For this to hold true however, the sphere would need to approach on, and maintain, a perfectly straight trajectory which is perpindicular to the surface of the net in all planes. In reality, the sphere has a curved trajectory in the vertical plane due to gravity, as well as an oblique approach angles on the horizontal plane due to the irregular method of launch. This oblique angle of approach reduces the area though which the sphere can pass without interacting with the net. This is illustrated in Figure 4.

This effect is very familiar to anyone who has ever thrown a basketball at a hoop. Despite the basketball having a diameter only half that of the hoop, its curved trajectory means that the ball rarely passes through the hoop without some contact.

The experiment reported here consisted of throwing a single sphere from each size class repeatedly at the net, with the number passing through, striking or bouncing back being recorded. This was repeated several hundred times.

The results, presented in Table 1, show that the smallest spheres achieved a no-strike rate exactly equal to theory. An increasing divergence between the experimental data and theory is apparent however as sphere diameter increases. This reflects the increasing importance of the oblique tractory for the larger spheres.

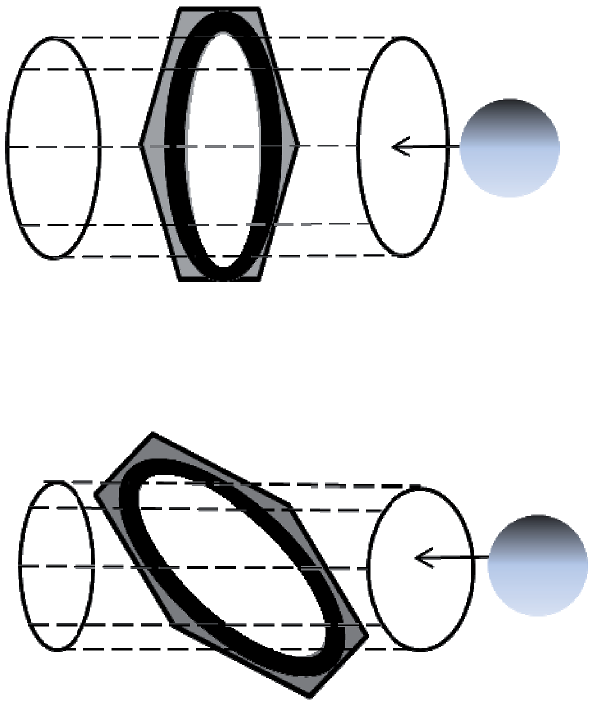


Figure 4: The effective reduction in the opening through which a sphere could travel without striking the side

Whether a sphere strikes the side of a net or not is of no importance when assessing the performance of a flexible barrier. The results of the initial tests do however validate the underlying assumptions on simple sphere-net interaction.

Case 2: Passage with, or without, striking the Net

From a theoretical standpoint, it is assumed that a sphere will pass through the net provided that the centre of the sphere is on the inside of the ring (i.e. Sphere "B" in Figure 3). The probability of this occurring (Pb) is therefore independent of the sphere size and reflects only the relative size of the central void to the enclosing hexagon. Although the geometry of the net indicates that this probability should be 66%, the oblique and curved trajectory of the spheres would be expected to result in a somewhat lower pass rate. What this reduced rate or probability is likely to be has not been assessed.

The results (Table 2) are consistant with the theory described above, with typically close to 60% of the spheres passing through. In addition, it appears that the probability of passage is indeed independent of sphere diameter. Although the results have a sound theoretical basis, it does seem somewhat counter intuitive that a sphere that almost fills the centre of the ring has as much chance of passing through as a sphere only a fraction of its size.

Sphere Diameter (mm)	Probability of No Contact	
	Theoretical (%)	Experimental Data (%)
4.5	48	48
7	39	35
11	27	24
15	18	1
22	6	0

Table 1: Probability of a sphere passing through the net without contacting a ring

Multiple Sphere – Net Interaction

With a success rate in passing through the net of approximately 60%, single spheres clearly do not demonstrate the retentive behaviour observed in flexible barriers. It is assumed that the key to barrier performance is the complex interaction of multiple spheres at the front of the net. In order to assess this effect, a series of experiments were undertaken in which dry "flows" of different sizes were directed towards the net. All tests were undertaken using the 15mm diameter spheres i.e. the ring diameter was approximately 2D.

Sphere Diameter (mm)	Probability of Passage (Pb%)	
	Theory	Experimental Data
4.5	66	59
7	66	54
11	66	61
15	66	56
22	66	59

Table 2: Probability of a single sphere passing through the net

The results (Figure 5) show that if the "flow" consists of 1, 2 or 4 spheres, the observed rate of passage through the net is the same as that recorded for a single sphere of this size i.e. the interaction effect was negligable. However, once the number of spheres within the flow increases beyond this level, there is a significant reduction in the proportion of the flow that passes through the net. The data indicates a log-linear relationship.

This reflects the jamming of the rings by multiple spheres trying to get through at the same time, as well as the blocking of those spheres that follow behind. It is clear from observing these experiments that whilst some of the spheres at the front of the "flow' are able to pass through, the bulk of the flow back-up behind the structure and are deposited.

When a number of different "mass flows" of uniform sphere size were modelled, this jamming effect was found, unsurprisingly, to be related directly to the diameter of the sphere (Figure 6). The difference between the proportion of the flow passing through the net compared to that of a single sphere (approximately 60%) is a measure of this jamming effect for a given size of flow. It is clear from Figure 6 that this jamming or blocking effect is negligable for the smallest spheres but becomes increasingly important as sphere diameter increases.

Multi-Diameter Flow Modelling

An attempt was made to model flows formed from spheres of different diameters (i.e. flows with a particle size distribution). The results suffered somewhat from an inability to achieve consistent grain size at the front of the flow. Nevertheless, is became clear from undertaking the experiment that having a mixture of coarse, medium and fine-grained particles signficiantly increased the ability of the flow to be retained by the net above that of a single grain size. In these experiments, the proportion of an 800 sphere flow that passed through the net was in the order of 15%. This conforms to the log-linear relationship described above and shown in Figure 5. The jamming effect is expected to be greater from natural rock materials as they have much more irregular and angular shapes than

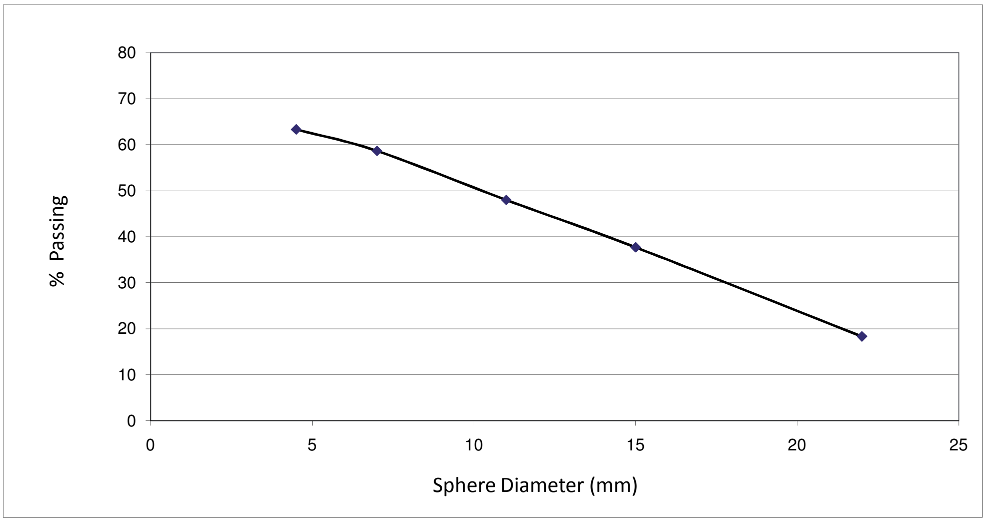


Figure 5: The relationship between the diameter of a uniform mass flow and the proportion of the flow passing through the net.

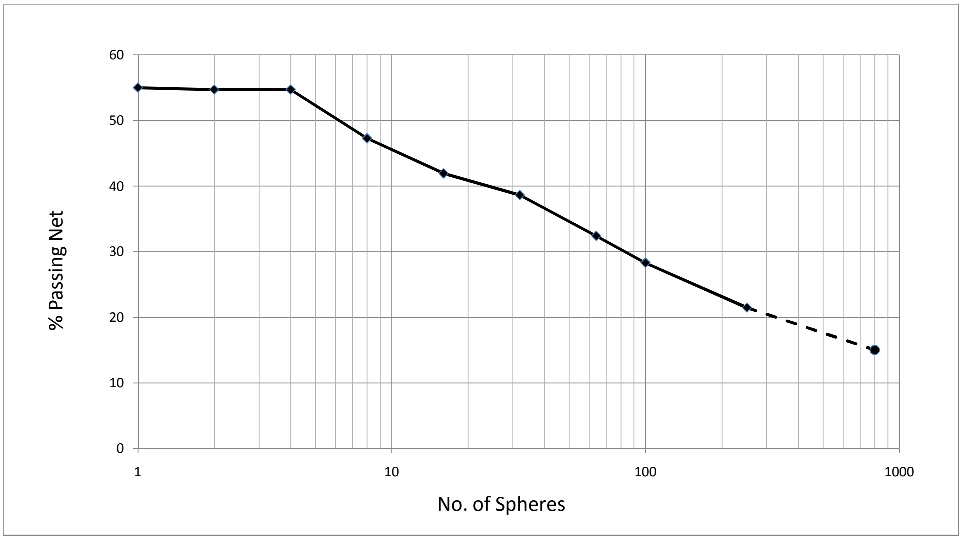


Figure 6: The apparent log-linear relationship (after a threshold value) between the number of spheres within a flow and the proportion passing through the net. The circular point is for a flow of several sphere sizes, the diamond points are for flows of a single grain size.

the spheres used in this experiment. The presence of water within a real debris flow, may on the other hand, aid the passage of material.

Conclusions

Flexible "ring net barriers" have found increasing application in recent years as debris flow control structures. This is inspite of the flexible barrier consisting predominantly of void space. A series of experiments has been undertaken using a model barrier in order to gain an understanding into the mechanism by which flexible barriers achieve their retentive characteristics.

The following conclusions have been made:

- A flexible barrier is effective at retaining debris flow material at ring diameters up to at least $2D_{90}$;
- The probability that a single particle will pass through the net is independent of its diameter;
- The ability of a flexible barrier to retain material is a result of multiple spheres attempting to pass through the rings at the same time;
- The relationship between the number of spheres within a "flow" and the proportion passing through the net appears to be log-linear;
- The tendency for barrier rings to jam or block is directly related to the size of the particles, with coarser grained flows being retained more readily than finer gained flows;
- A mixture of grain sizes appears to be the key to effective retention;
- The full impact of using dry flows of perfectly spherical material has not been determined.

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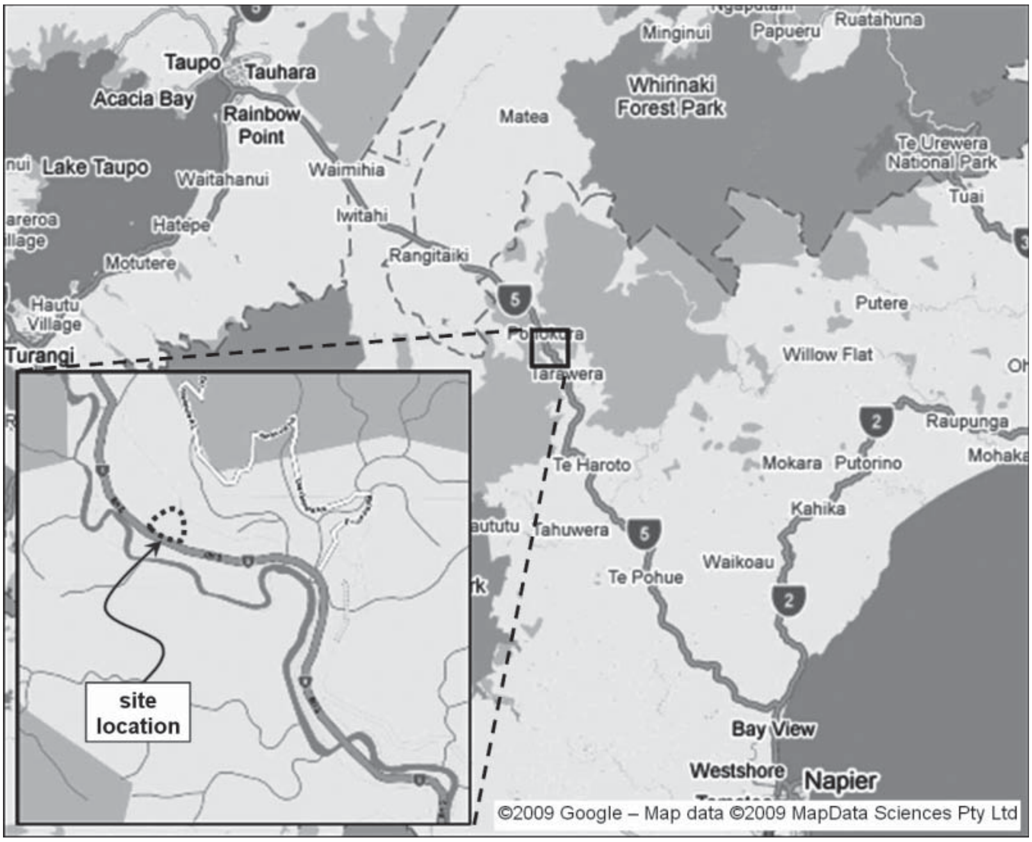
TECHNICAL ARTICLES

Tarawera Landslide – The Mechanism of a Near-Miss Tragedy

S.C.G. Dupré, Geotechnical Site Engineer, Aurecon Ltd



Map 1: Landslide location beside SH5 and the Waipunga River flowing towards Napier (Google Map)

Abstract

The Tarawera Landslide is an ancient landslide that reactivated at the beginning of 2006 following a period of heavy rainfall. It is a relatively small scale slip that has moved several times, creating road obstructions due to rock and debris falls (Varnes 1978).

