ISSN 0111-6851

N.Z. GEOMECHANICS NEWS

No. 47

JULY 1994

A NEWSLETTER OF THE N.Z. GEOMECHANICS SOCIETY

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NOTES FOR CONTRIBUTORS

NZ Geomechanics News is a newsletter for which we seek contributions of any sort for future editions. The following comments are offered to assist 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 the international journals and conferences
  - technical notes
  - comments on papers published in Geomechanics News
  - descriptions of geotechnical projects of special interest.
- General articles for publication may include:
  - letters to the NZGS
  - letters to the Editor
  - articles and news of personalities.

Submission of text material in camera-ready format is not necessary although typed copy is encouraged. Diagrams and tables should be of size and quality for direct reproduction. Photographs should be good contrast black and white gloss prints and of a suitable size for mounting to magazine format. Authors and other contributors must be responsible for the integrity of their material and for permission to publish.

Geoff Farquhar
EDITOR

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THIS IS A REGISTERED PUBLICATION

"NZ Geomechanics News" is a newsletter issued to members of the NZ Geomechanics Society. It is designed to keep members in touch with recent developments. Authors must be consulted before papers are cited in other publications.

Persons interested in applying for membership of the Society are invited to complete the application form at the back of the newsletter. The basic subscription rate is $36.00 and is supplemented according to which of the international societies, namely Soil Mechanics ($16.00), Rock Mechanics ($16.00) or Engineering Geology ($37.00) the member wishes to be affiliated. Members of the Society are required to affiliate to at least one International Society.

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EDITORIAL

Tim Sinclair recently stepped down from editorship of NZ Geomechanics News after 4 years of service. We are grateful to Tim for his enthusiasm and dedication, often producing issues whilst working overseas. His wit and many challenges issued in this journal will be missed. Mostly his challenges were not taken up by readers bursting into print with letters to the editor. However the new editors are wary of an increase in letters now that Tim is not constrained by writing to himself.

Mick Pender and Robert Anderson's paper on **geotechnical limit state design** is published in this issue of Geomechanics News. This aspect of design is causing difficulties for designers using NZS 4203:1992, and NZGS proposes to hold workshops later in the year to give insight into the difficulties. Attention is drawn to the conclusions in the paper by Pender and Anderson. The proposal for a load and resistance factored design approach needs to be debated and critically evaluated. Please use this newsletter to air the debate. We invite discussion of the issues by means of design examples, questions, notes etc sent to the editor. Comments will be obtained from the joint committee of the NZGS and the Structural Engineering Society. It is imperative that the geotechnical community gives input to the debate and the modifications required to geotechnical limit state design in NZS 4203. We can not afford to be ignored as we were in submissions for the NZ Building Code Handbook.

Debate is also invited on a sensible approach to **slope stability** as it affects land development. Now that the magic factor of safety of 1.5 has crept into the Building Code Handbook without any qualification, Territorial Authorities and consultants are having difficulties in evaluating slope stability assessments. What is your opinion on the basis for acceptance of 1.5 and situations where different factors of safety can be accepted? What is the place of engineering geological assessment of slope stability rather than numerical assessment? We the geotechnical community need to agree on a philosophy of stability analysis so that engineers, geologists, Territorial Authority officers and land owners prevent the waste of resources debating the issues on an ad hoc basis.

Geoffrey Farquhar
<u>EDITOR</u>

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REPORT FROM THE MANAGEMENT SECRETARY

1. MEMBERSHIP

1.1 New Members

The following new members are welcomed to the Society:

A. Longbein
V. Moon
D. Oakey
R. Sutherland
G. Stevens
N. Taylor
J. Underhill
G. Wakeman
A. Wallis

1.2 Student Grade of Membership

To encourage full time students to join the Society rule changes were introduced at the recent Annual General Meeting (13 May 1994) to create a student grade of membership. Subscription rates for student members will be set at half the full member's rate.

1.3 Overdue Accounts

A small number of members have overdue accounts. In some cases we have lost contact with members, and in other cases accounts have not been paid for some years and the outstanding amounts have accumulated. If you are in this position and have genuine reasons for not paying please contact us as we may be able to assist.

2. MANAGEMENT COMMITTEE

Eight nominations were received for the 1994 Management Committee and so a postal ballot was not necessary. The positions held by members of the Management Committee are as follows:

David Bell	Chairperson
Trevor Matuschka	Secretary
Colin Newton	Treasurer
Fred Smits	Vice Chairperson (ISRM)
Geoffrey Farquhar	Editor Geomechanics News
Ian McPherson	National Activities Officer
Stephen Crawford	Assistant Editor
Dick Beetham	Vice Chairperson (IAEG)

In addition the following members were co-opted:

Stuart Palmer	Vice Chairperson (ISSMFE) Publications Officer
Michael Pender	Australasian Vice President (ISRM)
Warwick Prebble	Australasian Vice President (IAEG)

Two new members, Geoffrey Farquhar and Stephen Crawford, are welcomed on to the Management Committee. They have responsibility for Geomechanics News, taking over from Tim Sinclair who retired as editor following a number of years in this role. Tim's efforts as editor, particularly his editorial comments, have been much appreciated together with his input into other Management Committee tasks.

Stuart Palmer has been co-opted onto the Management Committee for this year having been a serving Committee member for the past few years. Stuart has had a number of responsibilities including organising the recent Symposium. Professor Michael Pender and Dr Warwick Prebble have also been co-opted onto the Management Committee. Professor Pender is currently the Australasian Vice-President of ISRM and Dr W. Prebble will be taking over as Australasian Vice-President of IAEG in September 1994. These are notable positions, held for a term of four years, representing the Australasian region at Board meetings of ISRM and IAEG.

3. ANNUAL GENERAL MEETING

The 1994 Annual General Meeting of the Society was held during the recent Symposium "Geotechnical Aspects of Waste Management" on 13 May. Traditionally, it has been held during the annual IPENZ Conference in February. A very good attendance was achieved (39 members present). Rule changes were passed to introduce a student grade of membership (refer to 1.2) and to remove the necessity for IPENZ representatives on the Management Committee.

4. STUDENT PRIZE

The Management Committee approved rates for the New Zealand Geomechanics Society Student Prize at its last meeting on 12 May 1994. Further details of the prize are contained elsewhere in Geomechanics News. Please encourage all students enter and support them by attending the presentations of their papers at the Local Branch meetings.

5. NZNSEE RECONNAISSANCE TEAM POOL MEMBERS

We have received five replies to our request for volunteers for the New Zealand National Society for Earthquake Engineering (NZNSEE) reconnaissance teams pool. NZNSEE were seeking for us to nominate up to 10 members so if you missed the note in the last issue of Geomechanics News and are interested in joining, please write to the Secretary, NZGS.

6. REGISTRATION OF ENGINEERING GEOLOGISTS

At the present time, the future structure of IPENZ is under review, and IPENZ is considering expanding the scope of membership. A subcommittee has been formed to liaise with IPENZ to determine how best Engineering Geologists can fit into the new structures.

Trevor Matuschka
MANAGEMENT SECRETARY

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REPORT FROM THE VICE CHAIRMAN FOR ISSMFE

IPENZ currently hold a limited number of copies of the ISSMFE membership list. If anyone would like a copy of the list please contact the IPENZ Secretary.

The "XIII International Conference on Soil Mechanics and Foundation Engineering" was held in New Delhi 5-10 January 1994. New Zealand Geomechanics Society members Mick Pender, Laurie Wesley and Tam Larkin presented papers at the conference. Prior to the conference the ISSMFE Council met. Professor M.B. Jamiolkowski of Italy was elected as ISSMFE President for the period 1994-1997.

ISSMFE are reviewing their technical committees. Some 30 technical committees have been established to research specific topics. Anyone interested in serving on any of these technical committees please contact the writer for more details.

Stuart Palmer
VICE CHAIRMAN ISSMFE

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REPORT FROM THE VICE PRESIDENT FOR ISRM

This is a brief report on matters concerning the ISRM that will be of interest to members of our Society.

Board Meeting In Paris

In March a special Board meeting took place in Paris. This was convened by the ISRM President, Charles Fairhurst, to discuss some matters raised at the 1993 Board meeting in Lisbon but which, because of time constraints, we were not able to complete. Because some of these matters needed to be resolved prior to the 1994 Board meeting in Santiago this special Board meeting was convened. As things turned out I was not able to attend this meeting because of other commitments.

The three main items discussed in Paris were: the process of electing the President of the Society, co-ordination of activities with the ISSMFE and the IAEG, and the ISRM News Journal.

It was decided to propose to the Society that a President-elect be elected at an ISRM Symposium about two years prior to taking up office. This would give the incoming President an opportunity to become familiar with the operations of the Society before taking up office. Currently the whole Board changes at the end of each ISRM Congress. They have a brief meeting at the Congress and then do not meet for another year, thus a year or so goes by before the Board comes to terms with its role.

The President of the ISRM, ISSMFE, and the IAEG meet from time to time. In recent meetings there has been discussion as to how the three societies might be helpful to international organisations such as United Nations agencies and the World Bank in providing expertise for reviewing project proposals. Apparently this advice could be more effective and might be sought more regularly if the Societies acted together rather than independently. Because of this the Presidents have ben discussing closer co-operation. Another aspect of these discussions has been the hosting of four yearly international conferences and congresses of the Societies. These now require major financial backing. The presidents have been exploring the feasibility of having some type of joint congress rather than independent events. Clearly these developments will take some time but to member Societies like ours, which represent all three international groups, closer co-operation is seen as beneficial.

The ISRM News Journal was discussed. This seems to be getting good comments from those reading it. The editor would welcome articles from NZ. Does anyone want to write about an interesting rock mechanics project? It is supported by advertising revenue, readers might like to keep this in mind. Charles Fairhurst makes the point that advertising in the Journal may be a way to making contacts for joint venture proposals, etc.

Council and Board meetings in Santiago:

The Board of the ISRM met for one and a half days and there was also a half day Council meeting. Both these meetings took place in Santiago early in May and just prior to the IV South American Congress on Rock Mechanics.

The Board selected the Rocha Medal winner, awarded for the best PhD thesis submitted to the Society. The choice was Dr Derek Martin from Canada for his thesis entitled "The Progressive Fracture of Lac du Bonnet Granite".

The main issue at the Council was the plan to change the election process for the President. This was confirmed.

The next thing to be decided was the Muller Lecture to be presented at the Congress in Tokyo in 1995. The member Societies are asked to make nominations prior to the Council meeting and information is circulated. The choice of the Council was Professor Neville Cook nominated by the US national group.

The Board and Council received reports from the Japanese National Group on progress of the organisation of the 1995 Congress of the Society. Members of our Society will be pleased to know the Laurie Richards will be giving one of the invited lectures.

Mick Pender
AUSTRALASIAN VICE PRESIDENT ISRM

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REPORT FROM VICE CHAIRMAN IAEG

Recent IAEG Activities - up to June, 1994:

- Over recent years Mr John Braybrooke (of New South Wales, Australia) has been our Australasian Vice President of the IAEG. John has been effective in his communication with the NZ Geomechanics Society and in supporting our endeavours (6th International Symposium on Landslides and the Aust-NZ Geomechanics (Christchurch) Conferences), and we sincerely thank John for his efforts. His turn of office comes to an end later this year and being our turn to nominate a successor, we have put forward the name of Dr Warwick Prebble. Warwick's nomination is supported by the Australian Geomechanics Society and should soon formally be accepted by the IAEG. The NZ Geomechanics Society will be providing financial assistance for Warwick to attend the 7th IAEG Congress and Council Meeting at Lisbon in September, this year.