A tragedy almost occurred during the remedial works of April 2006 when a secondary slip carried a 15-tonne digger across a live road at the toe of the slope.

The slope failure occurred at the contact between the Urewera Greywacke rock mass and the Taupo Pumice Alluvium, which became saturated by preceding rainfall.

Introduction

The Tarawera Landslide is located on SH5, halfway between Taupo and Napier, in the centre of the Ahimanawa Range. The slope that failed is bounded to the South by SH5, which runs parallel to the Waipunga River (see map 1 below). The slope is around 40m in length (along the road) and has a height of approximately 35 to 40 m. The general slope profile is dipping 35-40° south.

The subject of this paper is the study of the last failure movement that occurred on the 21st April 2006 as remedial works were being undertaken. The aim of these works was to eliminate the continual slip hazards causing the road to be obstructed several times in the same month by rock and debris falls. This was undertaken using a 15-tonne excavator to remove the displaced material and then to erect a catch fence at the toe of the slope.

This paper presents the analysis of geological and geotechnical observations made by the author when working on site at the time for a different company.

Geological setting

The region of concern is cut by three main faults, Wheao, Rangitaiki and Te Whaiti, striking N-S. These mark the northern extent of the Ruahine fault, which itself is an extension of the Wellington Fault. In the area of study the line of the Wheao fault is marked by the Waipunga River which separates the Kaweka Greywacke, to the west, from the Urewera Greywacke, to the east (Geological Map - 1960).

The dark grey Urewera Greywacke present on site is overlain by the Taupo Pumice Alluvium which is mainly comprised of pumice sand and gravel with clayey lenses. The apparent line of contact between the two materials is dipping easterly as shown on Photo 1.

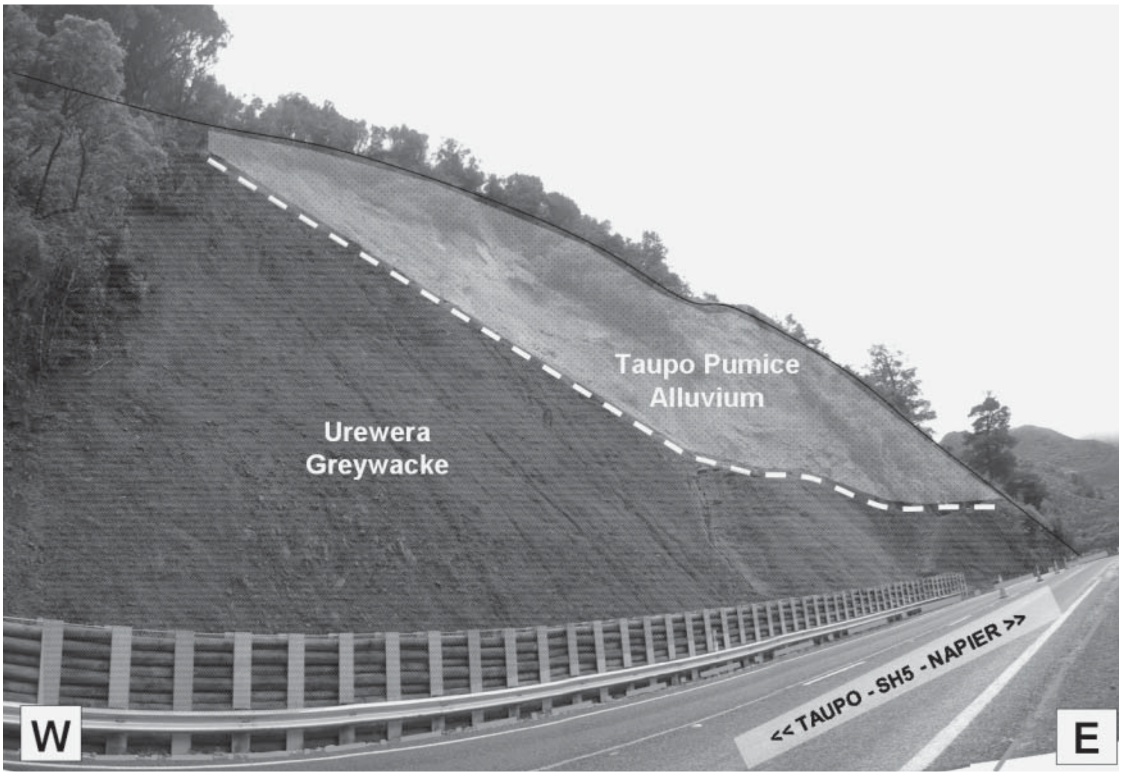


Photo 1: Apparent line of contact between Taupo Pumice Alluvium and underlying Urewera Greywacke

Site geomorphology

The observed geomorphological aspect of the valley of the Waipunga River is alluvial terraces that form obvious straight lines across the landscape. The Tarawera Landslide is located to the West of an alluvial terrace at the intersection between the rocky hills and the river, as highlighted in Photo 2.

Chronology of the Slope Failure

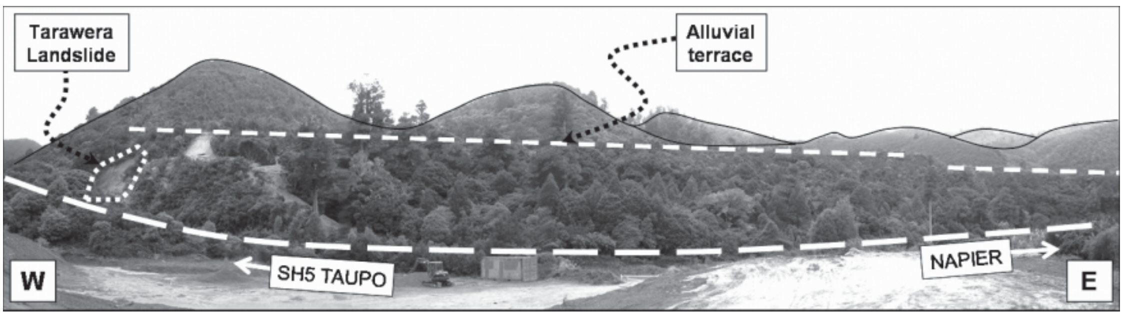
Phase 1: Initial observations

At the time of the first site visit, the road maintenance contractors explained that the slip had already moved several times requiring them to remove rock and debris falls from SH5 (landslide description after Varnes 1978, Saunders and Glassey, 2007).

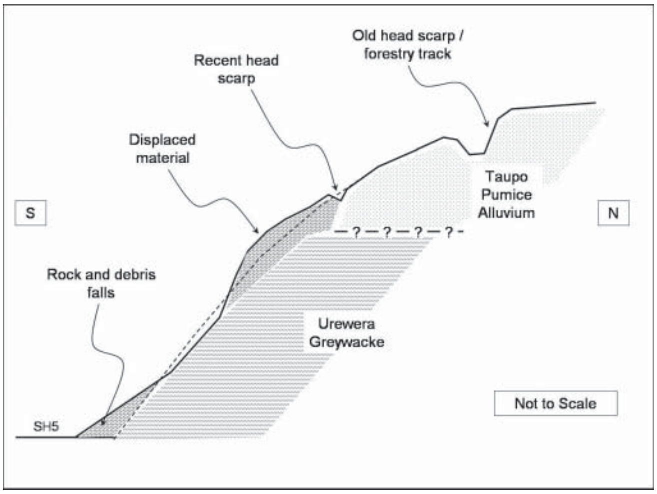
A recent head scarp was visible at the top of the slope and on the main body were some leaning and fallen trees (see Photo 3). Closer observations revealed a scarp, of approximately 1m in height, cut straight into the pumice. A recent side scarp, which disappeared approximately half way down the slope, was also noted along the eastern extent of the landslide.

Photo 3: Tarawera Landslide before remedial works. The displaced material is covered by leaning and fallen trees. A head scarp is visible at the top of the slope.

Photo 2: Location of the Tarawera Landslide and straight lines in the landscape formed by alluvial terraces



A 2-3m deep ditch was noted running along the top of the slope, the origin of this is unknown. It may be a back scarp of an old movement or an access track. A cross-section of the slope profile is shown in Sketch 1 below. The length of the ditch was approximately the same as the width of the slide. Further investigations at the eastern side of the landslide, showed lateral steps (around 1m in height) in the slope which were identified as old side scarps.



Sketch 1: Slope profile at the beginning of the remedial works

The western side of the slip was covered by vegetation that mainly comprised relatively mature trees with trunks of about 200-300mm diameter. The soil consisted of a scree type material of gravels and cobbles of greywacke. No mature trees were present on the eastern side of the landslide and the vegetation typically consisted of a dense flora. The soil type in this area was typically fine to coarse grained pumice sand. This observation indicates that it is likely that less ground movement occurred to the western side than the eastern side of the Tarawera Landslide.

Phase 2: Beginning of the remedial works

A digger was brought to site to excavate the displaced material. The excavator was positioned on a bench created about two thirds of the way up the slope and debris was removed from the top to the bottom.

Phase 3: Secondary slope failure

A secondary slope failure occurred in the pumice material above the digger bench. The plan of rupture clearly stopped against the pumice-greywacke contact (see Photo 4).

The front of the displaced material became fluid. It carried the digger down the slope, across the State Highway striking a moving car on its way. The earth flow and the digger ended in the Waipunga River approximately 40m below their initial location. (see Photo 5 below).

Photo 4: Before and after the secondary slope failure in the pumice material that carried the excavator across SH5.

Photo 5: View from the top of the slope just after the slope failure. The earth flow struck a moving car and carried the digger in the Waipunga River.

Mechanism of the Slope Failure

Unfavourable slope configuration

It is likely that the slope already moved and stabilised prior to the 2006 landslide, creating a ditch at the top and steps at the eastern side of the slip area. The different configurations between western and eastern sides were due to the different ground encountered i.e. greywacke to the west and pumice to the east.

Rainwater would have collected in the ditch, which acted as a drain, and infiltrated into the ground. The heavy rainfall that occurred prior to the most recent landslide would have produced a large volume of water seeping through the pumice soil to the top of the greywacke rock mass.

The extreme contrast in permeability between the highly permeable pumice and the relatively low permeable greywacke played a major role in increasing the pore water pressure at the bottom of the pumice layer. This phenomenon has previously been described in the eastern Bay of Plenty where similar slope failures occurred in highly permeable pyroclastic deposits underlain by low permeable alluvial silty clays or claystone/sandstone (Adhikary, 2006).

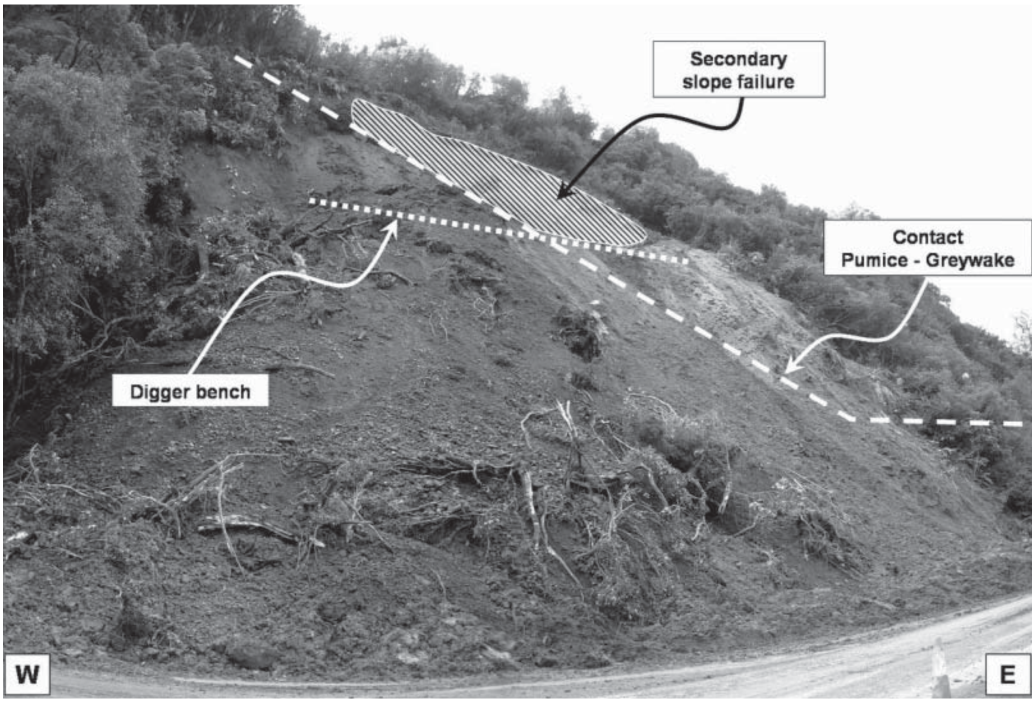


Photo 6: Bench that supported the lower part of the natural slope. The secondary slope failure took place directly above it and followed the inclined contact between greywacke and pumice.

Unfavourable material properties

The pumice sand particles are vesicular and contain a dense network of fine holes. Some of them may be interconnected and open to the surface while others may be isolated inside the particles (Wesley, 2003; Pender et al, 2006). This characteristic causes a higher void ratio than would be expected in typical sands, as the voids are not only between the particles but also inside them. It also increases the potential to retain water within the soil mass.

The grains of pumice sand are soft and have been found highly crushable even at low confining stress. The behaviour of the soil mass is therefore ruled by particle crushing (Pender et al, 2006).

Finally, when saturated, pumice sand is prone to liquefaction (Brabhaharan and Thrush, 2003). Disturbance of the material (here after slope failure) increases the pore water pressure and induces a loss of shearing resistance. The sandy material becomes fluid.

In the case of the Tarawera landslide, it is likely that:

- The excavation reached a critical angle of slope in the saturated and overweight pumice alluvium creating an unstable wedge.
- The bench created at two thirds of the slope height provided a passive support to the lower slope.
- The excess of pore water pressure along the contact of the greywake/pumice triggered the wedge failure directly above the bench. (see Photo 6 and Sketch 2).

The displaced material then liquefied causing a powerful earth flow that carried the 15-tonne excavator 40m below its original location (see Photo 7).

During the different phases of the recent slope movements, only translational slides were observed (description after Varnes 1978). This shows that the mechanism of the Tarawera Landslide was a simple type of movement that occurred in the subsurface of the slope in sensitive material i.e. pumice sand.

Conclusion

The Tarawera Landslide is a case of a simple slope failure mechanism that presented a significant risk due to its proximity to a State Highway and its road users. Anecdotal information indicates that the slope was continually moving; therefore, remedial works were required.

The secondary slope failure occurred during the works and was the result of a favourable combination of the following factors:

- The contact of two different materials of different permeabilities: the almost impermeable greywacke rock mass and the permeable alluvial pumice sand,
- The zone of contact that crossed the initial landslide diagonally from the top to the lower section of the slope,
- Heavy rainfall that triggered the movement by developing elevated pore water pressure in the permeable pumice in contact with the rock mass,
- The intrinsic characteristics of the pumice material allowing it to retain a significant volume of water, highly crushable and prone to liquefaction.

The latter characteristic was responsible for the rather powerful earth flow that endangered the site workers and the road users.

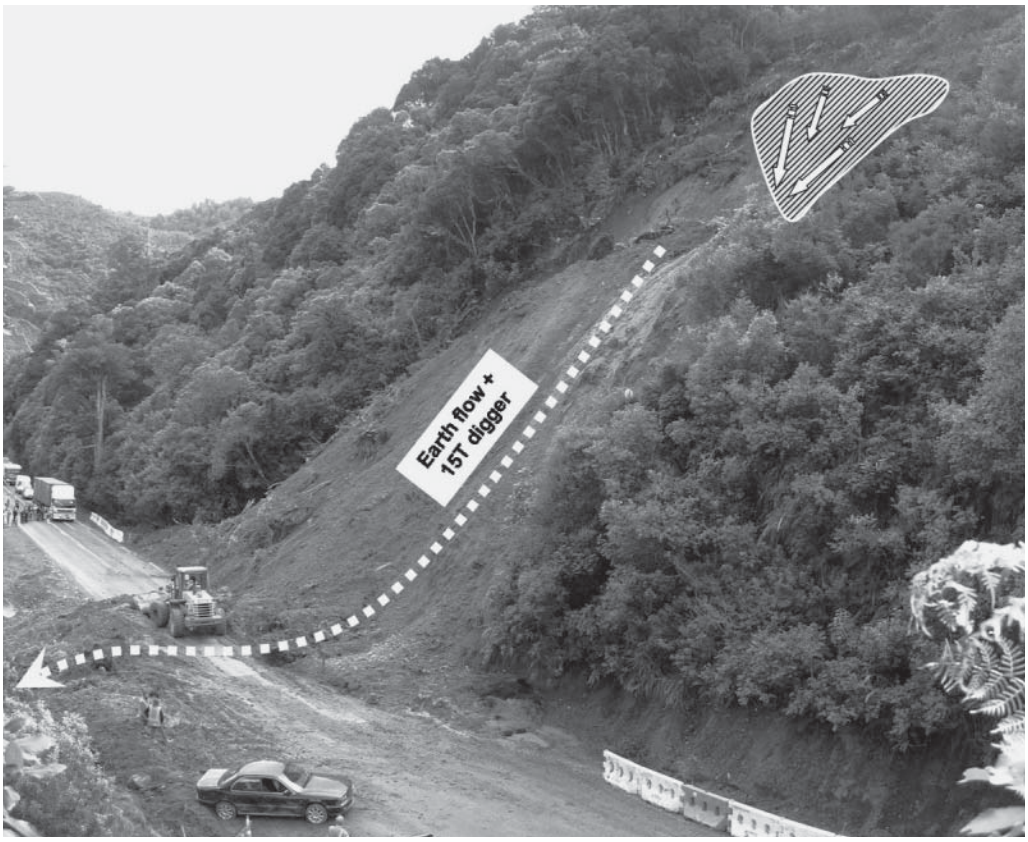


Photo 7: Direction of the earth flow that carried the excavator down to the river and struck the car in the foreground.

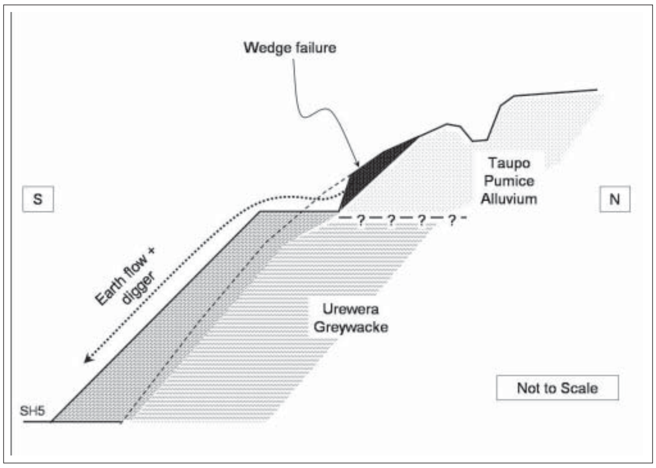
Important Information

The secondary landslide only resulted in minor injuries to the driver of the passing car.
The digger was not occupied at the time of the slope movement.

Acknowledgements

The author wishes to thank New Zealand Transport Agency (particularly Mr Gordon Hart) for their permission to publish this paper.

I offer my sincere appreciation to Mr Neil Hopkins for going through the manuscript.



Sketch 2: Slope profile at the time of the secondary slope failure.

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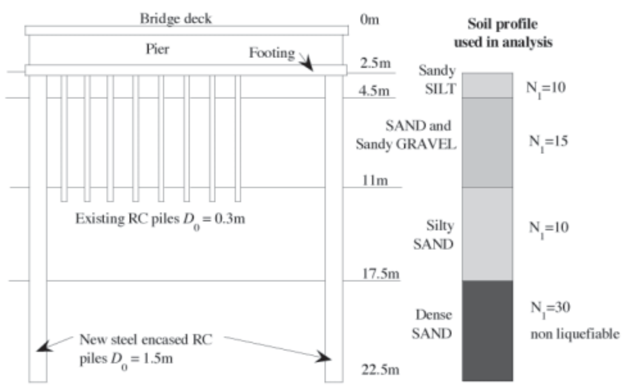
Pile foundations in liquefiable soil – A case study of a bridge foundation – Hayden Bowen, Tonkin & Taylor Ltd, Christchurch, New Zealand

1 Introduction

The loading of piles in liquefiable soils during earthquake shaking is a complex phenomenon involving interaction between the soil, pile and superstructure. During intense ground shaking and liquefaction of saturated sandy soils, rapid changes occur in the soil stiffness and strength, and the pile is subject to large ground deformations and inertial loads from the superstructure.

Based on observations from damage to piles in previous earthquakes, two phases in the seismic response of piles in liquefied soil have been recognized: firstly a cyclic phase during the ground shaking and development of liquefaction, and secondly lateral spreading following the liquefaction. The soil-pile interaction in the cyclic phase is characterized by dynamic loads on the pile from both ground movements and inertial loads from the superstructure. Lateral spreading can develop where driving shear stresses exist in the ground, such as sloping ground, riverbanks or backfills behind quay walls, and primarily occurs after the liquefaction has developed. It is characterized by large unilateral ground displacements and relatively small inertial effects. During this phase the stiffness and strength of liquefied soils are very low, reflecting features of a completely liquefied soil.

This paper describes an assessment of the seismic response of the Fitzgerald Avenue Twin Bridges over the Avon River in Christchurch, New Zealand. In conjunction with the University of Canterbury, Tonkin & Taylor has considered the two phases in the earthquake response; the cyclic phase during intense ground shaking, and the lateral spreading of soil toward the river. For the purposes of this paper only the response of the midspan piles during the cyclic phase, calculated using both a simplified design orientated analysis and an advanced dynamic finite element analysis, is presented. Analysis of the abutment piles response to lateral spreading is presented elsewhere (Bowen and Cubrinovski 2008a).



2 Fitzgerald Avenue bridges

The Fitzgerald Avenue Twin Bridges have been identified as an important lifeline for post-disaster emergency services and recovery operations. To avoid structural failure of the foundations or loss of function of the bridge in an anticipated strong earthquake affecting the Canterbury region, a structural retrofit has been proposed by the Christchurch City Council. In conjunction with bridge widening, this retrofit involves strengthening of the foundation with new large diameter bored piles. These piles have been designed to take the full gravity and seismic loads of the widened bridge. A cross section at the mid span of one of the bridges is shown in Figure 1 where both existing piles and new piles are shown. The existing bridges are supported by piled abutments on the banks and with a central piled pier at the mid-span. The existing piles are founded on potentially liquefiable soils, at about 11 m depth below the ground surface. The new retrofit piles will be connected rigidly to the existing foundation and superstructure at each end of the pile cap, and founded into deeper strata consisting of non-liquefiable soils, as shown in Figure 1.

3 Pseudo-static analysis

3.1 Model

The analytical model used in the pseudo-static analysis consists of a beam connected to a series of springs representing the lateral stiffness of the soil. The effects of liquefaction on the soil are accounted for by degrading the stiffness of the soil springs in the liquefied layers. The complex dynamic forces applied to piles in liquefied soil are approximated by the sum of two static loads applied to the pile:

(1) Kinematic loads from the soil movement are applied through free field ground displacements acting on a series of soil springs. This displacement represents the maximum cyclic ground displacements due to soil liquefaction and ground shaking. Note that this is a free field ground displacement unaffected by the presence or response of the pile foundation.

(2) Inertial loads from the superstructure are modeled as a lateral point load applied at the pile head. It was adopted in the analysis that this load acts in the same direction as the applied ground displacement.