- The 1994 IAEG fees for National Groups have been set by the Council at:

    - member without Bulletin 50 FF (approx. NZ$16)
    - member with Bulletin 140 FF (approx. NZ$45)

- The Hans Class Medal has been awarded to Prof. William Judd, USA.

- A new list of IAEG members is available on request.

- John Braybrooke has been requested to try and revive dormant IAEG groups in Indonesia and the Philippines, and to get Malaysia to form a national group. If anyone in NZ has the names and addresses/phone/fax numbers on any people active in engineering geology in these countries, could they please forward them to me (or direct to John Braybrooke, Fax (61-2) 809 4095).

- Registration of Engineering Geologists: You will probably not be surprised to learn that this issue is still alive. IPENZ are proposing extensive reorganisational and membership changes, and it is possible that they will accept engineering geologists into their widened membership.

Dick Beetham
VICE CHAIRMAN ENGINEERING GEOLOGY

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LOCAL GROUP ACTIVITIES

1. AUCKLAND BRANCH

The Auckland Branch is midway through an active programme of meetings for 1994. The following three meetings have been held in the first half of the year:

March	Discussion on Geotechnical Limit State Design led by Prof Mick Pender (University of Auckland), Trevor Matuschka (Engineering Geology Ltd) and Tony Gibson (Structural Consultant). This was joint meeting with the Auckland Structural Group and the good attendance reflected the interest in resolving the difficulties of geotechnical limit state design introduced by NZS 4203:1992. The active question time opened the debate on the subject. A paper by Mick Pender and Robert Anderson printed in this issue of Geomechanics News covers the content of the meeting.
April	1994 Geomechanics Lecture on Liquefaction presented by John Berrill (University of Canterbury). John presented an easily understood state-of-the-art review of liquefaction and its assessment
May	Health and Safety issues in geotechnical practice presented by Kerry Harris (Loss Control Management Systems).

<u>Planned Future Meetings</u>

July	Ian Gunn (University of Auckland) on Recent Developments in Effluent Disposal.
August	Prof Guy Houlsby (Oxford University)
September	Soil Improvement at Museum of NZ in Wellington and report on Tim Sinclair's sabbatical.
October	Sky City (Sky Tower and Casino) development.

Geoff Farquhar
<u>AUCKLAND BRANCH CONVENOR</u>

2. WELLINGTON BRANCH

The Wellington Branch has had an extremely active 1994. Organisation and running of the seminar on geotechnical aspects of Waste Management in May this year kept a large number of people very busy. We have had two talks to date this year. These were:

April	John Berrill of the University of Canterbury presented the 1994 Geomechanics Lecture on Aspects of Liquefaction.
May	P. Brabhaharan of Works Consultancy Services spoke on liquefaction hazards in the Wellington region. The talk discussed aspects of the Wellington Regional Council's programme of identifying seismic hazards in the Wellington region, in particular liquefaction and slope failures.

Talks coming up include:

July	Greg Saul of Works Consultancy Services on stabilisation of Cairnmuir Landslide in the Cromwell Gorge.
September	Graham Ramsay and Tom Marshall of Beca Carter Hollings and Ferner, on the Ewen Bridge Foundations. This bridge has deep foundations extending into the artesian Hutt Aquifer which supplies a significant portion of Wellington's water supply.
October	Ian Brown of Techbase Australasia on use of computers to manage geological data.

A talk will also be arranged for November/December.

Ian McPherson
WELLINGTON BRANCH CO-ORDINATOR

3. OTAGO BRANCH

John Berrill gave the 1994 Geomechanics lecture in May. Invercargill were well represented, as were non-members but a disappointing turnout from Dunedin members. Unfortunately, that will be the one and only time the drinks will be free so bad luck for those who missed out. Thanks to John for a very informative presentation.

We distributed a member's survey in order to gauge interest in types of future activities and a number of ideas have been put forward. We are considering running a forum on "Landfills and Liners" in late July which will follow on from the Waste Management Symposium held in Wellington on May. The forum will involve gaining perspectives from local and regional authorities (and their obligations/requirements under the RMA) and from an engineering design perspective. This may lead on to a full day workshop if enough interest is created.

We are also considering a Storm Damage forum, focusing on a few heavy rainfall events on Summer 1993/1994. These events caused considerable damage in the Otago/South Canterbury region. This may lead onto a storm damage control and remedial design workshop.

We are also trying to arrange to have Graham Salt give a presentation on aspects of the slope stability work he has been carrying out in Malaysia.

If anyone is interested in these events, or has other ideas/views please contact me or David Stewart.

Phil Glassey
<u>OTAGO/SOUTHLAND BRANCH</u>

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IPENZ AWARDS, 1994

You are reminded that IPENZ makes a number of Special Awards each year. Branches and Technical Groups are invited to nominate papers for each of the following Special Awards. Such information must be received by the Awards Committee by no later than 1 October 1994.

- **Furkert Award**

  For the best paper published by the Institution during the three year period to 31 July 1994 on a subject dealing with the action of water on the faces of nature, particularly such faces of nature as are connected with the works of man. Author(s) must be Members of IPENZ.

- **Angus Award**

  For the best paper published by the Institution during the three year period to 31 July 1994 on a subject possessing a substantial mechanical engineering interest. Author(s) must be Members of IPENZ.

- **Evan Parry Award**

  For the best paper published by the Institution during the three year period to 31 July 1994 on an electrical engineering subject. Author(s) must be members of IPENZ.

- **Skellerup Award**

  For the best paper published by the Institution during the three year period to 31 July 1994 on a subject possessing a substantial chemical engineering interest with a preference for papers dealing with the utilisation of New Zealand's natural resources or of the development of New Zealand's chemical process industry. Author(s) must be Members of IPENZ. When a paper is co-authored, the principal author must be a Member of IPENZ, but any co-authors may be non-engineers and as such, be ineligible to be members of IPENZ.

- **IPENZ Structural Award**

  For the best paper published by the Institution in the three year period to 31 July 1994 on some aspect of structural engineering design or construction in which materials other than prestressed or reinforced concrete fulfil a major role. Author(s) must be a Member of IPENZ.

- **Rabone Award**

  For the best paper published by the Institution during the three year period ending 31 July 1994 on a subject of a general nature which does not qualify for one of the other Special Awards. Author(s) can hold any class of Membership of IPENZ and should preferably be under 40 years of age.

- **Freyssinet Award**

  For the best paper published by the Institution during the three year period ending 31 July 1994 on some aspect of engineering structural design or construction in which prestressed or reinforced concrete fulfils a major role. Author(s) must be a Member of IPENZ but when a paper is co-authored the principal author must be a Member of IPENZ and the co-author(s) either a member or ineligible to be a Member of IPENZ provided that where a co-author(s) is normally located overseas these latter conditions need not apply.

Nominations of papers for Special Awards should include:

(a) title of Paper
(b) author(s) full name and membership grade
(c) details of where the paper is published
(d) award for which the paper is nominated
(e) general comment in support of the nomination

To facilitate consideration by the Awards Committee, Branches and Technical Groups are asked to nominate no more than two papers for any one award.

Technical groups are asked to pay particular attention to papers published in their Group Proceedings or equivalent publications which do not normally come to the attention of the Awards Committee.

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Any NZGS members who may wish to have papers they have presented during the last 3 years considered for an award should contact:

Stuart Palmer,
C/- Beca Carter Hollings & Ferner Ltd,
P.O. Box 3942,
Wellington
Phone: (04) 473 7511

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NEW ZEALAND GEOMECHANICS SOCIETY STUDENT PRIZE

The NZGS Student Prize will commence in 1994. The rules that apply to this prize are as follows:

1. The New Zealand Geomechanics Society wishes to recognise and encourage student participation in the fields of rock mechanics, soil mechanics and engineering geology. It has therefore agreed to present annually <u>two</u> merit awards, each to the value of $250 together with a suitably inscribed certificate, which shall be known as the "**New Zealand Geomechanics Society Student Prize**".

2. The award shall be made to the bona-fide full-time student of a recognised Tertiary Institute in New Zealand who makes the adjudged best presentation on any aspect or topic in the field of geomechanics to the designated Local Group Meeting in either Auckland or Christchurch. The award is open to both undergraduate and postgraduate students, but the same student is not eligible for more than one award.

3. In May of each year students shall be invited to submit a Synopsis of their topic to the Local Group convenor in either Auckland or Christchurch, and the due date for receipt of synopses will be 30 June. The Synopsis shall <u>not</u> exceed 1,000 words or two A4 pages typed.

4. Students whose synopses are accepted shall be invited to present their topic verbally at a Local Group meeting specially designated for that purpose, and this will usually be held in September. The Local Group convenor shall be responsible for the format and timing of the meeting, but students should normally be required to speak for 20 minutes followed by 5 minutes of questions.

5. The Prize shall be awarded to the student who is judged to have made the best presentation in terms of clarify (both written and verbal), and who is considered to have dealt with questions most competently. The composition of the judging panel is a matter for the Local Group convenor, and the judges' decision shall be final.

6. The Local Convenors in Auckland and Christchurch are expected to liaise regarding the timing, format and venue for the annual Student Prize Meeting in each centre, and to ensure that the awards are made each year under generally similar conditions.

NOTE : Students wishing to submit a paper for the 1994 NZGS Student Prize should contact David Bell of the Geology Department, University of Canterbury, Private Bag, Christchurch before 31 July 1994.

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GEOTECHNICAL LIMIT STATE DESIGN SEMINARS

RETAINING WALLS AND FOUNDATIONS FOR GEOTECHNICAL AND STRUCTURAL ENGINEERS

Geotechnical Limit State design became an issue with the publication of the 1992 version of the NZ Loadings Standard (NZS 4203:1992, SANZ). The fundamental assumption of this document was that all structural design would be done in a limit state format. The NZ Loadings Standard provides the loads whilst associated material standards provide the details of materials performance. To date the steel, timber and concrete codes have appeared, or are about to appear, in limit state format.

Since there is no geotechnical standard, and none is apparent on the horizon, the people drafting the NZ Loadings Standard present suggestions in the loadings code. For example, when designing retaining structures with the limit state approach, the resulting design seemed to be rather "heavier" than designs obtained following the 1984 edition of the NZ Loadings Standard.

As a consequence of this problem a small committee was formed representing the Geomechanics Society and the Structural Engineering Society. Trevor Matuscka and Mick Pender represented the Geomechanics Society and Ian Billings, Tony Gibson and Bob Anderson represented the Structural Engineering Society. This committee met fairly regularly for most of last year. There have been three outcomes so far:

- A paper in the SESOC Journal: "Suggestions for geotechnical ultimate limit state design in NZ", by M.J. Pender and R.G. Anderson (SESOC Journal, Vol. 6 No.2, December 1993, pp. 14-27), also published in this issue of Geomechanics News

- A joint Geomechanics Society - SESOC meeting in Auckland in March on the topic of Limit State Design. Speakers were Trevor Matuschka, Tony Gibson and Mick Pender.

- A meeting of the Auckland Structural Group held in April on the topic of Retaining Walls and Limit State Design, the speaker being Mick Pender.

Subsequent to this the Management Committee of our Society has decided that a **seminar should be held in a number of centres this year.** A joint Geomechanics Society/SESOC committee is currently planning this and the seminar is likely to occur in late October/early November. It is intended that this will be a one day event. It is expected to cover an introduction to limit state design concepts, discussion of the factor of safety approach and its inherent weaknesses, and the working of examples for a gravity retaining wall and a shallow foundation by the participants. By working through these example problems during the workshop the participants will get insight into how limit state designs are done and will compare these with designs from the traditional approach.

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Suggestions for Geotechnical Ultimate Limit State Design in NZ

M J Pender[1] and R G Anderson[2]

Introduction

Limit state design is a general term embracing many different procedures. The purpose of the method is (i) to deal with sources of uncertainty affecting the design, and (ii) to differentiate between situations leading to collapse of a structure (ultimate limit states) from those in which the limit state is unacceptable deformation (serviceability limit state). For both classes of limit state it is presumed that the designer has investigated a sufficient number of scenarios to identify the most critical case which becomes the basis for design. This was actually the basis of the geotechnical design process long before the rise of the term limit state design. The particular feature of limit state design is the separation of the uncertainties associated with loads from those associated with material properties. In the traditional approach these are lumped together in a single total factor of safety. Limit state methods have been used in structural engineering for some time but, as yet, they are not widespread in geotechnical engineering.

The intention of this brief paper is to promote discussion of procedures for ultimate limit state geotechnical design in NZ. One approach is given in NZS 4203:1992 but therein (clause C6.6.3, p. 93) it is implied that the suggested strength reduction factor is an interim one pending the advent of a geotechnical standard. The object of this paper is to promote consideration of this question by offering an alternative non-seismic ultimate limit state design method. The proposal is the outcome of a series of meetings over a period of several months of a small committee representing the NZ Geomechanics Society and the Structural Engineering Society. At this stage the proposal needs to be discussed and criticised widely by geotechnical and structural engineers as it is far from definitive, and, at this stage in the interests of presenting the concepts with clarity, it is incomplete. Hopefully this debate will lead to the development of formal geotechnical limit state design procedures for both seismic and non-seismic applications in NZ.

Internationally much attention is currently being devoted to the topic of limit state design in geotechnical engineering. In May 1993 version 4 of *Eurocode 7 Part 1: Geotechnical Design, General Rules* was released, and in 1994 it will be promulgated in three languages as a European pre-norm. This provides a period of about three years in which the code is used as a pre-standard while attention is devoted to confirming or amending load factors and partial factors of safety. In conjunction with this process an International Symposium on Limit State Design in Geotechnical Engineering was held in Copenhagen in May 1993.

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[1]Department of Civil and Resource Engineering, University of Auckland. [2]Beca, Carter, Hollings and Ferner, Auckland.

with those obtained using traditional methods. In particular one can make comparisons based on the traditional total factor of safety approach. This has been the process followed in arriving at the proposal made in this paper. Meyerhof (1993) explains that this was the procedure followed in arriving at partial factors of safety when limit state design methods were first proposed for geotechnical engineering in the early 50's. The underlying requirement for the proposal presented in this paper is that the resulting designs have similar proportions to those obtained with NZS 4203:1984. There is a consensus in the profession that designs based on this code have performed satisfactorily.

Structural design in New Zealand is based on an LRFD approach. The load factors are given in the Loadings Standard (NZS 4203:1992) and the various materials standards give values for the strength reduction factors. This being the case it is attractive to follow a LRFD approach in foundation design so that all aspects of civil engineering design in NZ are done on the same conceptual basis. LRFD methods for geotechnical work are under evaluation in Canada and have been suggested for some applications in the United States. The partial factor of safety approach is being followed consistently across the Eurocodes. As this is different from the method used in the NZ material codes it is probably best not to introduce the partial factor of safety method into NZ geotechnical calculations at this stage. The decision to propose a LRFD method does not imply that it is superior to a partial factor of safety approach. It is simply a matter of consistency.

In recent decades so-called reliability approaches have been developed to derive load factors, partial factors of safety, and strength reduction factors. These methods use probabilistic techniques and provide an alternative to calibration against existing standards for arriving at the required factors. Reliability methods have not been used in developing the proposal presented in this paper. To arrive at definitive values for soil strength reduction factors some reliability work will be necessary.

Developments elsewhere

There has been considerable discussion in the literature on the philosophy of limit state approaches in geotechnical design. Informative discussions of the issues involved are given by Bolton (1981 and 1993), Burland et al (1981), Duncan et al (1989), Green (1989), Ovesen (1992), Simpson et al (1981), and Simpson (1992). Inconsistencies with the total factor of safety approach and the need to move cautiously in codifying geotechnical procedures, because of the variability of natural materials and the uniqueness of each site, are emphasised by several of these authors. An historical perspective is given by Meyerhof (1993).

Eurocode 7 is an extensive document with much useful information and advice for those involved in geotechnical design. As this is well set out and clearly presented it is used to provide performance criteria and data for the worked examples discussed below. As with the other Eurocodes, EC7 factors the applied loads and uses partial factors of safety to derive design strength values from the assessed characteristic strength of the soil. The cohesion and the tangent of the friction angle are divided by numbers greater than unity. Partial factors for the undrained shear strength, cohesion, and tangent of the friction angle are different. This is consistent with the fact that the cohesion and undrained shear strength are known to be more variable than the friction angle. Documentation of this for one NZ material is given by Pender (1980). The Ontario Bridge Design Code and the Hong Kong Guide to Retaining Wall Design also use a partial factor of safety approach.

In Canada a LRFD approach, Becker (1993) and Golder and Associates (1992), is currently

As has been explained above consistency with NZ material codes means that it appears desirable to adopt a LRFD approach for geotechnical work.

(c) The provision of one "strength reduction factor" (= partial factor of safety) is too simple. It has long been known that soil cohesion and friction angle parameters have different amounts of variability (Pender (1980) gives some NZ evidence). Other partial factor of safety approaches recognise this by specifying separate values for use with friction angle and cohesion.

(d) The value of 0.6 given in clause 6.6.3 is too severe when applied to tangent of the friction angle. This is the main reason why retaining walls designed with the provisions of NZS 4203:1992 are heavier than those obtained from the use of NZS 4203:1984 which are known to have performed satisfactorily.

Proposal

The proposal given here is intended for shallow foundations and retaining walls. At this stage we have endeavoured to keep the details simple thus a range of strength reduction factors, such as appears in Appendix I, is not given. Rather the values suggested are representative of the values given by Barker et al (1991) and by Becker et al (1993). Final adoption of values for strength reduction factors must await consensus of the profession along with reliability studies to supplement the factor of safety calibration discussed herein.

Proposals along similar lines could be presented for deep foundations, and, in fact, are given by Barker et al (1991) and Becker (1993).

A factored load and resistance approach is in two parts; (i) the load factors and load combinations and (ii) the strength reduction factors applied to the ideal strength and resistance.

(i) Load Factors and Load Combinations

When calculating design loads in accordance with NZS4203:1992 we consider that it is necessary to introduce an additional load factor of 1.6 on active earth pressures to derive appropriate values for the stem moment of cantilever retaining walls. This is akin to the earth load factor of 1.7 in NZS 4203:1984. The calculations described in example (b) below show that this 1.6 load factor gives about the same stem moment for a cantilever wall as do the provisions of NZS 4203:1984.

In many stability calculations the dead load contributes to the resistance which is generated by mobilised soil strength. In this case a load factor on the dead load of 0.9 is applicable in accordance with the value suggested by load combination 4 in clause 2.4.3.3 on p. 19 of NZS 4203:1992. (The factoring of the unit weight of the soil is not followed in EC7 as the committee drafting this document have marshalled several arguments suggesting that this factor should always be unity, Orr (1993c).)

For the design of a gravity retaining wall under static conditions the following load factoring would be done. The backfill generates active thrust. This thrust is factored by 1.6 when determining the required proportions for the wall. However the unit weight of the backfill is not also treated as a dead load and factored. The weight of the wall acts to promote the stability by generating sliding resistance. Because of this the weight of the wall is factored by 0.9. If there is some surcharge in front of the wall which is considered as contributing to the resistance then the unit weight of this is also factored by 0.9. Finally

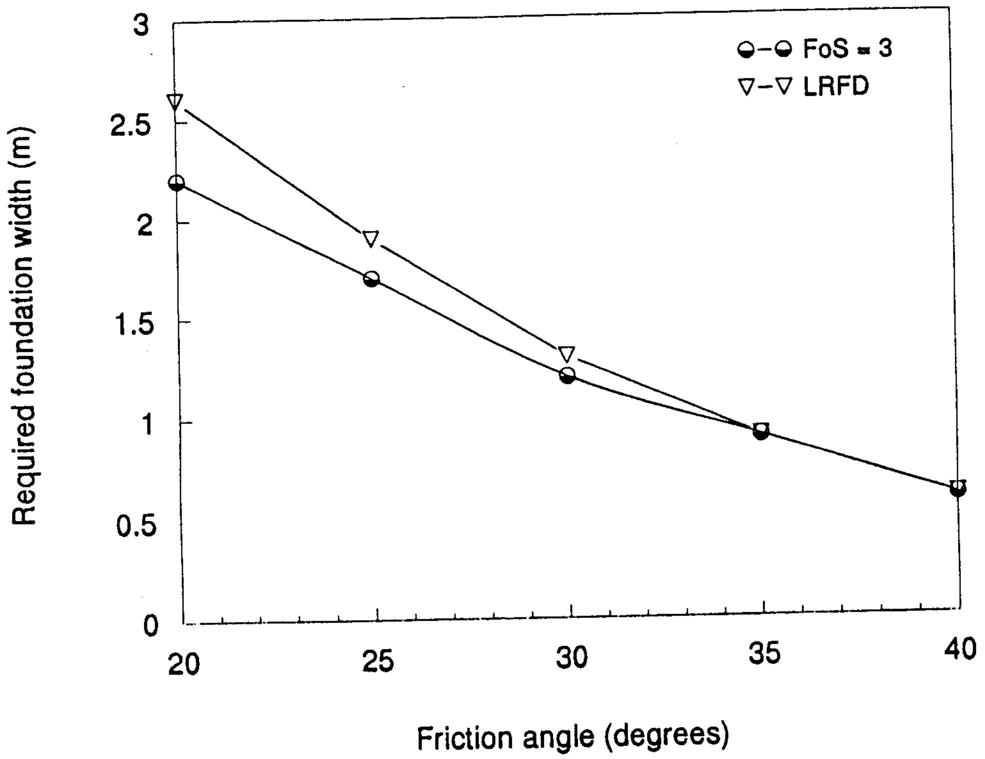

Figure 1 *Required strip foundation width for foundations in dry sand. Comparison of the widths required using the proposed LRFD and total factor of safety methods.*

Examples

Three examples illustrating the application of the proposal are given in this section.

(a) **Width of a strip footing in dry sand**

Figure 1 gives a comparison between the required foundation width for a the conventional total factor of safety approach with FoS = 3 and the proposed LRFD method. The sand is dry, the unit weight was taken to be 18 kN/m^3, and the embedment of the foundation was half the width. Vertical load was applied to the foundation consisting of 50 kN dead load and 100 kN live load. It is apparent that the two approaches give similar widths for the foundation, the LRFD method giving slightly greater values than the conventional method. For the usual range of sand friction angles, 25° to 35°, the widths by the two methods are very similar. The calculations are set out in Appendix III.

(b) **The maximum ideal stem moment in a cantilever retaining wall**

This example covers calculation of the maximum ideal stem moment for a simple cantilever retaining wall. The intention of the calculations is to compare the maximum ideal stem moment given by a number of approaches. Active earth pressures on the retaining wall were derived from the Coulomb theory for a dry backfill with a unit weight of 16 kN/m^3. The ideal section strength was derived from the limit state or ultimate strength modified by the strength reduction factor consistent with the design and materials code combinations being considered. On this basis of the ideal maximum stem moments for each code can be compared. The calculation of the stem moment for the proposed method is as given in the example set out in Appendix IV under the heading Third Limit State (page 4 of the worked solution). The bending moments calculated using various material codes are compared in Figure 2. It is clear that the maximum moment at the base of the stem calculated using the proposal of this paper is almost exactly the same as that given by the provisions of NZS 4203:1984 and several other methods of design. In comparison NZS4203:1992 gives values which are 20-40% higher for the range of backfill friction angles considered.