3.2 Parametric Study

Cubrinovski and Ishihara (2004) identified the following key parameters affecting the pile response in liquefied soils:

- The stiffness degradation factor due to liquefaction, β
- The ultimate pressure exerted by the crust layer (if present)
- The magnitude of the lateral ground displacement, U_G
- The inertial load applied at the pile head from the superstructure

When designing piles to account for liquefaction, the selection of these parameters is very difficult as nearly all of them are subject to large variation throughout the course of pore pressure build up and eventual liquefaction. These intrinsic uncertainties associated with piles in liquefiable ground are directly reflected on these key parameters. Therefore for the analysis of the Fitzgerald Avenue Bridges these parameters were not uniquely determined; rather a range of values were considered. Full details of the parametric study are given in Bowen and Cubrinovski (2008a), where all of the above parameters are addressed. Here only the first parameter, the stiffness degradation due to liquefaction, is addressed.

The stiffness degradation of the soil due to liquefaction is quantified by multiplying the lateral stiffness of the soil by a constant β. Based on case studies and experimental tests, the stiffness degradation of liquefied soils can be assumed to vary between $\beta = 1/10$ and 1/50 during the cyclic phase. Figure 2 shows the results two analysis cases with $\beta = 1/20$ and $\beta = 1/50$. Both cases were analyzed assuming the soil profile shown in Figure 1 and external loads consisting of the free field ground displacement shown in Figure 2b and an inertial load at the pile head. The free field ground displacement was determined using the method of Tokimatsu and Asaka (1998) assuming a peak ground acceleration of 0.4g. The magnitude of the inertial load at the pile head was calculated as the tributary bridge mass supported the pile multiplied by the peak ground acceleration.

Figure 2 shows that for both cases the maximum bending moments occur at the pile head and at the interface between the liquefied and non-liquefied soil layers. With regard to the pile displacements the $\beta = 1/50$ case demonstrates stiffer pile behavior, where relatively stiff pile resists the movement of the surrounding ground. For the $\beta = 1/20$ case the liquefied soil is softer, the pile shows more flexible behavior and the pile displacement and bending moment is higher than the $\beta = 1/50$ case.

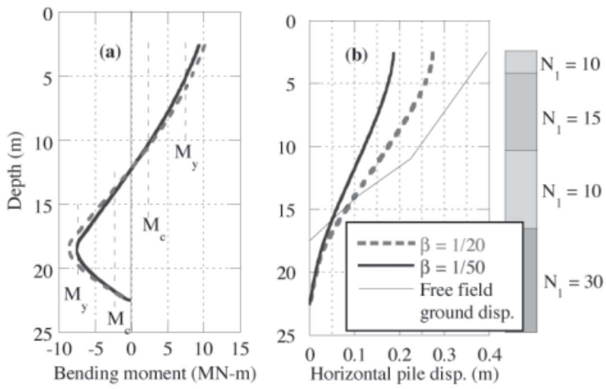


Figure 2: Variation of pile response due to liquefied layer soil stiffness; (a) pile bending moment, (b) pile displacement

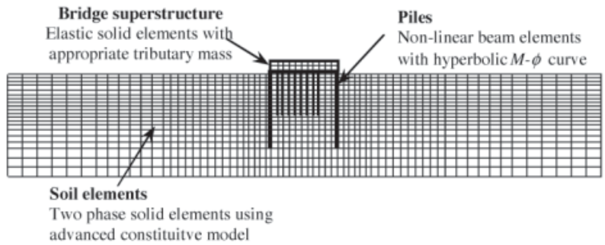


Figure 3: Numerical model used in the analysis

4 Effective stress analysis

4.1 Model

A fully coupled effective stress method was used to analyze the soil-pile-bridge system. This is an advanced analysis that permits consideration of excess pore water pressure, flow of pore water through the soil and detailed modeling of the stress-strain behavior of soils.

Conventional finite element modeling techniques for liquefaction problems model the foundation system using post-liquefaction soil properties, liquefaction induced ground displacements and inertial loads from the superstructure that have been determined using very approximate empirical correlations. In contrast, effective stress analysis, based on a detailed model of the initial state of the soil and foundation, calculates these parameters throughout the course of earthquake shaking.

The accuracy of the analysis has been extensively verified through case studies and large-scale shake table tests. The 2-D numerical model used in this study is 160m x 30m in size and is shown in Figure 3. Solid elements are employed for the soil and bridge superstructure, while beam elements are used for the piles and footing. The piles were modeled as non-linear members with a moment-curvature relationship approximated using the hyperbolic model. The footing, bridge deck and pier were all modeled as elastic materials with an appropriate tributary mass. The soil elements were modeled as two phase solid elements using an advanced constitutive model, which uses an elasto-plastic deformation law for sandy soils. Further details are provided in Bowen and Cubrinovski (2008b).

The finite-element model was subjected to a base input motion with similar general attributes to those relevant for the seismic hazard of Christchurch. An acceleration record obtained during the 1995 Kobe earthquake (M=7.2) was used as an input motion in the effective stress analysis; this motion was recorded 50km away from the epicenter in a down-hole array at a depth of 25m. The motion was scaled to have a peak acceleration of 0.4g. Needless to say, the adopted input motion is neither representative for the source mechanism nor for the path effects specific to Canterbury, but rather it was considered a relevant input motion that represents the general features of the earthquake event considered in this study.

4.2 Ground and Pile Response

First we will examine the computed ground response in the free field, which is not affected by the presence of the pile foundation. Figure 4a shows computed time histories of excess pore water pressure at two different depths throughout the soil profile in the free field. Here z = 9.5 m and z = 15.7 m depths correspond to the top parts of the second layer (N_1=15) and the third layer (N_1=10) respectively.

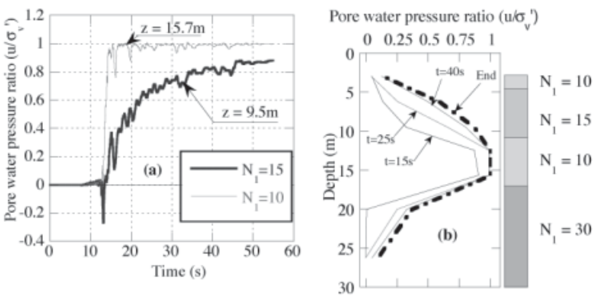


Figure 4: Excess pore water pressures in the free field; (a) time histories showing the development of pore pressure and eventual liquefaction at different depths, (b) distribution of excess pore water pressure ratio throughout the depth of the profile at different times

In the weaker layer (N_1=10), liquefaction occurs straight after the first cycle of strong shaking, as indicated by the rapid increase in the pore water pressure ratio (u/σ'_v) in Figure 4a. In the stronger layer (N_1=15), however, the excess pore pressures build up gradually with the application of cyclic shear stresses and liquefaction does not fully develop; the pore pressure ratio reaches a value of about 0.88 at the end of the shaking (t = 55 seconds). Note that here a pore pressure ratio of unity indicates complete liquefaction. Figure 4b illustrates the development of the excess pore pressure throughout the depth of the deposit by depicting snapshots of the pore pressure ratio values and hence the extent of liquefaction at different stages of shaking or time sections. This plot shows that the looser sand layer (N_1=10) completely liquefied whereas in the denser sand layer (N_1=15) the peak pore pressure ratio was in the range between 0.4 and 0.9 at the end of the shaking.

Effects of liquefaction on the ground response are evident in where acceleration time histories at three different depths are shown in Figure 5. Following the complete liquefaction in the mid layer at about 13-14 seconds, the accelerations above the liquefied layer decrease significantly and the ground motion shows elongation of the vibration period and loss of high frequencies. This diminished ground shaking and consequent reduction in the shear stress can explain the slower gradual build-up of the excess pore water pressure in the layers above the liquefied layer. This trend is continued to the response at the ground surface; the response of the weaker layer controls the response of the layers above.

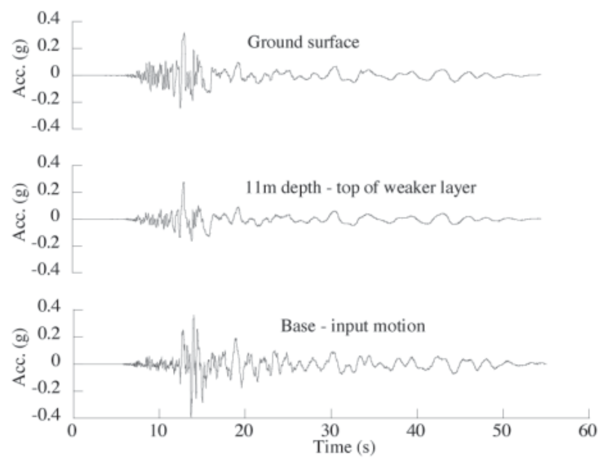


Figure 5: Free field acceleration time histories for different depths of the soil profile

The maximum values of the ground acceleration, shear strain and ground displacement, plotted in Figure 6, clearly display the effects of liquefaction on the free field ground response. Figure 6a shows a decrease in acceleration above the weaker layer that liquefied rapidly; this phenomenon has been observed in down-hole array records during past earthquakes and in many shake-table experiments on scaled-down models of relatively loose sands that do not exhibit cyclic mobility. Figure 6b and Figure 6c show that the majority of the ground deformation occurs in the mid layer with N_1=10, where the peak shear strains reach about 4%. The strains in the shallow part of the deposit are well below 1%, which is consistent with the lower excess pore water pressures generated in these layers. Clearly, the response of the weaker layer significantly affects and, in this case, practically governs the response of the layers above.

In general terms, the pile foundation provides a stiffening effect to the surrounding foundation soil. This effect is apparent in Figure 7, where time histories of horizontal ground displacements in the foundation soil and free field are compared. The peak horizontal displacement in the free field reaches 0.3 m whereas the peak displacement of the foundation soil is less than 0.2 m.

A time history of the bending moment at the pile head is presented in Figure 8, showing that throughout the most intense part of the shaking the pile response is well above the cracking moment (M_c) of the pile. The yielding moment (M_y) of the pile is approached a number of times throughout the shaking and is exceeded at approximately t = 19 seconds for the particular input motion used in this study.

5 Conclusions

The seismic response of the Fitzgerald Avenue Bridges, including the effects of liquefaction and lateral spreading, was analyzed using both an advanced effective stress analysis and a simplified pseudo-static analysis. The two approaches focus on different issues and play complimentary roles in providing a comprehensive assessment of the bridge foundation response to a major earthquake.

Figure 6: Maximum free field response: (a) accelerations, (b) shear strains, (c) ground displacements

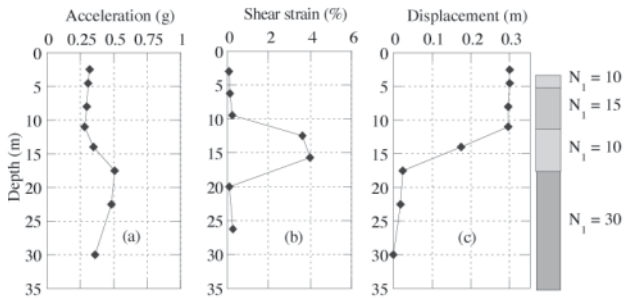
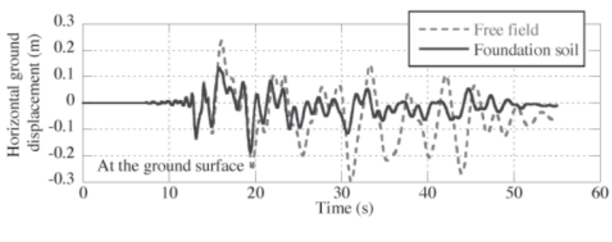


Figure 7: Comparison of the horizontal displacement at the ground surface for locations in the free field and in between the piles



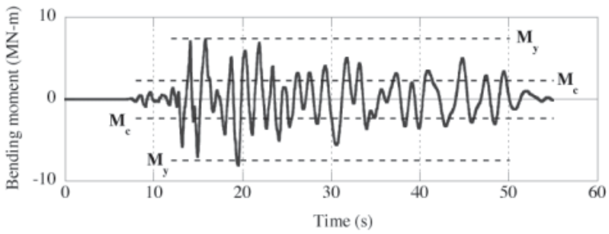


Figure 8: Bending moment time history at the pile head

Due to its simple and design orientated nature, the pseudo-static analysis is well suited to conduct parametric analyses in order to account for the significant uncertainties inherent in liquefaction problems. For the response of the midspan piles to the cyclic phase of liquefaction, it was shown that the liquefied soil stiffness has a large bearing on the analysis result. Thus for design of piles in liquefiable soils, it is essential to consider the large aleatoric uncertainties present. Parametric studies using pseudo-static analysis are therefore recommended in practice.

This case study demonstrated the capability of the effective stress analysis to capture important features of the soil-pile interaction in liquefying soils, including:

- Detailed development of excess pore water pressure through time and space including effects of soil density and complex interaction between intensity of shaking, pore pressures and associated ground deformation.
- The soil-pile interaction, which significantly affected both the response of the foundation soil and piles. The presence of piles increased the stiffness of the foundation soil and consequently reduced its deformability as compared to the free field ground.
- A rigorous evaluation of pile performance, determined by taking into account the highly complex dynamic nature of loads and soil-pile interaction.