(c) **The retaining wall example given in Example C6.6.1 of NZS 4203:1992**

To illustrate the application of the code provisions to geotechnical situations example calculations were given for a gravity retaining wall resting on saturated clay and backfilled with dry sand having a horizontal unloaded surface. The same example has been re-worked herein applying the provisions of the proposal. The annotated calculations and a definition diagram are given in Appendix IV. In the example in the Loadings Standard Commentary the required width for the base of the wall is 2.5 m. In Appendix IV the required width is 1.6 m.

Comparison

Appendix II gives a table providing an overview of the various approaches to geotechnical ultimate limit state design.

Firstly it needs to be appreciated that the traditional factor of safety approach can be criticised. A factor of safety of 2 does not mean the same thing in a bearing capacity situation as in a slope stability application. For a cohesionless soil with a friction angle of 30°, a unit weight of 18 kN/m^3, and zero pore water pressures a slope stability factor of safety of 2 means that the mobilised friction angle is $\tan^{-1}((\tan 30)/2) = 16°$. The bearing capacity factor of safety depends on the width of the foundation. For the same soil a surface foundation of width 2 m having a bearing capacity factor of safety of 2 has a mobilised friction angle of 25°.

The partial factor of safety approach has the advantage of dealing directly with the differing variability of the cohesive and frictional components of soil strength. These factors are fixed and as is explained below this puts emphasis on the **correct** selection of the characteristic strength of the soil.

The LRFD method does not address the different variability of the friction angle and cohesion directly. However it does have the potential to allow for the method of estimating the soil strength values as is illustrated in Appendix I.

Material parameters

None of the methods that have been considered in reaching the proposal set out herein has a formal process for selecting material strength parameters. Eurocode 7 introduces the idea of a characteristic value of the soil strength. This is described as "..a cautious estimate of the value affecting the occurrence of the limit state." The shear strength parameters associated with this characteristic value are modified by the partial factors of safety to give the design strength parameters. There has been considerable discussion of the meaning of the term characteristic strength, Baguelin (1992), Bolton (1993), Orr (1993a and b) and Denver and Ovesen (1994). Orr (1993b) points out that the reliability of designs based on Eurocode 7 is critically dependent on the choice of strength parameters. The same could be said for the proposal presented herein. More discussion of this topic can be expected.

Conclusions

The ultimate limit state proposal presented in this paper is based on a load and resistance factored design approach. There are other equally valid limit state design approaches but as the NZ material codes are, or are about to be, expressed in a load and resistance factored design format it seems appropriate to present geotechnical methods in the same manner.

The basis for the proposal herein is the requirement for foundation designs to be similar to those achieved with the earlier editions of the NZ Loadings Standard. It is seen in example (b) that the stem moment of a cantilever retaining calculated with the proposal is about the same as that obtained with NZS 4203:1984. Examples (a) and (c) set out the various calculations for the design of strip footings in sand and a gravity retaining wall respectively.

What is required now is a critical evaluation of the proposal for ultimate limit state geotechnical design and a testing of it in a number of different situations and applications. A process not unlike the pre-norm trial period which will commence in 1994 Eurocode 7. If it is agreed that the LRFD approach is appropriate for use in NZ then the values for strength reduction factors need to be confirmed using reliability methods. Strength reduction factors are also needed for deep foundations. Eventually the discussion needs to be widened to cover serviceability limit states and aseismic design.

Acknowledgements

The joint Committee of the NZ Geomechanics Society and the Structural Engineering Society referred to earlier consisted of R G Anderson, I J Billings, and J A Gibson, representing SESOC, and M J Pender and T Matuschka representing the Geomechanics Society.

One of the authors (MJP) wishes to acknowledge the large amount of information provided by engineers involved in developing geotechnical limit state approaches elsewhere. These responded promptly to faxes (in some cases to a series of faxes) and provided extensive information. Without this assistance, so willingly given, it would not have been possible to develop the proposal presented in this paper.

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Appendix I: Range of Strength Reduction Factors Proposed for Shallow Foundations in NCHRP Report 343, Barker et al (1991)

Type of Limit State	Strength Reduction Factor
1 Bearing Capacity	
(a) Sand	
- Semi-empirical Procedure using SPT data	0.45
- Semi-empirical Procedure using CPT data	0.55
- Rational Method --	
using ϕ estimated from SPT data	0.35
using ϕ estimated from CPT data	0.45
(b) Clay	
- Semi-empirical Procedure using CPT data	0.50
- Rational Method	
using shear strength measured in lab tests	0.60
using shear strength measured in field vane tests	0.60
using shear strength estimated from CPT data	0.50
(c) Rock	
- Semi-empirical Procedure (Carter and Kulhawy)	0.60
2 Sliding	
(a) Precast concrete placed on sand	
using ϕ estimated from SPT data	0.90
using ϕ estimated from CPT data	0.90
(b) Concrete cast in place on sand	
using ϕ estimated from SPT data	0.80
using ϕ estimated from CPT data	0.80
(c) Clay (where shear strength is less than 0.5 times normal pressure)	
using shear strength measured in lab tests	0.85
using shear strength measured in field tests	0.85
using shear strength estimated from CPT data	0.80
(d) Clay (where shear strength is greater than 0.5 times normal pressure)	0.85

Note:

1 ϕ = friction angle of sand

2 The maximum shear stress on the interface between a footing and clay is one-half the normal stress on the interface or the adhesion of the clay, whichever is the smaller. For cast in place concrete the adhesion may be taken as the full undrained shear strength of the clay. The adhesion may be reduced to 0.5 to 0.7 times the undrained strength is the concrete is wet.

Appendix II: Comparison of various geotechnical ultimate limit state design methods

Method	Factors for Ultimate Limit States							
	Partial safety factors			Strength reduction factors		Load factors		
	$P_{f\phi'}$	$P_{fc'}$	P_{fsu}	Φ_{sl}	Φ_{bc}	α_D	α_L	α_E
Proposed herein				0.8	0.5	1.4/0.9	*	1.6
Becker (NBCC proposal) (1993 and 1992)				0.8	0.5	1.25/0.85	**	1.25
NCHRP Report 343 (1991)				≈0.8	≈0.5	1.3/1.0†	2.17	1.69
Ontario Bridge Design Code (1986 + updates)	1.25	1.5-2.0	1.5-2.0			1.25/0.8	1.4	1.25
EC7 (4th. version February 1993)	1.25	1.6	1.4			1.35/1.0	1.5	1.0

Notes:

Partial safety factors, the design value of the various parameters is obtained from: tanφ' divided by $P_{f\phi'}$, c' divided by $P_{fc'}$, the undrained shear strength, s_u, is divided by P_{fsu}.

Performance factors: resistance based on sliding is multiplied by Φ_{sl}; resistance based on bearing capacity, passive earth pressure, and pile shaft capacity is multiplied by Φ_{bc}.

Load factors: α_D dead load, α_L live load, α_E earth pressure. The smaller of the paired values for α_D are for use in situations where the dead load has a beneficial effect on stability by contributing to the resistance. †Report 343 gives another value of α_D (=0.75) for checking members with minimum axial load and maximum moment, this seems to be applied to structural capacity only.

* 1.2D + 1.6L (plus other combinations given in the Loading Standard NZS 4203:1992).

** 1.25D + 1.5L (depends on wind to live load ratio as explained by Becker 1993).

The ≈ notation for the NCHRP Φ values is to indicate that there is a range of values (given in Table 2.3 on page 6 of Part I, and Table 4.3 on p. 72 of Part 2 of the report) which refine the values for different methods of determining soil parameters, the values given above are "averages".

For the Ontario Bridge Code $P_{fc'}$ and P_{fsu} are 1.5 for stability and earth pressures and 2.0 for footings and piles. It also gives special Φ values for pile axial capacity based on prototype tests and dynamic assessments (p. 151-152), without this data the above values are to be used.

The partial safety factors given above from EC7 are taken from Table 2.2 (page 14). A slightly different set of values is given for pile foundations in Table 7.2 (page 62).

Appendix III: Calculations for the required width of vertically loaded strip foundations in sand

File: BCFSFLFR.mcd 11/9/93, 16/9/93, 4/12/93

Required foundation width comparison for conventional factor of safety and load and resistance factor design methods.

Vertically loaded foundation in dry sand.

Depth of foundation: $Df = \beta D$.

$ORIGIN := 1$

$DegRad := \frac{\pi}{180}$

$RadDeg := DegRad^{-1}$

$VD := 50.$	Dead load (kN)
$VL := 100.$	Live load (kN)
$\beta := 0.5$	Df/B
$\alpha D1 := 1.4$	Load factor for dead load
$\alpha D2 := 0.9$	Dead load factor for surcharge
$\alpha L := 1.6$	Load factor for live load
$\Phi bc := 0.5$	Strength reduction factor
$F := 3.0$	Bearing capacity factor of safety
$gamma := 18$	Unit weight of soil
$i := 1..5$	
$\phi_i := 20 + (i-1)\cdot 5$	Array of friction angles
$\phi_i := \phi_i \cdot DegRad$	

Bearing capacity factors:

$$Nq_i := e^{\pi \cdot \tan(\phi_i)} \cdot \tan\left(\frac{\pi}{4} + \frac{\phi_i}{2}\right)^2$$

$$Ng_i := 2 \cdot (Nq_i - 1) \cdot \tan(\phi_i)$$

Depth correction factors (Brinch Hansen DGI Bulletin No. 28):

$$dq_i := \left[1 + 2 \cdot \tan(\phi_i) \cdot (1 - \sin(\phi_i))^2\right] \cdot \beta$$

$$dg := 1$$

Alter Nc, Nq, and Nq:

$Nq\lambda_i := Nq_i \cdot dq_i$

$Ng\lambda_i := Ng_i \cdot dg$

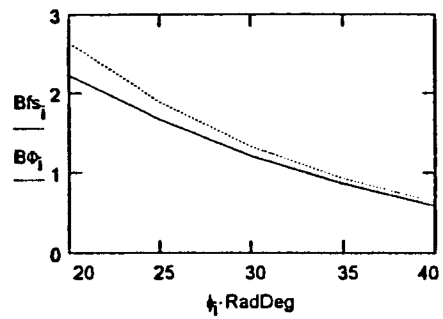
Now evaluate required width of foundation using LRFD approach:

$$B\Phi_i := \left[\frac{VD \cdot \alpha D1 - VL \cdot \alpha L}{gamma \cdot \Phi bc \cdot \alpha D2 \cdot \left(\beta \cdot Nq\lambda_i + 0.5 \cdot Ng\lambda_i\right)}\right]^{0.5}$$

Now evaluate required width of foundation using factor of safety approach:

$$Bfs_i := \left[\frac{F \cdot (VD - VL)}{gamma \cdot \left[F \cdot \beta - \beta \cdot \left(Nq\lambda_i - 1\right) + 0.5 \cdot Ng\lambda_i\right]}\right]^{0.5}$$

$\phi_i \cdot RadDeg$	$B\Phi_i$	Bfs_i	Nq_i	Ng_i
20	2.642	2.221	6.399	3.93
25	1.884	1.667	10.662	9.011
30	1.333	1.214	18.401	20.093
35	0.927	0.857	33.296	45.228
40	0.626	0.583	64.195	106.054



Appendix IV: Calculations for the retaining wall example given in clause C6.6 of NZS 4203:1992

File: WALL4203.mcd 14, 30/9/93 & 13/10/93 & 26/11/93

Calculations by:
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Limit State Design calculations for the cantilever retaining wall Example C6.6.1 in the Commentary of NZS 4203:1992, p. 90-92.

Following the geotechnical Load Factor and Strength Reduction Factors suggested by the NZGS and SESOC committee, September '93.

Conventional cantilever wall with heel (zero toe projection) and a virtual back.

Dry sand backfill and clay foundation.

Short term analysis of bearing capacity and sliding of foundation.

Will not include the effect of the surcharge at the toe of the wall.

We will need formulae for bearing capacity, recommendations for sliding resistance, and values for earth pressure coefficients. Eurocode 7 provides a convenient source of this information and so will be used. Clearly there are many alternative sources for such information.

Five possible ultimate limit states:

(i) rotational deformation because of short term bearing capacity failure towards the toe of the wall,

(ii) outward sliding of the wall at foundation level (short term),

(iii) structural failure of the stem of the wall,

(iv) overall short term slope stability failure,

(v) long term effects need to be considered for cases (i), (ii), and (iv).

MathCAD housekeeping:

$DegRad := \frac{\pi}{180}$	Conversion from degrees to radians
$RadDeg := \frac{180}{\pi}$	Conversion from radians to degrees

WALL AND BACKFILL PROPERTIES:

$Tf := 0.4$	Thickness of foundation (m)
$Hw := 3.6 - Tf$	Wall height (m)
$Bw := 1.60$	Width of foundation (m)
$Tw := 0.3$	Thickness of wall stem (m)
$Tp := 0.0 \cdot Bw$	Toe projection in front of stem (m)
$Th := Bw - Tw - Tp$	Heel projection behind stem (m)
$Tp = 0$ $Th = 1.3$	
$\phi := 30$ $\phi := \phi \cdot DegRad$	Backfill and foundation friction angle
$\beta := 0$	Inclination of backfill surface
$su := 100$	Undrained shear strength of clay (kPa)
$gammabf := 16$	Unit weight of dry sand backfill
$gammac := 25$	Unit weight of reinforced concrete

LOAD FACTORS & STRENGTH REDUCTION FACTORS (NZGS & SESOC committee):

$LfD := 0.9$	Load factor for soil dead loads which improve stability
$LfC := 0.9$	Load factor for concrete dead loads which improve stability
$LfP := 1.6$	Load factor for active thrust
$\Phi bc := 0.50$	Strength reduction factor for bearing capacity.
$\Phi sl := 0.80$	Streng.h reduction factor for sliding.
$\Phi bm := 0.85$	Strength reduction factor for flexure of concrete masonry (Park, SESOC Journal 1993).

ACTIVE PRESSURE COEFFICIENT:

Before we can assign Ka we have to decide whether the virtual back of the wall is rough or smooth. If the heel projection is wide enough relative to the width a double active wedge can form. One half is above the heel projection and the other in the backfill behind the wall. In this situation symmetry means we have zero angle of wall friction. For a friction angle of 30 degrees and a smooth interface the lower surface of the active wedge is at 60 degrees to the horizontal. The angle defined by the geometry of the heel is atan(3.6/1.3) = 70 degrees. This means we cannot have the full active wedge so there must be some wall friction. We will simply assume that $\phi w = \phi$.

$\phi w := \phi$	Angle of wall friction, assumed equal to ϕ
$Ka := 0.27$	Ka (horizontal component, $\delta = \phi$) Fig. A8.1 of EC7

ACTIVE THRUST:

$$Pah := Ka \cdot gammabf \cdot LfP \cdot \frac{Hw^2}{2}$$

$$Pav := Pah \cdot \tan(\phi w)$$

$Pah = 55.296$ $\quad Pav = 31.925$ (kN/metre run of wall)

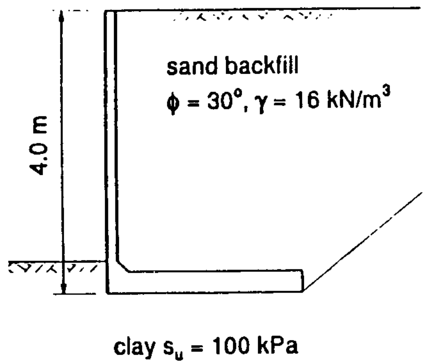
FIRST ULTIMATE LIMIT STATE (bearing failure beneath the wall)
(see attached diagram for geometry)

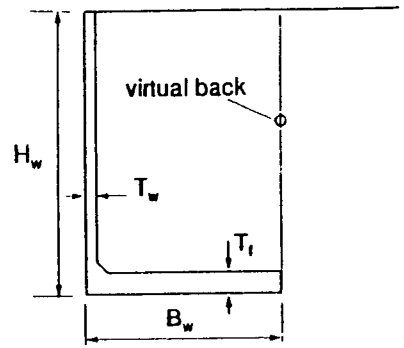
$Ws := (Hw - Tf) \cdot Tw \cdot gammac \cdot LfC$	$Ws = 24.3$
$Wf := Bw \cdot Tf \cdot gammac \cdot LfC$	$Wf = 14.4$
$Wbf := (Hw - Tf) \cdot Th \cdot gammabf \cdot LfD$	$Wbf = 67.392$
$Vert := Pav + Ws + Wf + Wbf$	$Vert = 138.017$

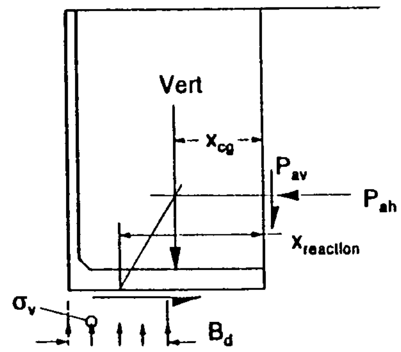
Taking moments about heel of wall to get position of resultant vertical force:

$$Mom := Wf \cdot Bw \cdot 0.5 + Ws \cdot (Tw \cdot 0.5 + Th) + Wbf \cdot Th \cdot 0.5$$

$xcg := \frac{Mom}{Vert}$	$xcg = 0.656$	Distance from heel to Vert.
$xreaction := xcg + \frac{Pah \cdot Hw}{3 \cdot Vert}$		Where reaction crosses base.
$Bd1 := 2 \cdot (Bw - xreaction)$		$Bd1 = 0.819$
$Bd2 := 2 \cdot (xreaction)$		$Bd2 = 2.381$
$Bd := if(Bd1 > Bd2, Bd2, Bd1)$		$Bd = 0.819$
$\sigma v := \frac{Vert}{Bd}$		$\sigma v = 168.455$







Refer to page 92 of the Commentary volume of NZS 4203:1992

Since the wall is long we do not need shape correction factors. We need to evaluate the bearing capacity correction factor for inclined loads. Annex 2 of EC7 gives the appropriate formula.

$$ic := 0.5 \cdot \left(1 + \sqrt{1 - \frac{Pah}{su \cdot Bd}}\right) \qquad ic = 0.785$$

$$Nc := 5.14$$

$qu := su \cdot Nc \cdot ic$	$qu = 403.534$	$qu\Phi := qu \cdot \Phi bc$
$\sigma v = 168.455$	$qu\Phi = 201.767$	If $qu\Phi > \sigma v$, then OK

(Note that these calculations are extremely sensitive to base width of the foundation.)

SECOND ULTIMATE LIMIT STATE (sliding):

Page 48 of EC7 requires that H/V <= 0.4 if it is possible for air or water to reach the interface between the foundation and clay subgrade:

$$\mathrm{RatioHV} := \frac{\mathrm{Pah}}{\mathrm{Vert}} \qquad \mathrm{RatioHV} = 0.401$$

Page 47 of EC7 explains that the sliding resistance is to be based on the effective width (Bd) of the foundation.

$$\mathrm{Rsd} := \mathrm{su}\cdot\mathrm{Bd} \qquad \mathrm{Rsd}\Phi := \mathrm{Rsd}\cdot\Phi\mathrm{sl}$$

$$\mathrm{Pah} = 55.296 \qquad \mathrm{Rsd}\Phi = 65.545$$

If RsdΦ > Pah, then OK

THIRD ULTIMATE LIMIT STATE (structural failure of the cantilever stem)

Moment for design of the cantilever stem. The maximum value occurs at the base of the stem:

$$\mathrm{BM} := \mathrm{Ka}\cdot\mathrm{gammabf}\cdot\mathrm{LfP}\cdot\frac{(\mathrm{Hw}-\mathrm{Tf})^3}{6}$$

Maximum bending moment in wall stem (kNm/metre run of wall)

Now convert this to a required ideal moment capacity for the stem:

$$\mathrm{BMcap}_{\mathrm{ideal}} := \frac{\mathrm{BM}}{\Phi\mathrm{bm}} \qquad \mathrm{BMcap}_{\mathrm{ideal}} = 63.233$$

(kNm/metre run of wall)

FOURTH ULTIMATE LIMIT STATE (slope stability behind and beneath the wall)

We are considering here slope stability failure through the sand backfill and through the clay beneath the wall. This is unlikely to be a problem as the bearing capacity of the clay beneath the sand backfill is about 500 kPa whilst the vertical stress imposed by the backfill is 4x16 = 64 kPa.

FIFTH ULTIMATE LIMIT STATE (Check on long term effects)

The above bearing capacity and sliding resistance calculations are based on an undrained shear strength of the clay beneath the base of the wall of 100 kPa. The long term shear strength will be different. The bearing pressure beneath the toe of the wall calculated above is 189 kPa. For a cohesionless material a friction angle of 28 degrees would be required for a shear strength of 100 kPa at that normal effective stress. A clay with su = 100 kPa would be expected to have some cohesion and perhaps a friction of 20 to 25 degrees. Looking at results of consolidated undrained triaxial tests on clays from around Auckland, which frequently have undrained shear strengths of about 100 kPa, it is found that the undrained shear strength for an effective consolidation pressure of about 180 kPa is in excess of 100 kPa, the drained (long term) value will then be greater. Thus it seems that long term effects will not be critical for bearing beneath the wall and sliding.

As fas as slope stability aspects are concerned (fourth limit state) we would need to do a slip circle analysis to accomodate both the shear strength of the sand fill and that of the underlying clay. For such a circle the long term shear strength of the clay will very likely be less than 100 kPa as the vertical effective stress beneath the backfill is 64 kPa.

CONCLUSION:

With a foundation width of 1.5 m the bearing capacity and sliding criteria are not satisfied. With a width of 1.55 m both are OK. However we need the width of the wall to be 1.6 m before the ratio of horizontal thrust to vertical force satisfies the criterion of 0.4 explained under the second limit state.

THUS THE WIDTH OF THE BASE OF THE WALL NEEDS TO BE AT LEAST 1.6 m.

GEOTECHNICAL ASPECTS OF WASTE MANAGEMENT

WELLINGTON SYMPOSIUM
MAY 1994

The Society hosted the above two day symposium at Victoria University on 13 and 14 May 1994. The Symposium was another in the continuing series which the Society arranges on a two to four year cycle.

The subject of the Symposium was selected by the Management Committee as an area of current interest and expanding work opportunities as a result of the requirements of the Resource Management Act. The interest in the topic was confirmed with some 120 participants including 35 presenting papers.

The proceedings were structured to give a wide coverage of the topic as is demonstrated by the list of papers presented in the appendix to this issue of Geomechanics news.

Laurie Richards of Lincoln University presented the keynote address which gave a good overview of the topic including some of Laurie's International experience.

Details of the requirements and procedures associated with the Resource Management Act were presented by lawyers and planners experienced in this field. A special thanks to these authors for their valuable contributions.

The technical overviews and case studies presented in the "Waste Containment" session prompted some interesting discussion on the question of whether or not current practices are unnecessarily conservative. It will be interesting to see how practice develops in the future as we learn more about contamination potential.