Therefore this analysis methodology can provide a rigorous evaluation of the seismic performance of pile foundations of important structures. It can explain complex features of the response and verify design assumptions, and hence, it provides a unique contribution in the assessment and design of piles.

Acknowledgements

Financial assistance from the New Zealand Earthquake Commission and the New Zealand Geotechnical Society is greatly appreciated.

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Geotechnical aspects of the Upper Harbour Bridge duplication and causeway widening

– L. Coe, Beca Infrastructure Ltd, Auckland, New Zealand

1 Introduction

The Upper Harbour Bridge (UHB) Duplication and Causeway Widening Project was constructed for Transit New Zealand between 2003 and 2006. The widening of State Highway 18 (SH18) between Greenhithe and Hobsonville forms part of the proposed motorway linking Albany Highway with SH16 in Auckland, New Zealand. The project works have been completed under a Design and Build contract with Fletcher Construction (Fletcher) as the Contractor and Beca Carter, Hollings and Ferner Ltd (Beca) as the Designer.

This project consists of two main elements: construction of a second Upper Waitemata Harbour crossing, adjacent to the existing Greenhithe Bridge, and widening of the existing causeway to form the western approach for the new crossing. The general layout of the site is shown in Figure 1.

The new bridge provides three east bound lanes, including a passing lane, and a designated cycle and pedestrian lane.

The existing 850m long causeway has been widened by up to 20m over weak marine sediments and alluvial deposits. The widening has been constructed using cement stabilized marine sediments (Mudcrete) to form a shear key and outer facing, combined with rock and sand fill confined within the embankment. The construction below water level was an alternative design proposed by Fletcher and Beca at the time of tender.

2 Site Description

2.1 Setting

The site is located on SH18 between Greenhithe and Hobsonville, crossing the Upper Waitemata Harbour to the north of the existing bridge and causeway. The bridge crosses an incised inlet of the Waitemata Harbour formed following a post-glacial rise in sea level.

The approach causeway is situated in a sensitive coastal environment within the inter-tidal zone and has been extended over areas of mangroves and mudflats. The seabed slopes gently out from the existing causeway at approximately 5 degrees. The new causeway is a maximum height of 5m above the seabed at the western abutment where the seabed dips steeply towards the channel. The bridge spans across the navigation channel with average water depths of 8 to 10m.

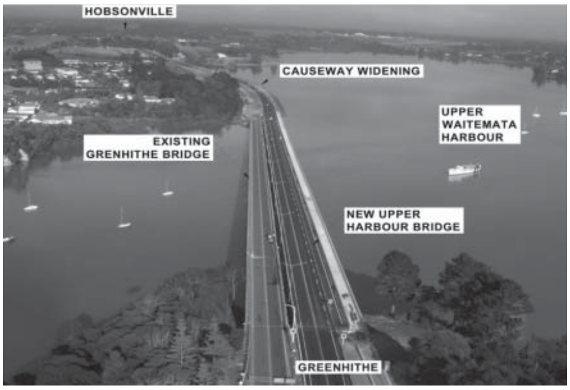

Figure 1: Site layout.

2.2 Soil profile

The soil profile typically comprises of recent marine sediments and Tauranga Group alluvium, overlying a weathered Waitemata Group profile ranging from residual soils to slightly weathered siltstone and sandstone.

The existing causeway is comprised of silt and clay fill of variable strength ranging from a soft fill directly over the original seabed to very stiff close to the pavement. The recent marine and estuarine sediments (Holocene aged) are a combination of very soft silts and clays, and loose sands. These sediments are up to 2m thick and are located on either side of the causeway and under the existing footprint. The Tauranga Group alluvium (Pleistocene aged) is up to 3m thick and consists of firm to stiff clays and silts, and loose to medium dense sands. The underlying Waitemata Group weathered profile consists of up to 4.5m of residual soils over slightly weathered rock. The residual soils are stiff to very stiff silts and clays with medium dense to dense silty sands. The very weak to weak siltstone and sandstone lies approximately 1.5 to 5.0m below the seabed along the causeway alignment.

3 Geotechnical Design of the Causeway Widening

The design of the causeway widening adopted for construction is based on an alternative design proposed by Fletcher and Beca at the time of tender. The alternative design utilises Mudcrete and locally available materials to replace large quantities of rock fill and large boulders required in the conforming design. The design was developed to suit construction methods and improve design standards.

The causeway widening design involved construction of a shear key into underlying stiff materials, a rock fill bund with a Mudcrete outer facing and sand fill at the rear of the widening. Geotextile and armour rock were placed on the seaward face of the widening for erosion protection. A typical cross-section for the causeway widening is presented in Figure 2.

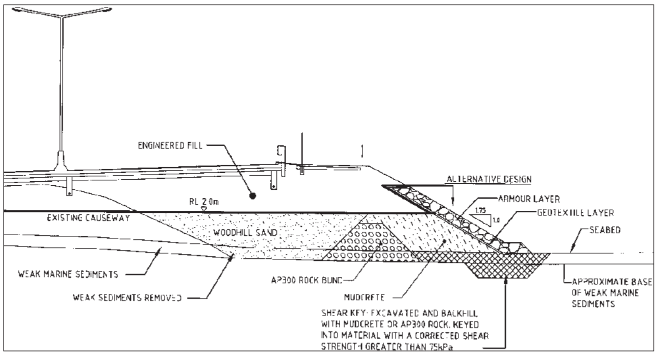

Figure 2: Typical causeway widening cross-section.

3.1 Conforming design comparison

The conforming design proposed utilising granular rock fill below RL2.0m and cohesive fill above this level. The design involved two treatment concepts depending on the depth of marine sediments encountered as detailed below. For marine sediments less than 1.0m thick, large boulders greater than 1.0m were to be punched into the marine sediments and AP300 rock fill was to be placed above this level up to RL2.0m. For marine sediments of thicknesses greater than 1.0m, the sediments were to be excavated and replaced with AP300 granular fill to RL2.0m. Both treatments were combined with a shear key constructed using rock fill and keyed into the underlying Tauranga alluvium and Waitemata Group residual soils.

The alternative design philosophy was considered to have the following advantages over the conforming design: the requirement for large boulders that are relatively scarce in the Auckland region is eliminated, the quantity of AP300 rock fill required below the water level is reduced, all marine sediments excavated are re-used in causeway construction eliminating the requirements associated with transportation and disposal of contaminated sediments offsite, and the use of Mudcrete increases the stability and reduces the risk of long-term settlements of the causeway. In addition steeper side slopes were used, therefore minimising the causeway footprint.

3.2 Key design and construction concepts

The key design issues for the causeway widening, with respect to the presence of weak marine and alluvial deposits, were:

- The stability of the causeway, including performance under static and seismic load cases;
- The potential for long-term settlement of the causeway and the differential movement relative to the existing causeway and new bridge structure, including the impact on services, ride quality and long-term maintenance requirements; and
- The constructability of the causeway widening.

The design principles adopted to address these issues are described below.

3.2.1 Removal of soft marine sediments

All soft to very soft marine sediments with a corrected shear strength of less than 25kPa were removed from beneath the widening footprint. These sediments were removed to improve stability of the causeway and reduce consolidation settlements.

The thickness of these very soft sediments varied from the eastern end of the causeway alignment, where the underlying siltstone and sandstone was exposed in a wave cut platform, to over 3.0m thick within an infilled channel running across the causeway alignment. Generally the very soft sediments were encountered up to 1.0m below the seabed and there was a marked increase in strength between the recent sediments and underlying soil layers.

3.2.2 Granular fill below the water table

Granular fill was used below the water table to enable compaction requirements to be met. This included the use of AP300 quarried rock to construct a bund behind the Mudcrete face and Woodhill sand, a fine uniformly graded sand, to fill between the rock bund and existing causeway. Limiting the quantity of rock fill and utilising locally sourced sand reduced the cost of materials and transportation. The rock bund is required for the stability of the causeway, and was also utilised by the Contractor as a working platform during construction. The sand is confined between the existing causeway and the rock bund with a Mudcrete facing to prevent migration and potential piping failures. The sand was compacted to a medium dense standard to reduce the potential liquefaction risk under the 1000 year return period earthquake event. The compaction standard was generally achieved by track rolling and tidal action and confirmed using Scala Penetrometer Tests.

3.2.3 Shear key

A shear key was excavated at the toe of the widening and backfilled with sections of Mudcrete and AP300 rock fill. The shear key extends at least 0.5m into the underlying stiff alluvial deposits and residual soils with a minimum corrected shear strength of 75kPa. The shear key assists with the overall stability of the causeway widening and allows minimum stability design standards to be achieved. The design provided flexibility to the Contractor by allowing Mudcrete and rock fill to be generally interchangeable for the shear key backfill. Rock fill material was utilised when the marine sediments for Mudcrete were not available.

3.2.4 Mudcrete

Mudcrete was used as the outer facing of the causeway bund and within the shear key. Cement was mixed onsite with excavated silts, clays and sands to form Mudcrete, a relatively lightweight, high strength, cohesive fill product that does not require compaction. The strength of Mudcrete improves the stability of the causeway and enabled the face to be constructed at a steeper angle reducing the causeway widening footprint. The lightweight nature of the fill is expected to reduce the long-term consolidation settlements of the causeway.

The marine sediments are contaminated with heavy metals from the harbour and untreated stormwater run off from the existing causeway. The use of Mudcrete eliminated the requirements for storage, transportation and disposal of these contaminated, saturated sediments. Cement stabilisation binds contaminants such as DDT, lead, zinc, organics and TP4 into the product. As such the use of Mudcrete is considered an environmental enhancement for this sensitive coastal setting.

In addition, Mudcrete is a relatively low cost fill material when compared to crushed aggregate.

3.2.5 Construction program

The construction program was developed to reduce the post-construction settlements. The causeway embankment adjacent to the new western bridge abutment was constructed early to allow the majority of settlement to occur prior to construction of the abutment structure and piles. The paving along the causeway was delayed until the end of the project to allow the causeway to be re-levelled and reduce the long-term settlement effects on ride quality and ongoing maintenance.

3.3 Design stability analyses

The causeway was analysed at critical locations along the alignment based on a combination of the fill height and underlying soil conditions. The load cases modelled and minimum factors of safety (FOS) against instability adopted for design are summarised in Table 1.

Table 1: Design load cases and required factors of safety.

Design case	Analyses type	Loading	Target FOS
Short-term construction	Total stress parameters	Construction traffic loads	1.3
Long-term static	Effective stress parameters	Transit New Zealand traffic loads	1.5
Seismic	Total stress parameters	Seismic acceleration of 0.11g	1.1

For the design Mudcrete has been conservatively modelled with a shear strength of 150kPa. The Mudcrete field strengths are described in Section 4.

The seismic acceleration was based on the 1000 year return period earthquake event determined from a site specific hazard assessment and was modelled using a pseudo-static approach.

4 Mudcrete

Mudcrete is a relatively high strength, lightweight fill material produced by stabilising weak marine sediments with cement. The variability of the product is determined by both the construction methodology and variability of sediments (i.e. sand, silt and clay contents). The field mixing techniques utilised for the causeway produced highly variable strength results, however a minimum strength was specified and achieved as a routine (albeit with much stronger zones).

Beca originally developed Mudcrete for the reclamation constructed at the Auckland Viaduct for the Whitbread Round the World Race in 1994. Mudcrete has subsequently been utilised for the Americas Cup Village extension to the Auckland Viaduct and is currently being used for the Ports of Auckland Container Terminal Extension.

In addition to the causeway construction, the Mudcrete was utilised for the stormwater retention ponds designed by the Client's Engineer, temporary mixing basins for Mudcrete and to encase the double stormwater culvert that was extended under the new causeway widening.

4.1 Material characteristics

Mudcrete was utilised as a structural fill with specific requirements including a minimum Unconfined Compressive Strength (UCS) of 90kPa at 7 days, a minimum UCS of 200kPa at 28 days and an average UCS greater than 400kPa. Trial stabilisation batches were produced prior to construction to confirm the mixing technique and cement ratio.

The Mudcrete was batched with the minimum 80kg/m^3 cement ratio for the majority of the causeway widening and generally achieved all specified strength requirements. As construction progressed a 65kg/m^3 mix was trialled and also achieved the required strength in the sandier mix. The 28 day strength results from the northern causeway widening are presented in Table 2 and Table 3. Bulk density, moisture content and dry density based on both 65kg/m^3 and 80kg/m^3 cement content samples. Results based on ten 65kg/m^3 cement content samples and fifty five 80kg/m^3 cement content samples. These results indicate a large variability in Mudcrete strength produced, with the UCS ranging from 114 to 3670kPa. This variability is a result of the uncontrolled nature of the sediments and water content.