Investigation techniques to locate areas of contamination formed a large part of the "Rehabilitation of Contaminated Sites" session. Geophysical methods in particular were discussed. Ray Ryan of New Zealand Rail Limited presented an interesting and entertaining paper in this session on a development over an old landfill (rubbish tip) in 1973.

The "Mining" session provided an interesting overview of recent mining works in New Zealand in relation to environmental aspects and the Resource Management Act.

This report would not be complete without reference to the symposium dinner. A very pleasant evening was spent at the Skyline Restaurant overlooking Wellington City. A theatre Sports Group provided some excellent entertainment at the cost of some symposium participants.

The organising committee would like to take this opportunity to thank all the participants and particularly the authors for making the symposium a worthwhile experience for all.

The papers presented at the symposium are listed in the appendix to this issue of geomechanics News.A copy of the proceedings may be obtained by completing the order form presented elsewhere in this issue of Geomechanics News.

STUART PALMER
for the Organising Committee

THE FIRST AUSTRALIA-NEW ZEALAND YOUNG GEOTECHNICAL PROFESSIONALS CONFERENCE

(9th to 12th February 1994)

In February of this year a number of young NZ professionals attended the conference in Sydney. Of these, five were sponsored by the EQC and NZGS in the form of special awards.

The winners of these awards were:

Mr P. Bosselman	(Auckland)
Mr P. Brabhaharam	(Wellington)
Mr M. Fraser	(Auckland)
Mr R. Vreugdenhil	(Christchurch)
Mr S. Terzaghi	(Auckland)

In addition other NZ attendees were:

Dr V. Moon	(Waikato)
Mr N. Crampton	(Christchurch)
Mr S. Christensen	(Christchurch)

The conference was attended by thirty six registrants, eight directly from New Zealand and an additional five working in Australia but of New Zealand origin and education. One registration came from Papua New Guinea. As part of the technical programme, each registrant was required to give a 12 minute presentation on their paper which was included in the proceedings. The papers covered almost the entire spectrum of geotechnical activity and were uniformly of high quality. The papers were divided into six sessions, as outlined in the following pages.

By all accounts the conference was a great success and enjoyed by all.

Reprints of the technical papers presented by the above people are contained in the following pages of this issue of Geomechanics News along with a list of all papers presented at the Sydney conference. Impressions of the conference by two attendees are also included.

A copy of the bound proceedings can be obtained from Mr Garry Mostyn of the School of Civil Engineering, University of New South Wales, Sydney 2052, Australia. The price of the proceedings is around $30 (Aus) plus postage.

IMPRESSIONS OF THE CONFERENCE

"A senior geotechnical professional kept the proceedings honest"

The conference began in a proper fashion on the Wednesday evening with a social get-together in the Hall of residency where everyone was staying. The idea was that even the local delegates would stay at the Hall in order to foster a convivial spirit, and to help everyone to get know each other. It was a good evening, and certainly helped to break the ice. It was fun meeting a lot of people with very different backgrounds, as well as meeting with people from varsity days, and I am sure that a number of other people felt likewise.

The first real session began Thursday morning with all the delegates introducing themselves with thumbnail sketches of themselves and their careers, and the conference organisers laying down some rules for the sessions. Each session from the delegates (except for the first) would consist of some five or six speakers, each of a nominal fifteen minutes. The speakers would present their papers, with a question period at the end of the session. A senior geotechnical professional attended each session to keep the proceedings honest. That professional then took the second session of the day and provided an hour long talk on a subject of his speciality. (i.e. Ian Swayne from Golders on Environmental Geotechnics, John Braybrooke on The Many Handed Geologist, Phillip Pells repeating his EH Davis Memorial Lecture). Thursday night was dinner at a local Asian restaurant with a local providing a bit of free entertainment.

Friday afternoon was nominally free, for a site visit, but, instead we had a late afternoon early evening tour around the Sydney Harbour, with dinner aboard the vessel. This was followed up by a visit to one of the Darling Harbour Pubs, testing out the quality of the local brews.

Saturday was a short day, as the conference ended about 3 p.m. that afternoon.

As a footnote, well done Canterbury University. It had supplied highest number of graduates at the conference of any of the universities in Australasia (including me).

The conference was well worthwhile, and was thoroughly enjoyable. It was interesting to share in one's trials and successes with a number of other young professionals from a variety of backgrounds. I wish to thank the New Zealand and Australian Geomechanics Societies for making the event happen and also their support for the delegates, and wish them the best in future events. Also I wish to thank the Earthquake and War Damages Commission for their sponsorship of New Zealand Delegates.

Sergei Terzaghi

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"Geologists regarded as being easier to teach!"

Six New Zealanders were generously sponsored by the New Zealand Geomechanics Society and EQC, these being a mix of people both from University and consulting backgrounds. In addition, another six working in Australia also attended, bringing the total to twelve out of thirty six. It was interesting to note the number of New Zealand geologists at the conference was around nine, predominantly ex-Canterbury University.

Some general impressions:

- New Zealand was seen as "green as a golf course" and "full of earthquakes" (which may have reflected the general tone of "our" papers).
- The Sydney C.B.D. has around twenty, 15 to 25 metre deep vacant foundation excavations (within the Hawkesbury Sandstone), that are generally not retained apart from a rock bolted near surface weathered zone (certainly very different from the soft rock Waitemata we have in Auckland for example).
- Geologists were regarded as being easier to teach engineering to rather than trying to teach geology to engineers and certainly had a much higher profile within the consulting industry than appears to be the case in New Zealand.
- Professional isolation was fairly common among many of those present, especially younger geo's in larger offices or in small offices scattered around Australia.
- Apparently in Darwin a lot of fieldwork is undertaken with three people, two to drill boreholes and one to shoot any crocodiles that may venture too close. In Alaska the problem is bears and the amount of QA/QC required on environmental investigations - one engineer on secondment reported five auditors were monitoring the sampling work being performed by two people.

Overall the conference was excellent - there was a good degree of interaction right from the start and everyone contributed both to the discussion and social functions. Everyone I spoke to found the conference worthwhile from both a personal and professional viewpoint - despite the wide diversity of projects, most people had common experiences to share.

In summary, I would recommend that we keep the ball rolling on this type of conference - the time and effort required for the Sydney conference was well repaid by the outcome and not only are they valuable, I would argue that they are an investment in the profession's future.

Finally, if you get the chance to attend one in the future - do it.

Maurice Fraser

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FIRST AUST-NZ YOUNG GEOTECHNICAL PROFESSIONALS CONFERENCE
OUTLINE OF CONFERENCE PAPERS

Session 1: Numerical methods

Dr. David Ho, Numerical analysis of a large reinforced soil structure by explicit finite difference codes.

Tuan Duc Pham, A new approach to measuring discretisation error.

Simon Quinn, Prediction of surface settlements due to shallow tunnelling in granular soils.

Yat Fung Yu, Random field modelling for the effect of cross-correlation.

Session 2: Earthquakes & Geology

P. Brabhaharan, Assessment and mapping of earthquake induced liquefaction hazards in the Wellington region, NZ.

Steven Christensen, Prediction of liquefied strata from the 1987 Edgecumbe earthquake.

Neil Crampton, Engineering geology and remedial works design, Brewery Creek Landslide, Clyde Power Project, NZ.

Maurice Fraser, Subdivision in the greater Auckland area.

Tim McMorran, Seismotectonics of the Hope Fault, South Island, NZ.

Dr. Vicki Moon, Geotechnical characteristics of ignimbrite: A soft pyroclastic rock type.

Benjamin Panga, Permeability characteristics of volcanic ash soil in PNG.

Ryszard Poplawski, Seismicity underground with particular reference to rockburst problems.

Session 3: Ground improvements

Strath, Clarke, Aspects of rock anchors for the Sydney Athletics Centre.

Robert Day, Granular column stabilisation of a variable fill reclamation.

Stephen Jackson, Embankments constructed over soft clays in the Darwin area.

Ms Julie Nabke, Reinforced soil wall design: A comparison between BE3/78 and AS DR91273.

Dr. Doug Stewart, Studies into the reinforcing effect of stone columns in soft clay.

Ms Heidi Viereckel, The use of geogrids in highway construction in NSW.

Session 4: Miscellaneous

Gregory Cocks, Numeric coding of rock, soil and defect descriptions.

Mark Eggers, Geotechnical studies for Kelian River diversion project, Indonesia.

Mark Foster, Experience with the DP conductivity cone.

Peter Reid, Monitoring of coal mining around large dams.

Sergai Terzaghi, Geotechnical aspects of the Maui II pipeline.

Roger Vreugdenhil, On the interpretation of cone penetrometer data.

Session 5: Piles & Slope Stability

Grant Adams, Current pile design methods.

Serhat Baycan, Improving pile capacity by the use of expansive concrete additives.

Gavan Hunter, West Cut Tailings Dam.

Dr. Hackmet Aly Joer, Model tests on grouted driven piles in calcareous soil.

John Kennedy, Geotechnical considerations following a major haul road collapse, Paddington Goldmine, WA.

Ms Kirsten Souter, Cheero Point Landslide.

Session 6: Environmental Geomechanics

Peter Bosselmann, Interaction between shallow foundation systems and clay shrinkage.

Ian Kluckow, Contaminated site investigations: An American experience.

Chin Jian Leo, Contaminant transport in porous media.

Ian McComb, Cleaning up your own backyard.

Anthony Peyton, Design and construction of the Brooklyn Landfill, Victoria.

Dr. Gareth Swarbrick, Shrink-swell prediction using the water balance method.

Interaction between shallow foundation systems and clay shrinkage

Peter B.C. Bosselmann, B.E.(Civil)
Foundation Engineering Limited, Auckland, New Zealand

ABSTRACT: The consequential effects of clay shrinkage and swelling occurs in numerous centres around New Zealand, particularly in the Auckland region, where it is a common problem for houses of brittle construction (i.e. brick veneer and plaster exterior) constructed on shallow strip or pad foundations. The extent and magnitude of the shrinking and swelling of plastic clays is predominantly related to seasonal variations within the soil-moisture regime. Shrinkage can also be exacerbated by the presence of high water demand tree roots. The resultant movement can be differential settlement and building distress.

1. INTRODUCTION

It is generally accepted that movements of residential and commercial foundations and slabs, pavements and other structures are primarily related to moisture conditions. This is particularly pertinent in respect of buildings with shallow foundations bearing on clay soils which exhibit seasonal volume changes.

These clay soils are referred to in the literature as 'active', 'heavy', 'plastic', 'susceptible', 'expansive' or 'shrinkable'.

The problems associated with buildings constructed on these clay soils relate predominantly to the interaction of the foundation of the structure and movements of the clay soil.

In addition, the extent and magnitude of these movements can be magnified and markedly influenced by the presence of vegetation and/or trees close by which, by the process of transpiration, can cause depletion of the soil moisture regime from its original equilibrium situation. This process is cumulative to that by which moist clay soils lose moisture near the ground surface due to evaporation. The magnitude of this imbalance in moisture equilibrium is dependent on climatic conditions, soil type and location of the water table. The process is referred to as desiccation.

Associated foundation movements can be cyclic on a seasonal basis, progressive settlement where the vegetation is establishing a permanent soil moisture deficit, or progressive heave when such vegetation is later removed.

This paper reviews the literature relating to factors which produce volume changes in shrinkable clays, looks at the capacity of vegetation and trees to cause shrinkage and swell in clay soils and finally gives suggested design procedures and practices for minimising the effects of these movements.

2. IDENTIFICATION CHARACTERISTICS OF "ACTIVE" SOILS

Soils exhibiting significant swelling with an increase in moisture content followed by shrinkage after drying out, as a result of wetting and drying cycles corresponding to seasonal fluctuations in soil moisture conditions, are often termed as 'shrinking', 'plastic', 'expansive', 'active' or 'heaving' clays. These soils contain substantial quantities of certain minerals such as montmorillinite.