Table 2: Northern causeway Mudcrete field results.

Properties	Bulk Density t/m^3	Moisture Content %	Dry Density t/m^3
Average	1.75	37.0	1.29
Range	1.53 – 1.92	27.0 – 57.7	0.97 – 1.51
Standard Deviation	0.08	8.04	0.12

Table 3: Northern causeway Mudcrete field strength results.

Properties	28 Day UCS (1) kPa	Young's Modulus MPa
65kg/m^3 of Cement		
Average	668	273
Range	459 – 945	51 – 459
Standard Deviation	170	126
80kg/m^3 of Cement		
Average	1460	542
Range	114 – 3670	22.6 – 1370
Standard Deviation	870	322
(1) All results based on 28 day UCS samples using standard 21 degree cure.		

4.2 Construction methodology

Mudcrete was produced by mixing sediments and cement within small on-site mixing basins and insitu within the shear key trench excavation, creating 30 to 50m^3 per batch. The methodology adopted for mixing required little additional plant and resources, utilising hydraulic excavators to mix the product. The mixing basins were located in close proximity to the current work faces to improve efficiency and minimise haulage.

The Mudcrete was mixed and placed as described below.

Figure 3: Mudcrete mixing equipment.

The sediments were placed within the mixing basin or excavated key trench with the volume accurately recorded. Prior to mixing, sediments were checked for consistency and large clumps of clay were premixed or removed. The cement quantity was calculated and spread over the surface of the sediments. The cement was tilled in with a hydraulic mixing head attached to an excavator; the mixing head is shown in Figure 3. Each batch was mixed until a uniform visual consistency was achieved. Mudcrete batched in mixing basins was transported to the placing excavator and placed at low tide within an hour of batching. The cohesive nature of Mudcrete prevented it from being washed away as the tide rose. The surface of the Mudcrete was shaped using the back of the excavator bucket as a trowel. In order to achieve an adequate bond between subsequent layers, the hardened surface of the Mudcrete was roughened with a toothed excavator bucket prior to application of the next layer.

The water content of the sediments affects the strength of the final stabilised product; however it is not practical to control the water content. The construction methodology adopted the following steps for working within the tidal environment and reducing the water content of the sediments.

- No additional water was added to the Mudcrete mix.
- The sediments were excavated at low tide and stored behind a Mudcrete or sand fill outside the extent of tidal influence. If this was not possible sediments were drained at low tide prior to mixing or premixed with drier excavated materials to reduce the water.
- Mudcrete was generally placed at low tide or above the tide level to minimise the requirement to place it below water.
- The shear key constructed was generally excavated and either Mudcrete placed or sediments stored within the key over the low tide cycle to prevent water becoming trapped within the shear key and to limit the risk of side collapse. Where the design required the shear key depths to increase, AP300 rock fill or insitu mixing was adopted to enable the key to be backfilled quickly.

Key construction features of Mudcrete and the adopted on-site methodology include:

- Mudcrete does not require compaction.
- The cohesive nature of Mudcrete prevented it from being washed away as the tide rose.
- The Mudcrete generally gained enough strength for people to walk on the surface within a few hours and to traffic vehicles within one or two days, allowing construction to proceed quickly.
- Minimal control was placed on the sediments or water content, resulting in a cost-effective product.
- Minimal additional plant and resources were required to mix and place the Mudcrete.

These features resulted in a cost effective fill material that met the constructability requirements.

5 Conclusion

The UHB duplication and causeway widening project has been successfully designed by Beca and constructed by Fletcher, using an alternative causeway design philosophy. Key geotechnical issues addressed by the design include the presence of weak marine and alluvial deposits, slope stability and long-term settlements. The alternative design aimed to address these geotechnical issues and to improve constructability by utilising Mudcrete for the shear key and outer facing, combined with a rock bund and locally sourced sand fill.

The use of Mudcrete allowed excavated, very soft sediments to be re-used on site to produce a relatively lightweight, high strength fill and eliminated the requirement for disposal off site. The Mudcrete was batched on site using small temporary mixing basins with a mixing head attached to a hydraulic excavator. All minimum strength requirements were achieved in the field, however the Mudcrete strengths produced were highly variable due to the nature of the sediments.

Acknowledgements

The writer would like to thank Nicola Ridgley and Dr Do Van Toan of Beca Infrastructure Ltd for help throughout the project and the review of this paper, and Fletcher and Transit New Zealand for permission to publish this paper.

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'NZGS, Rocks are our life too' **Mike Allis** Graduate Engineer
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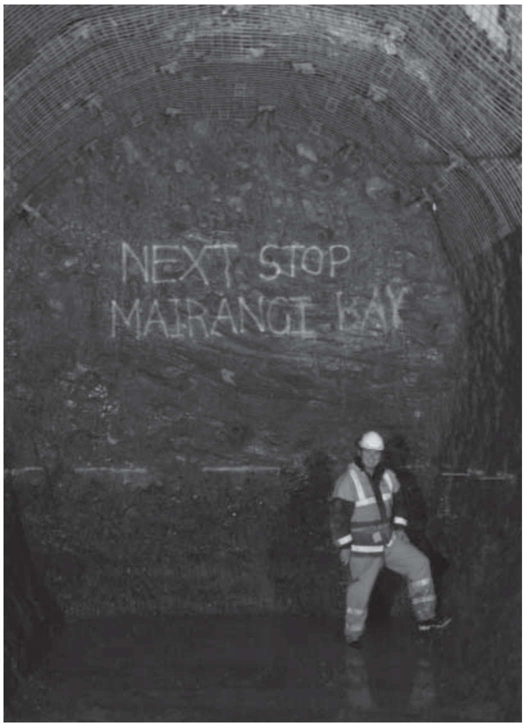

▲ HIGHLY COMMENDED

Clockwise top left: “Reflecting our work” **Christine Parkes**, *Opus International Consultants Ltd*; “Next Stop Mairangi Bay - End of the foreshunt excavation at Rosedale, North Shore” Note the classic geologist pose!!! (Struck by Mr Daniel Costello, *Aurecon*) **Adam Broadbent**, *Aurecon*; “Arriving at work in Western Australia” **Karen Jones**, *Sinclair Knight Merz*; Drilling anchors up to 6.5m deep into hard sandstone behind the Cobb Power Station, Takaka. **Martin Wilson**, *Abseil Access Ltd*; Drilling multiple 5m deep blast holes 150m up on Nevis Bluff, Queenstown, **Martin Wilson**; Investigation of reactivated ancient landslide, Manuka Gorge, New Zealand. **James Grindley**, *Opus*; “Minor services issue during the construction” **Sigfrid Dupre**, *Aurecon*.

FOREIGN CORRESPONDENT

Steve Crook

Doha, Qatar

Field Engineering Manager, Overseas Bechtel Incorporated

From a statistical point alone Qatar is an unusual country. Bordering the Persian Gulf and Saudi Arabia Qatar has a land area slightly smaller than Northland, yet it has the second largest gas reserves in the world and an enviable reserve of crude. The population of only 350,000 Nationals is served by 1.1M foreign workers (mostly construction) making the male to female ratio the most skewed in the world.

In 2003 Qatar embarked on an ambitious infrastructure programme investing billions in gas exporting facilities, industrial and commercial development, roads, ports, and a new airport. I was working in NZ when my previous employer Bechtel, asked me to fly out to Doha to work on the construction management team for a new airport. When I arrived the project was beginning to reclaim 22 km of land to form an airport platform. Initial pilling and pavement works were also underway. Since that time we have completed the reclamation and are well underway on the structures.

Working in the Gulf certainly has its challenges. The region has been through a massive boom which over the last years which has led to a severe shortage of skilled professionals. Maintaining quality from the many nationality firms working in the region is a constant struggle and there have been many times when I have looked back wistfully on the reliably high quality of engineering design produced in NZ.

On the other hand with funding generally not the primary issue and schedule paramount, design is generally conservative. Working on a large project also has its challenges – just keeping the 22,000 workers on the project safe – some who have never worked in construction before – is a major achievement.

It has been interesting to watch the development of Qatar over the last 3 years, especially through the oil and property boom that peaked in 2008. During that time it seemed that every day a new and incredibly ambitious development was announced – taller buildings, grander theme parks and residential super-communities that were difficult to imagine ever being fully populated. With salaries being talked up to compete with the skills shortage as well as tax free incomes, the place had real "gold rush" mentality. It was not uncommon to see graduate engineers driving Porsches and flying to Europe business class for their skiing holidays!

Above: Doha Airport reclamation

Reality returned in 2009 and although the pace of development is still hectic by most country's standards it no longer has the out of control feel of previous years.

Religion is a big part of life in the Middle East and anyone working here has to be comfortable with the role it plays in everyone's daily lives. Although Qatar is a moderate country there are still restrictions on alcohol and other aspects of western life (you can't have a beer in a sidewalk café for instance). Most people don't find it an issue and contrary to what you see on BBC World, the Middle East is generally very safe – theft is almost unheard of and most people don't bother locking their cars or their houses.

Overall, I would recommend the Middle East for anyone looking to spend some time working abroad. You will certainly gain exposure to engineering that goes with large, fast paced development, but perhaps not always that which goes with typical NZ projects which have a stronger focus on environmental issues and value engineering. Cost of living is something else that you need to consider carefully before signing up to work in the Middle East. Although "all in" packages can seem very attractive, especially if tax free, the cost of housing and education can be prohibitive – a modest family home in Qatar for example will set you back $ 9,000 per month. Nevertheless if you are keen and can forego your bacon and eggs on a Sunday morning (pork is not imported in much of the Middle East) then give it a go. The region could always do with the versatile skills and can do attitude that most NZ engineers have as second nature.

COMPANY PROFILES

Ice Construction Ltd

Ian Shaw and Marianne O'Halloran, who have backgrounds in construction and engineering going back to the Ministry of Works, formed Ice Construction Ltd in 1996. Ice consists of one geotechnical engineer, with leanings towards civil engineering and project management, two grey haired blokes with decades of experience in all types of civil construction and a few good keen workers. We may be small but we try to be good at what we do; hence we have ISO9001 certification, tertiary level ACC accreditation, pre-qualification with the Land Transport Agency and Ontrack, the required training to work in power generation facilities and are working towards environmental certification with Enviromark.

We try to carry out work which bigger companies are often not interested in or for which smaller companies do not have the skills. We endeavour to work with clients and engineers to achieve good results with minimal disruption to normal activities. As we have a geotechnical engineering division, i.e. Marianne, we do get involved with work with a geotechnical bias. Discussed below is the wide variety of work we carry out as some areas may be of direct or peripheral interest to geotechnical engineers.

Roped Access Work

Ice has assisted geologists or carried out our own logging of cliff faces in various places such as around the Tauranga Harbour, Bluff Hill in Napier and the 160m high Ongaroto Bluffs beside Lake Whakamaru. We have carried out many rock scaling contracts in places as diverse as the road to Lake Waikaremoana, Auckland Boys Grammar and Eden Gardens. Some of this work was carried out in conjunction with an explosives company. Using roped access we clear vegetation, carry out spraying, install rock bolts, soil nails, rock fall mesh and enviromat type products, repair services and apply shotcrete.

Roped access work particular to power generation companies includes cleaning out weep holes, fixing leaks in structures, shotcreting around penstocks, inspecting the inside and outside of penstocks, repairing dams and spillways, installing deformation monitoring points and helping with the monitoring of dams such as the beautiful Moawhanga arch dam. One unusual job involved the replacement of hangars on the old high level suspension bridge at Arapuni.

Work around Power Generation Facilities

Work around power generation facilities requires specific safety training and procedures. We have built foundations for new transformers, bunding around existing transformers, blast walls between transformers, sealed leaks in all sorts of places, repaired spillways below water level, repaired stoplogs, realigned gate rails and replaced joints.

One of our biggest projects has been the stopping of leaks and the lining of the invert of the 1km long Waihohonu Tunnel which diverts water from a tributary into the Tongariro River upstream of the Rangipo Intake. The invert of the tunnel had been scoured out by gravel coming down the tunnel, almost down to the formwork in places. We had to build a cofferdam to prevent flow coming up the tunnel from the river and clean out the tunnel before installing the fibre reinforced concrete liner. The work was carried out with forced ventilation. In spite of having one flood that came up the tunnel, we managed to finish this contract early and without releasing anything nasty into the famous trout fishing river.

At the Poutu Intake the concrete sills had taken a beating from gravel coming down the river. We won a contract to fix curved steel plates over the four separate sills and grout the gap between the damaged concrete and the plates; all while the Tongariro River roared about us. Once again, due to the dedication of our workers, we finished early.

Above: Completed steel sill at Poutu

Last year we strengthened the abutments and installed some pneumatic gates across the spillway of the Mokauiti Dam for King Country Energy. This was an interesting project requiring some quite fine tolerance work. The steel gates are designed to be able to be raised and lowered by changing the air pressure in the large rubber bladders supporting them. They are also designed to flex to enable debris to pass over the spillway. This project has significantly increased the storage capacity of the dam.

Above: The almost completed Mokauiti Spillway Gates

Bridge and Wharf Strengthening and Repair

We have repaired and strengthened many concrete, steel and timber bridges and wharves throughout the North Island. Much of the work is routine concrete repair and we have recently purchased an ultra-high pressure water blaster to remove defective concrete without creating micro-cracks, and a gunite sprayer to speed up the application of the repair compounds and reduce wastage. On concrete bridges we also carry out epoxy crack injection and replace joints. We have jacked up bridges to replace bearings, re-built hardwood railway bridges and strengthened piles and piers with grouted steel casings and fibreglass "Towsheet". We have a 9m work boat, small enough to go on a road trailer but with its own loading ramp, to provide access on any inland waterway or harbour.

Some of our bridge work, such as seismic strengthening, requires heavy steel components and bolting. One of our fine tolerance projects was the installation of new steel tensioned hangars between the arches and the deck of the Karapiro Low Level Bridge to allow heavy loads to cross to the power station. Another challenging project was the installation of precast beams transversely under the Aerodrome Bridge in Tauranga. These beams, plus high strength bars installed in grooves cut into the deck, tie together the longitudinal deck planks that were moving.

Walking Tracks and Footbridges

Our roped access work has given us an inclination to work in difficult to get at places. We have therefore developed a bit of a specialty in designing, building and upgrading walking tracks and their associated structures. We have acquired some very small walk behind dumpers for narrow tracks (0.3m^3 capacity) and a 1 tonne excavator that can be lifted in and out by helicopter. We have also some large dumpers (1m^3 capacity) which are presently transporting rock rip rap around the Mauao (Mt Maunganui) base track for wave protection. In addition to general track shaping and drainage work we build boxed steps, boardwalks, stairs and bridges. Our clients often take us to look at a well worn track and leave us to sort out how to upgrade it in accordance with the Track and Outdoor Visitor Structures handbook (SNZ HB8630:2004).

In addition to timber bridges we build suspension bridges. The most remote of these is at Te Totara Bay on the far side of Lake Waikaremoana. This bridge is anchored into rock at each side of a slip which took out the around the lake track.

We have recently developed masonry skills, as at Mauao we were required to build a wall under a large pohutukawa to prevent it from being undermined (mainly by children). We built a stone faced reinforced concrete retaining wall fixed to the face with grouted rock anchors and backfilled with drainage sand. The stone used was sourced from a quarry in the same type of rock as at Mauao.

Above: Mauao stone faced wall

Our plans are to continue developing our skills and workforce so that we can provide good quality and efficient work for our clients.

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ICE CONSTRUCTION LTD

Pattle Delamore Partners Ltd

Pattle Delamore Partners Ltd is a fully New Zealand owned consultancy that from the early days of 1986 has grown to a medium sized engineering and environmental consultancy.

We are a team of experienced practitioners dedicated to finding sound environmental solutions that are based on scientific thinking, and a thorough understanding of the project needs. We remove the barriers between scientific study, engineering application, and environmental management. This seamless integration of related disciplines is we believe unique to PDP.

PDP's strong client partnerships are built on understanding, trust and performance. Our reputation is for integrity, technical excellence and high standard of work. This is PDP – developing solutions for your environment.

The company has its beginnings in Auckland in 1986 when Alan Pattle returned from 10 years abroad in Europe and Asia and met Keith Delamore who had also returned from Canada. The two decided to team up and so PDP was born. From the humble beginnings in Auckland in 1986 PDP has grown over the years with the Christchurch office opening in 1991 followed by Wellington in 1994.

With the growth has come a broader range of specialised skills allowing PDP to offer an increasingly wider range of skills to our clients. The desire of our people to provide the best solutions and out of the box thinking has allowed PDP to develop innovative and unique solutions to many projects.

Be it administrative or technologically based, PDP is in the forefront with the development of software systems, asking questions of the software developers on the utilisation of the software and often being the first to highlight issues and ways of using the software to the local support staff. At PDP we are always on the lookout to push the envelope of the technology to its limits. "Let's consider this" has often led to innovative solutions or ways of doing things.

We have worked with a wide range of clients in the public and private sectors, including central and local government agencies, international and home-grown companies, other consultants, and members of the public. We have undertaken projects throughout New Zealand, and in Australia and the Pacific Islands.

Our range of services is focused towards natural and physical environmental issues. We apply engineering, science and planning to find sound solutions for your environmental issues. We have worked on projects ranging in type and scale from installing groundwater wells to remediating contaminated land at timber mills, from resource consent applications to design and construction management of wastewater treatment systems and landfills.

PDP has and continues to work all over New Zealand. Milestone projects over the years have included:

- Investigation and cleanup of 1 million litre fuel spill in 1986.
- Construction of land treatment system for paper mill effluent.
- Leachate related consents for New Zealand's first engineered regional landfill.
- Investigation and remediation strategies for provincial town gasworks site.
- Construction of 800m deep well for New Plymouth water supply.
- Groundwater control for Clyde Dam landslides.
- Environmental Management Systems research project for New Zealand Government.
- Stormwater contaminant management for Auckland City Council network consents.

Below: Working in the field with PDP Ltd

- Upgrade of Milton waster water treatment plant.
- Integrated Catchment Management Plan for Papatoetoe for Manukau City Council.
- International Patent obtained for an industrial wastewater treatment system.
- Water management for underground mines for Solid Energy.
- Consents for major hydropower development at Wairau River, Marlborough.
- Consents and construction of large industrial landfill in Bay of Plenty.
- New Zealand wide contaminated land survey for petrochemical company.

What makes PDP different? We believe our clients differentiate us on the basis of trust. They know we will not let them down and that our work will meet their highest standards of quality. Enduring client relationships are testament to this. Internally, we are different because of the way we work together. We have a flat management structure meaning our people receive rapid development opportunities and direct exposure to senior people in related fields. Our company is owned by PDP senior management as shareholders and directors who are invested into the future of the company. Our culture is supportive and friendly—we have a lively young team and an active social environment. We encourage all our people to strive for the best solutions brought about by an environment of senior and junior staff interacting at all levels and at all stages of a project. Innovative thinking is encouraged and all ideas are collectively discussed to provide our clients with the best solution for their environment.

We have a reputation for integrity, technical superiority and high standards. Our strong client relationships are built on confidence and trust; we support the development of sustainable, responsible solutions within the environment.

PDP encourages broad thinking, holistic thinking and striving for the best solution for the problem in the context of environmental sustainability, economic viability and social responsibility. This is PDP – developing solutions for your environment.

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MEMBER PROFILES

Pierre Malan

Occupation
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Having an interest in maths and applied science should naturally lead to an engineering career. In my case, teenage rebellion kicked in and it took a brief but successful diversion to the Law and Science faculties at Otago University to realise that Engineering was my calling. I soon transferred to Canterbury University and as anyone who attended Canterbury will tell you, it's a great place to study and enjoy life as a student. While getting into the workforce at the finish of undergraduate studies was tempting, a masters thesis on strong ground motion in the 1994 Arthur's Pass earthquake was more exciting and I spent a happy year looking at accelerograms and directivity effects.

As my thesis was drawing to a close I accepted a job with URS Auckland as a graduate engineer and quickly found out that the applied side of geotechnical engineering is often a long way from the cleaner theory of the classroom. My first project was an (already designed) retaining wall where the contractor encountered unforeseen ground conditions, with the first problems encountered at Chainage 666. You'd think we might have seen a sign in that! I spent a significant proportion of my first two years modelling the 40 metre high zoned earthfill Cosseys dam using both finite element, 2D and 3D stability models under the guidance of an overseas expert. This analysis was supplemented by field investigations, logging and getting dirt under my nails onsite. Working at URS was a great experience and I made a number of lasting friends as well as getting some very interesting experience. However, adventures overseas beckoned and so, after a couple of months touring archaeological sites in Peru and Bolivia I planned on arriving in London with my wits and a backpack. Unfortunately, I was flying British Airways, so while I landed on my feet, the backpack arrived a week later.

Braving the transition from desert to English winter, I broke with tradition and took a job with Webber Associates, a specialist geotechnical company based near Harrogate, North Yorkshire. The company was at that stage small with half a dozen staff and worked out of a beautiful 1700's stone mansion (previously owned by Lord Mackintosh of toffee fame) with a river winding around the back, a nine hole golf course out the front and tennis courts. I signed up for a month and stayed a very happy four years, marrying my Kiwi sweetheart half way through.

Webbers was a 'contractor friendly' geotechnical design consultancy offering specialist geotechnical design support services to contractors and other consultants. We had a large number of design/build roading projects on the go, with my personal highlights being the design of over 2km of retaining walls for the M42 ATM pilot, earthworks and foundations for an 80,000m^2 warehouse with another 50,000m^2 of hard standing, an 11,000m^2, 5m deep dewatered soil nailed basement with artesian groundwater on the west coast of Ireland and motorways both in the UK and Ireland. Geotechnical work in the UK is a different experience to NZ with no earthquakes to deal with, 1:10,000 geological maps showing exposures, boreholes and even individual houses, glacial geology and plenty of leftover made ground and mining spoil to squeeze projects through. While there I was exposed to fascinating critical state concepts, plenty of soil nail design and construction (my manager sat on the steering committee for the CIRIA soil nailing 'best practice' guidelines), embankments and cuts in soft ground, technical support for expert witness legal jobs and the cut and thrust of tendering multi million pound design/build road projects. It was a wonderful mix of working in a specialist small company on large projects and travelling around the UK, Ireland and Europe – we were lucky enough to attend weddings in five separate countries!

All good things must come to an end and so, in late 2006, we packed our belongings and set off to work at Tonkin & Taylor Auckland. Coming back to NZ after cutting my professional teeth overseas was both exciting and interesting. I soon travelled north to put my roading experience to good use on the Northern Gateway Toll road and have since dabbled in a number of interesting projects ranging from liquefaction resisting stone columns for storage tanks to marinas, clifftop properties and everything in between. It took a bit of a mind shift to get back into the detail of seismic design, stability assessments in a dynamic geological environment with no glacial geology to speak of and also starting to raise a very active wee boy. However, first principles are the same everywhere that gravity acts downwards, so I've happily settled into the NZ environment. I've joined the Auckland Branch Organising Committee of NZGS so if you come to any of the Auckland branch meetings stop by and say hello. Alternatively, you can look for me at Eden Park where I'm obliged to pay the Auckland Rugby Union for the privilege of watching top flight rugby. Well, at least they sell it as top class rugby...

Kori Lentfer

Occupation

Senior Engineering Geologist
Coffey Geotechnics
Tauranga

It all started for me in the King Country backwater of Taumarunui where for 18 years I had the farmland, rivers, bush covered hills and valleys and nearby volcanic central plateau as a backyard playground. Dodging large volcanic boulders in a kayak on the upper stretches of the Whanganui River, digging tunnels in the pumice deposits of the Taupo 186AD eruption, thoroughly enjoying the many earthquakes that had the pictures rattling on the walls and once in particular making the family car rock quite strongly backwards and forwards in the driveway; I think all of these things and more led me along the initially geological and later engineering path.

First year at University of Waikato (UoW) was in a Social Science degree majoring in geography. But after taking a couple of geology papers with some quality fieldtrips I quickly and eagerly jumped into a BSc (Tech) majoring in Earth Sciences. The beauty of this programme is the variety of available subjects including: geology, soil science, hydrology, geochemistry and coastal science. Having a broad general knowledge is a great grounding to specialise from. This degree also included summer break industry placements generally in line with your chosen career path. For me this included two gruelling, hot and very dusty summers undertaking scala penetrometer tests every 100 metres within the left and right hand wheel tracks of every gravel road in the entire Ruapehu District. This added up to quite a few blows per 50mm I can tell you. My final industry placement was quite the opposite with many hours of desktop study and poring over geology maps working for a construction company in Taupo on a quarry establishment project under the guidance of Guy Grocott. This was my first real taste of engineering geology and it had me hooked.

I headed back to UoW to have a crack at a Masters degree and finished the course work, but couldn't find a thesis topic that got me going. Well, what really happened was a friend came back from working on the rivers in Nepal absolutely raving about the place and after being a kayaking bum for nearly a year, I scratched up an airfare and worked as a safety kayaker for four months with guided raft trips down the wild Himalayan rivers. This started with monsoon fed, huge volume (>4000 m^3/sec), heavily silt laden cataracts early in the season in sub-tropical humidity and heat. In these conditions it's a case of hold on for the ride, and often hold your breath. Over the duration of the season as river levels dropped the steeper and much more technical upper reaches became runable in raft and kayak and conditions became temperate then damn cold as winter started knocking.