Many identifiable areas of shrinkable clays. exist in the Auckland region. The clays showing this characteristic are mainly the plastic residual soils derived from the insitu weathering of Miocene Waitemata Series sandstones and siltstones.

Professor Leonards (1962) describes the phenomena of shrinkage and swelling as:

"In regions which have well defined alternatively wet and dry seasons, susceptible soils shrink and swell in regular cycles. Beneath the centre of a building where the soil is protected from both sun and rain, the moisture changes are small and the soil movements the least. Beneath the outside walls the movements are the greatest. The result is cracking, differential movement and progressive damage."

The terms 'susceptible', 'active' and 'expansive' relate predominantly to volume change. Volume change in a soil mass due both to natural and artificial causes introduces problems to soils that are not encountered with other construction materials.

Volume decrease is caused by load; it is a function of time; it is associated with changes in water and air content; and it is produced by rolling or vibration. Volume increase is a function of load, density, water content and type of soil.

There are special terms used to describe each of these different volume change phenomena and these are listed below as given by the 'Earth Manual'.

Compression defines the volume change produced by application of a static external load.

Consolidation defines volume change that is achieved with the passage of time.

Shrinkage is the volume change produced by capillary stresses during drying of a soil.

Compaction is the volume change produced artificially by momentary bad application.

It has been claimed that shrinkage is the mirror action of expansion and that in terms of the mechanics causing building damage, the properties of both expansive and shrinkable clays are similar.

There are three recognised methods of classifying potentially expansive clays, as documented by Fu Hua Chin:

(a) Mineralogical Identifications

In this method the swelling potential of any clay can be evaluated by identification of the constituent mineral of the clay. The five techniques commonly used are:

(i) X-ray diffraction
(ii) Differential thermal analysis
(iii) Dye absorption
(iv) Chemical analysis, and
(v) Electron microscope reduction

(b) Single Index Method

Simple soil property tests can be used for the evaluation of the swelling potential of expansive soils; such tests may include:

(i) Atterberg limit tests (liquid limit, plasticity index, liquidity index),
(ii) Linear shrinkage tests
(iii) Free swell tests, and
(iv) Colloid content tests

Relation between swelling potential of clays and plasticity index can be established as follows:

Swelling Potential	Plasticity Index
Low	0 - 15
Medium	10 - 35
High	20 - 55
Very High	35 and above

Professor Teng states that expansive soils are often characterised by a high liquid limit and plasticity indicies as a result of the more active clay minerals.

(c) Classification Method

By utilising routine laboratory tests such as Atterberg limits, colloid contents, shrinkage limits and others, the swelling potential can be evaluated without resorting to direct measurement. Some of these methods are listed below:

USBR method
Activity method
Indirect measurement
PVC metre
Soil suction

Overall it is considered that the original soil consistency tests of Atterberg (1911) are the best overall guide to identifying susceptible clays. These tests give 'engineering' properties of fine grained soils in which clay minerals predominate by giving measures of the water contents at which certain changes in the physical behaviour can be observed. It should however, always be borne in mind when using the Atterberg limits for engineering purposes, that since the limit tests are performed on remoulded soils, they are at best only indicative of their physical properties of the remoulded soil and cannot accurately reflect any mechanical properties of the clay in an undisturbed state.

3. TREE INDUCED CLAY SHRINKAGE

In the process of minimising the bareness of the urban residential environment and maximising privacy, the average homeowner typically plants trees and shrubs around and beside the house, which in turn markedly and detrimentally affects the soil-moisture regime as the roots extract moisture from the soil to satisfy the water demand of the tree.

Demand for water varies according to the species, age, location and climate. Reports from the Forest Institute in Rotorua indicates that under typical New Zealand climatic conditions, a realistic estimate for a single large fast-growing tree would be at least 9,000 litres per year. There is also evidence to suggest that a mature poplar tree requires 60,000 litres per year.

'Active' tree roots, by virtue of their root growth and orientation towards moisture, create a subsoil drainage network which maintains a suction in the soil to remove water. The subsequent decrease in length of the drainage paths for the clayey soils and the lowered water table induces dissipation of pore pressures and hence induces secondary consolidation settlement or elastic compression. This increases the effective load on the soil layers below the foundations.

Indirect actions or effects by trees or buildings are recognised in the British Standard Code of Practice, BS5837 (British Standards Institution, 1980) which records that:

"Indirect action by trees results from the removal of soil moisture by tree roots. Where the soil is a shrinkage clay the changes in soil moisture are accompanied by volume changes that can cause movement of building foundations. Such action is very complex because it involves the tree, the local climate, the soil and groundwater conditions, the building, and the interaction between them. While it is seldom possible to make precise predictions of the influence of trees on building performance, some understanding of the above factors is required when making a reasonable judgement."

The common culprit of most damage in the Auckland region is the silver dollar gum tree (eucalyptus cinerea). These trees can grow from 1.0 to 1.5 metres in height per year and reach a mature height of 15 metres. These trees have a preferential lateral root development which provides stability against overturning. Observed rates of lateral root sprawl are believed to be up to 0.5 metres per year under average conditions.

"Kozlowshi (undated) credits the eucalyptus with "extensive leaf surfaces perforated with stomata and hence a tree type admirably constructed to use large amounts of water (transpiration). Since transpiration is controlled largely by atmospheric factors (e.g. light intensity, temperature, humidity, wind); influencing the vapour pressure gradient between the leaf and air, water loss tends to exceed absorption by roots and the typical eucalyptus lateral root system becomes 'aggressive'."

"With reference to the above factors which influence transpiration it can be seen that these features are frequently why the property owner plants trees. The tree (typically silver dollar gum trees) is needed to provide shade, privacy and a wind break. The equation for potential clay shrinkage and resultant house damage is satisfied".

4. BUILDING CONSIDERATIONS

The consequential effects of shrinkage and swelling of clay soils are important considerations in the design and construction of one and two storey residential and commercial buildings within the urban environment.

Settlements, and particularly differential settlements, must be kept within reasonable limits.

The main element to be considered in the design of structures on plastic clay is the foundation.

Foundations throughout the Auckland region most commonly consist of reinforced concrete strip footings. Traditionally strip footings are located below the ground surface rather than at the surface. This is to permit removal of the surface layer of organic soil, gain additional bearing capacity that usually comes from increased embedment and to place the footing below the zone of soil which experiences major volume changes because of seasonal climatic changes or other effects.

Damage to a structure can be attributable to a number of causes such as foundation movement, material drying, shrinkage, temperature contractions and expansions, structural deflections, etc.

In New Zealand there is no statutory obligation on anyone to identify a risk of shrinkage and modify foundations accordingly. To build foundations which are unlikely to be affected by shrinkage involves extra expenditure, but the cost of underpinning, should it become necessary, is considerably greater.

Frequently the problems associated with clay shrinkage are only noticed after drought conditions, generally in late autumn. However, minor blast or earthquake forces, sudden drops in temperature or an extended period of low humidity, can also cause distress cracks and damage.

Burland and Wroth (1975) described the problem of interaction between a structure and the underlying ground on which it is founded, both during construction and subsequently during service. Interaction is a complex problem because it is the combination of several different effects, some of which are time-dependent and none of which are truly linear.

These factors include:

(1) The immediate settlements caused by each increment of load.

(2) The long term consolidation settlements (both primary and secondary) which overlap with immediate settlements.

(3) The changing stiffness of the structure.

(4) The redistribution of loads and stresses within the structure due to differential settlement.

Given the complex interaction of factors involved in causing movement of building foundations, it is the writer's opinion that a compromise or equilibrium approach is an appropriate method of attempting to minimise soil-moisture changes and also to accommodate some degree of building flexibility.

To this end, the equilibrium philosophy and approach should be to:

(a) Carry out soil classification tests (Atterberg limit tests, Cassagrande classification) to determine the soils' potential for shrinkage and/or swelling.

(b) Instruct Local Authorities to identify specific districts where 'expansive' or 'active' clays occur, either at the subdivisional stage or by damage investigations. Once these areas have been identified, it is essential that designers of buildings be advised of these soil conditions.

(c) Ensure designers take specific structural design steps in areas of plastic clays to provide foundations capable of accepting seasonal volume changes of clay; minimise where possible gravity loadings of the structure; pay specific attention to detailing of elements and specification of construction procedures; and attempt overall the design of buildings to accommodate unpredictable foundation movements without damage.

These would include reinforced edge beams having significant torsional stiffness and rigidity, perimeter strip footings founded at a depth compatible with minimal soil moisture variations, bored or driven piles, etc.

It is not considered that a blanket requirement of deepening foundations would solve the problem, rather specific structural design is required.

However, in terms of structural design options, the stiffened foundation is probably more economical but is structurally indeterminate, whereas the post hole bore or driven pile approach, although more expensive, has the advantage of being structurally determinate.

(d) Enable appropriately detailed and located movement control joints to be provided for houses of brittle construction. These joints are desirable to accommodate secondary or second order movements and mainly occur due to temperature changes, drying, shrinkage, curing, building flexibility, vibration, etc.

The advantage of this mode of specific design is that it can enable dwellings such as brick and masonry structures to be constructed, rather than the traditional legislative and sterile approach of prohibition of all brittle-type construction.

(e) Remind homeowners that in areas of clay shrinkage and swell it is essential to maintain the equilibrium soil-moisture regime. Because of this, extensive areas of impervious paving and/or saturation planting of high water demand trees or shrubs should be undertaken with great caution. Of all the input variables which can lead to structural damage of buildings erected on plastic clays, it is the 'house-keeping' measures which ultimately can be the 'straw that breaks the camel's back'.

(f) Provide, with the assistance of the landscape and arbouricultural industry, a catalogue of information pertaining to suitable tree types, root growth and characteristics, tree growth rate, mature height, transpiration rates, etc. Through experience and Engineering judgement, a hierarchy of suitable trees having acceptable water demand characteristics for the urban environment could be established.

This information would naturally assist both the property owners and Authorities in establishing suitable trees to a scale appropriate to that of the soil characteristics.

5. CASE STUDIES

(1) No. 9. Hayes Place, Pakuranga, Auckland (Foundation Engineering reference number 5583)

Settlement of the south-eastern garage portion of the dwelling as a direct consequence of excessive soil moisture depletion was found to be caused by the action of the roots of three blue gum trees situated some 5 to 6 metres away in the neighbouring Council reserve.

Substantial settlement was reflected in cracks to the brick veneer exterior and in the block base wall, together with racked and jammed garage windows and doors, plus large cracks and subsidence within the adjacent concrete pathway.

Hand auger boreholes revealed that there had been a notable reduction in soil moisture content in the vicinity of the cracked portion of the dwelling.

Our recommendations to our clients included:

(i) Formally requesting that the Manukau City Council cut down the trees in the neighbouring reserve and poison the tree roots.
(ii) Observe the situation for at least 1 year and monitor any rebound that may occur as the subsoils move towards equilibrium water contents.
(iii) After approximately 1 year, assess the amount of observable damage remaining and decide whether underpinning was warranted or not.

If the observable damage is still unacceptable then we recommended they install underpinning piles at a series of positions along the affected footing.

(2) 38 Stanley Point Road, Devonport, Auckland (Foundation Engineering reference number 5497)

The dwelling on this level property comprises a 15 to 20 year old architecturally designed two level structure with a brick veneer exterior, founded on 30mm wide x 250mm deep concrete footings. Several large, mature trees were noted growing around the property.

We observed that the south-western and north-western corners of the brick veneer were moving out and down (approximately 25mm displacement).

All movements were essentially of an aesthetic nature and there was no danger to the structural integrity of the dwelling.