Having got a little bit of adventure out of the way, but also picking up a bit of a travel bug, I soon headed of (as many a kiwi does) to the barmy shores of the motherland and was most lucky enough to land on my feet with WJ Groundwater based in north London, a groundwater engineering consultancy, specialising in construction dewatering. This small company was involved with some fantastic projects including the Channel Tunnel Rail Link (that's the one under the English Channel) and many other large infrastructure projects. Immediately I was set-up as their site engineer within the geotechnical design team on the Heathrow Terminal 5 project. This was interesting and challenging not only as a very large and high-profile construction project, but also because the site being developed was previously incarnated as a sewage treatment works with effluent contaminated alluvial gravels to dewater. Completely in the deep end and franticly treading water I seemed to manage not to make too many blunders and over a year later handed over to another up and coming young engineer. The next project of note was a bitterly cold posting in the heart of the British winter running deep-well pump tests for a cut and cover tunnel feasibility study right along side Stone Henge. Working away and having only to look over one shoulder at one of the oldest (that I've ever seen anyway) and possibly least understood engineering marvels is quite bizarre for a Kiwi boy and rather thought provoking. During my time in Europe I actually developed quite a fixation with castles. I couldn't get enough of wandering along dusty galleries, climbing turrets or creeping through dank dungeons.

After Stone Henge it was time to thaw out with another airport terminal project, this time in Dubai and thankfully during their 'winter'. Pretty much constant mid to high 20° Celsius days and nights hardly counts as winter, except when you compare it to their summer temperatures. This was another challenging project with a ring array of deep-wells with submersible pumps initially drawing down then keeping dry a 20 metre deep, two times rugby field sized excavation. A telemetric monitoring and alarm system was required for this project as should even one of the pumps fail for long enough at the wrong time during construction then the perimeter diaphragm walls without internal structural support could catastrophically fail. There were some frantic times but thankfully no disasters occurred. Another challenge was the ultra-saline (more saline than sea water) groundwater to deal with. The initial lab results failed to identify this fact, and heads were being severely scratched as after a few weeks pumping one after another top-of-the-line Grundfos submersible pumps rapidly reduced their output rates and died. With ten's of thousands of Pounds of equipment down the drain

Above: Just another day checking out some local Bay of Penty welded ignimbrites

very regularly replaced cathodic protection significantly extended the pumps lifetimes to limit losses and keep the project running.

A wife sick to death of hours every day on the dirty old London 'Tube' plus both of us missing that great kiwi outdoors lifestyle contributed to a somewhat reluctant end to the foreign groundwater foray and return back to NZ. I quickly landed a job with Foundation Engineering in the beachside town of Orewa in 2003 just as the property market was really taking off. There I gained my Auckland geology stripes on many a land development project in Waitemata Group and Northland Allochthon terrain. The poorly understood, apparently chaotic, and highly unstable 'Onerahi Chaos Breccia' really plucked at my geology strings again and the many earthworks projects Foundation Engineering were running provided the ideal fieldwork opportunity for a Masters thesis. Warwick Prebble at the University of Auckland was very supportive as a supervisor for my part time study. He was also very patient, as two and a half years later a study on the engineering geology of the Northland Allochthon was produced.

Having finished the main task keeping me bound in one place for 5 years, it was time to find a new challenge and broaden my knowledge base. A relocate to the Tauranga branch suited particularly well and the family shifted down the east coast a bit, in early 2008. My work with Coffey Geotechnics (having acquired Foundation Engineering in 2006) has provided a huge variety of geotechnical challenges ranging from small landslip repair work through to national infrastructure projects around the country. Certainly never a dull day! The global focus of Coffey has dramatically increased and more and more time seems to be spent internationally particularly with technical development work.

And just because I felt that I had too much time in the day, the opportunity arose to take on the Bay of Plenty/ Waikato Branch Coordinator role for the New Zealand Geotechnical Society. This role is really a great vehicle for meeting and getting to know fellow practitioners and it's satisfying being able to give something back to a society that works hard for its members. So if you've managed to wade through this ramble, then this is my chance to say; Get along to those presentations, site visits and events that the NZGS puts on. Often a lot of effort goes in to putting them on and we can always improve our technical knowledge and also get to know each other a bit better.

NEW ZEALAND GEOTECHNICAL SOCIETY INC.

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ADVERTISERS DIRECTORY

EVENTS DIARY

Links are available from the NZ Geotechnical Society website – www.nzgs.org

2010

2-4 February 2010 Turkey
2nd International Congress on Marble and Natural Stones. www.marble.jmo.org.tr

7-9th April 2010, Adelaide, Australia
3rd International Conference on Problematic Soils. www.cipremier.com

12 and 13 April 2010, Auckland, New Zealand
Piling and Deep Foundations: New Zealand Details to come

20-22 April 2010, Western Australia
Second International Symposium on Block and Sublevel Caving. www.caving2010.com/

9-11 May 2010 California, United States
2nd International Symposium on CPT, CPT'10 www.cpt10.com/

10-13 May 2010, Taipei, Taiwan
17th South East Asian Geotechnical Conference www.17seagc.tw/index.htm

14-20 May 2010 Vancouver, Canada
The World Tunnel Congress 2010 and 36th ITA general Assembly. www.wtc2010.org

23-27 May 2010 Brazil
9th International Conference on Geosynthetics www.9icg-brazil2010.info

24-29 May 2010 San Diego, California, USA
5th International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics conference.mst.edu/5geoeqconf2010

2-4 June 2010 Bratislava, Slovak Republic
14th Danube-European Conference on Geotechnical Engineering www.decge2010.sk

7-10 June 2010 Moscow
ISSMGE - International Geotechnical Conference - Geotechnical Challenges in Megacities www.geomos2010.ru

20-25 June Bulgaria
10th International Multidisciplinary Scientific Geoconference and Exp SGEM1010 www.sgem.org

28 June - 1 July 2010 Hönggerberg Campus , Zurich, Switzerland
7th International Conference on Physical Modelling in Geotechnics www.icpmg2010.ch

5-10 September, 2010 Auckland, Aotearoa New Zealand
11th IAEG Congress - Geologically Active www.iaeg2010.com

8-12 November 2010 New Delhi, India
6th International Congress on Environmental Geotechnics 6icegdelhi@gmail.com

2011

21-26 May 2011 Helsinki, Finland
The World Tunnel Congress 2011 and 37th ITA general Assembly www.ril.fi/web/index.php?id=641

23-28 May 2011 Hong Kong, China
XIV Asian Regional Conference Soil Mechanics and Geotechnical Engineering

13-16 June 2011 Maputo, Mozambique
XV African Regional Conference on Soil Mechanics and Geotechnical Engineering

31 August to 3 September, 2011 Seoul, Korea
Fifth International Symposium on Deformation Characteristics of Geomaterials www.isseoul2011.org

13-19 September 2011 Athens, Greece
XV European Conference on Soil Mechanics and Geotechnical Engineering

NEW ZEALAND GEOTECHNICAL SOCIETY INC.

Objects

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.

Membership

Engineers, scientists, technicians, contractors, students and others who are interested in the practice and application of soil mechanics, rock mechanics and engineering geology.

Members are required to affiliate to at least one of the International Societies.
Students are encouraged to affiliate to at least one of the International Societies.

Annual Subscription

Subscriptions are paid on an annual basis with the start of the Society's financial year being 1st October. A 50% discount is offered to members joining the society for the first time. This offer excludes the IAEG bulletin option and student membership. No reduction of the first year's subscription is made for joining the Society part way through the financial year.

Basic membership subscriptions (inclusive of GST), which include the magazine, NZ Geomechanics News, are:

Members	**$75.00**
Students	**Free**
Annual IPENZ service centre fee applies to all NZGS members who are not members of IPENZ	**$42.75 (incl GST)**

Affiliation fees for International Societies are in addition to the basic membership fee:

International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE)	**$24.00**
International Society for Rock Mechanics (ISRM)	**$33.00**
International Association of Engineering Geology & the Environment (IAEG)	**$21.00**
(with bulletin)	**$70.00**

All correspondence should be addressed to the Management Secretary. The postal address is:

NZ Geotechnical Society Inc, P O Box 12 241, WELLINGTON

The Secretary
NZ Geotechnical Society Inc.
The Institution of Professional Engineers New Zealand (Inc)
P.O. Box 12-241, WELLINGTON

NEW ZEALAND GEOTECHNICAL SOCIETY INC.
APPLICATION FOR MEMBERSHIP

(A Technical Group of the Institution of Professional Engineers New Zealand (Inc))

FULL NAME (Underline Family Name): ……………………………………………

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Phone No: (   )………………… **Fax No: (   )** ………………… **E-MAIL:** ……………………

DATE OF BIRTH ……………………………

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EXPERIENCE IN GEOMECHANICS: ……………………………………………

STUDENT MEMBERS: ……………………………………………

TERTIARY INSTITUTION: …………………………… **SUPERVISOR:** ……………………

SUPERVISORS SIGNATURE: ……………………………

Note that the Society's Rules require that in the case of student members "the application must also be countersigned by the student's Supervisor of Studies who thereby certifies that the applicant is indeed a bona-fide full time student of that Tertiary Institution". . . ; Applications will not be considered without this information.

Affiliation to International Societies: All full members are required to be affiliated to at least one society, and student members are encouraged to affiliate to at least one Society. Applicants are to indicate below the Society/ies to which they wish to affiliate.

I wish to affiliate to:

International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE)	Yes/No
International Society for Rock Mechanics (ISRM)	Yes/No
International Association of Engineering Geology (IAEG)	Yes/No
& the Environment (with Bulletin)	Yes/No

DECLARATION: If admitted to membership, I agree to abide by the rules of the New Zealand Geotechnical Society

Signed …………………………………………… Date ……/……/……

ANNUAL SUBSCRIPTION: Due on notification of acceptance for membership, thereafter on 1st of October. Please do not send subscriptions with this application form. You will be notified and invoiced on acceptance into the Society

PRIVACY CONDITIONS: Under the provisions of the Privacy Act 1993, an applicant's authorisation is required for use of their personal information for Society administrative purposes and membership lists. I agree to the above use of this information:

Signed …………………………………… Date ……/……/……

(for office use only)

Received by the Society ……………………………………………

Recommended by the Management Committee of the Society ……………………………

NEW ZEALAND GEOTECHNICAL SOCIETY INC. PUBLICATIONS 2009

Publication Name	List Price Members	List Price Non-Members
New Zealand Geomechanics Society Conferences:		
Proceedings of Technical Groups, Vol 22, Issue 1G **(1 left)** *Geotechnical Issues in Land Development* Hamilton 1996	$20	$35
Proceedings of the New Zealand Geotechnical Society Symposium – *Roading Geotechnics 98* Auckland 1998	$40	$70
Proceedings of the New Zealand Geotechnical Society Symposium – *Engineering and Development in Hazardous Terrain* Christchurch 2001	$50	$70
Proceedings of the New Zealand Geotechnical Society Symposium – *Geotechnics on the Volcanic Edge* Tauranga 2003	$50	$70
Proceedings of the New Zealand Geotechnical Society Symposium – *Earthquakes and Urban Development* Nelson 2006	$50	$70
Proceedings of the 18th New Zealand Geotechnical Society Symposium –	$50	$70
Soil-Structure Interaction, Auckland 2008. (CD)	$20	$25
Australia – New Zealand Conferences on Geomechanics:		
Proceedings of the 2nd Australia – NZ Young Geotechnical Professionals Conference, Auckland, December 1995	$25	$40
Proceedings of the 5th Australia – NZ Young Geotechnical Professionals Conference, Rotorua, March 2002 (spiral bound reprint)	$75	$85
Proceedings of the 6th Australia – NZ Conference on Geomechanics Christchurch, February 1992	$50	$100
Proceedings of the 9th Australia – NZ Conference February 2004 – 'To the enz of the Earth' **(Vol 2 only)**	$150	$200
Other Publications:		
NZ Geomechanics News Collection 1970–2003 Volumes 1–66 (CDRom)	$25	$40
2005 Soil & Rock Guideline	$25	$50
Shear Vane Guidelines	$15	$20
Back Issues of *NZ Geomechanics News* **(selected issues)**	$20	$20

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Orders to: Amanda Blakey, Management Secretary. Email: nzgeotechnicalsociety@xtra.co.nz

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NZ Geomechanics News is published twice a year and distributed to the Society's 740 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.

Advertisement Location	Single Issue	Advert. Size (mm)
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Half Page Internal	$210	90 wide x 265 high
		185 wide x 130 high
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Insert to be posted with magazine – $240/flyer
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1. All rates exclude GST.
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3. A 3mm bleed is required on all ads that bleed off the page. Bleed must be set up on all files that are supplied.
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