Although some filling was encountered in two of our three hand auger boreholes (probably associated with backfill of service trenches), there was no indication of significantly organic, compressible, or otherwise unsuitable substrata within the natural ground that could be considered to be contributing to the observed building damage.

Measured water contents were fairly low within the top metre, indicating some desiccation had occurred.

In our opinion, the principal causes of the veneer cracking were due to the shallowness of the strip footing and the effects of seasonal shrinkage and swelling brought about by soil moisture fluctuations, which have been exacerbated by the presence of several large trees growing along the western boundary.

Our recommendations included:

(i) Removal of the large tree species.
(ii) Monitoring of the cracks over the winter period when soil water contents should adjust to equilibrium conditions.

To restore the veneer back to its original condition, it was recommended that a programme of underpinning would need to be undertaken.

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Assessment and mapping of earthquake induced liquefaction hazards in the Wellington Region, New Zealand

P Brabhaharan
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ABSTRACT The liquefaction hazards in the Wellington region have been systematically mapped. A study of the earthquakes and records of liquefaction in the region during the past 150 years confirmed the presence of significant liquefaction hazards. Two earthquake scenarios, a moderate distant event and a large event on the local Wellington Fault, were considered in the liquefaction assessment and mapping. The liquefaction hazard was mapped using a method specifically developed for the study, which comprised detailed liquefaction assessments at key locations where sufficient geotechnical information was available, followed by mapping using these point estimates and the near-surface geology of the area. Hazard maps for liquefaction susceptibility, potential and the consequent ground damage were prepared.

1 INTRODUCTION

The Wellington region, at the southern part of the North Island, is among the most seismically active areas in New Zealand. Past earthquakes and studies have shown that there is a high risk of damage from earthquakes in the region. The Wellington Regional Council is developing a strategy with an aim to achieving an acceptable level of risk from seismic and geological hazards in the region. Studies to date have involved mapping the surface geology and active faults of the region, and assessing the potential for ground shaking and liquefaction due to earthquakes.

Previous studies such as the Wellington Lifelines Study (Centre for Advanced Engineering, 1991) have highlighted the presence of a number of areas in the region, which have soils susceptible to earthquake induced liquefaction and consequent ground damage. Liquefaction can cause considerable damage to structures and services, and poses a significant hazard to the infrastructure of the region.

Liquefaction is a phenomena giving rise to a loss of shearing resistance or to the development of excessive strains as a result of transient or repeated disturbance of saturated cohesionless soils (National Research Council, 1985). It can be defined as the act

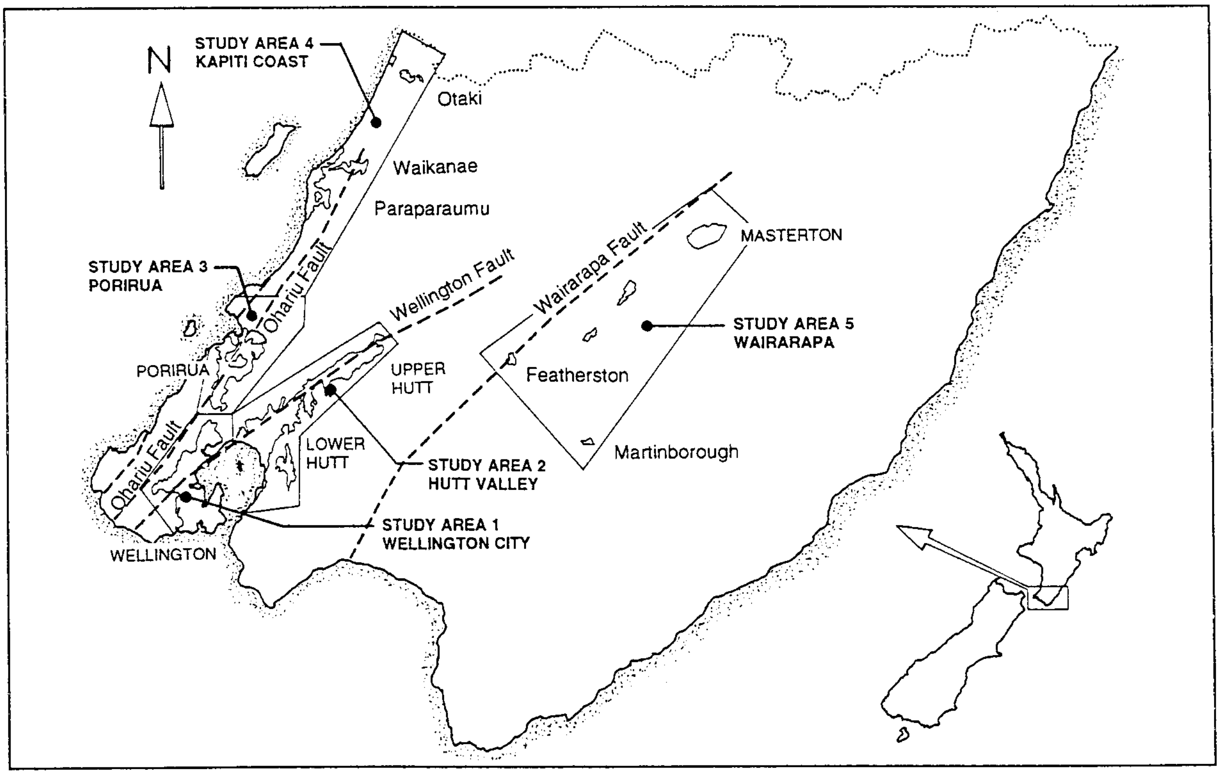

Figure 1 Wellington region and faults

or process of transforming cohesionless soils from a solid state to a liquefied state as a consequence of increased porewater pressure and reduced effective stress. Liquefaction most commonly occurs in loose sands and silty sands, but is also found to occur in loose sandy gravels, sandy silts and silts of low plasticity. Ground damage caused by liquefaction may be of the form of sand boils, subsidence, lateral spreading and flow slides.

'Liquefaction susceptibility' reflects the ground conditions in term of its vulnerability to liquefy when given sufficient ground shaking. 'Liquefaction opportunity' is the occurrence of earthquake shaking strong enough to generate liquefaction. 'Liquefaction potential' is the presence of both 'susceptibility' and 'opportunity' at a particular site, giving the likelihood that liquefaction could take place.

The liquefaction hazard study was carried out for the Wellington Regional Council (Works Consultancy Services, 1993). Five developed areas of the Wellington region, Wellington City, Hutt Valley, Porirua Basin, Kapiti Coast and Wairarapa (the "study areas"), were covered by the study, see Figure 1.

2 GEOLOGICAL SETTING

The Wellington region generally has a rugged terrain, predominantly composed of Wellington and Ruahine Greywacke of Triassic and Jurassic age, overlain by Holocene and Pleistocene age sediments. The Holocene age deposits comprise alluvium in fluviatile, floodplain, estuarine and beach deposits, and dune sand along the west coast. Pleistocene age Hawera Terraces comprise gravel, sand and silt.

A number of major northeast trending faults cross the Wellington region, see Figure 1.

In addition to the natural deposits, large areas of reclamation fill are also present, particularly around the Wellington and Porirua Harbours. These materials vary from soft or loose hydraulic fill to dense rockfill.

3 SEISMICITY

3.1 Regional setting

The Wellington region lies within the most rapidly deforming zone of New Zealand, which is associated with the collision of the Pacific and Australian crustal plates. Much of the deformation is concentrated on major northeast trending dextral strike-slip faults that can move to cause earthquakes of magnitude 7.4 to 8.0. These active faults include the Wairau, Shepards Gully, Ohariu, Wellington and Wairarapa faults, see Figure 1. Under the whole region, there is also a seismogenic zone where the Australian Plate overrides the subducting Pacific Plate.

3.2 Earthquake scenarios

Two earthquake scenarios were considered for the liquefaction hazard assessment. Scenario 1 is a large, distant, shallow (<60 km) earthquake that produces Modified Mercalli (MM) intensity shaking of V-VI in bedrock over the Wellington Region, with a return period of about 20 to 80 years. Such an event could be a magnitude 7 earthquake centred 100 km from the study area, at a depth of 15 km to 60 km.

A Scenario 2 event is a large earthquake centred on the Wellington - Hutt Valley segment of the Wellington fault. Rupture of this fault segment is expected to be associated with a magnitude 7.5 earthquake at a depth less than 30 km.

The scenario approach was chosen by the Wellington Regional Council, given the potential for rupture of the Wellington fault to give a large magnitude 7.5 earthquake in close proximity to the developed areas of the region. The other scenario was chosen to represent ground shaking, from a distant earthquake with a higher probability of occurrence, which could be amplified by the soft soils present in some areas of the region.

3.3 Ground shaking

Observations during overseas earthquakes such as the 1989 Loma Prieta earthquake in California had shown that a large distant shallow earthquake (such as a Scenario 1 event) could result in a marked difference in the ground motions between rock sites and soft/deep soil sites where the ground motions may be amplified. Studies carried out by the Department of Scientific and Industrial Research (1992) have included an assessment and mapping of the likely ground shaking associated with these two scenarios, taking into consideration the distance from the epicentre and the ground conditions. These studies indicate that significant amplification of the ground motions could occur during a Scenario 1 event, in a number of areas of the Wellington region.

Table 1 Ground shaking in Wellington city

Zone	Scenario 1		Scenario 2	
	MM I	PGA (g)	MM I	PGA(g)
1	V-VI	0.02- 0.06	IX	0.5- 0.8
2	VI	0.02- 0.1	IX-X	0.5- 0.8
3 - 4	VI-VII	0.02- 0.1	IX-X	0.5- 0.8
5	VIII-IX	0.1- 0.3	X-XI	0.6- 0.8

(PGA - Peak Ground Acceleration)

Table 1 shows the peak ground accelerations and MM intensities assessed for the two scenarios, in the various ground shaking zones of the Wellington city study area. As shown in the table, there is a large difference between ground shaking in different zones during a distant Scenario 1 earthquake, while there is little variation in the ground shaking during an earthquake centred on the nearby Wellington fault.

4 LITERATURE REVIEW

Methods that have been used overseas to map liquefaction hazards are reported in the published literature. Youd (1991) presents a useful summary of the state-of-the art methods used by various people in different countries. A review of published literature showed that the following three approaches have been used by previous researchers for assessing liquefaction susceptibility :

- Historical liquefaction - this method involves mapping all historical records of liquefaction, and showing zones encompassing these previously liquefied areas as having a liquefaction hazard ;
- Geological mapping - while a number of variations to this method have been used, they essentially involved mapping the near-surface geology (and in some instances geomorphology), considering the age and type of soils contained in the deposits, and assigning a liquefaction hazard based on these parameters. Some studies have also included consideration of the groundwater level ;
- Geotechnical engineering assessment - this involves using various geotechnical engineering methods to assess whether soils will liquefy, and are based on geotechnical engineering parameters assessed from site investigation results. Methods commonly used were the Seed and Idriss' "simplified procedure for evaluating liquefaction potential" and the Japanese Bridge Code method (Youd, 1991). The latter method is widely used in Japan.

The liquefaction opportunity has been assessed using different methods, such as :

- Magnitude-maximum distance - in this method, historical earthquakes and the furthest distance of liquefaction effects have been plotted, and are essentially used as a basis for liquefaction opportunity. However, different researchers have used different data sets and classified the depth of earthquakes and level of liquefaction effects in different ways ;
- Liquefaction severity index (LSI) - Youd and Perkins introduced this method to take into account the severity of liquefaction effects which would be more important for engineering purposes (Youd, 1991). They correlated the maximum horizontal ground displacement of lateral spreading on liquefiable, gently sloping, late Holocene fluvial and deltaic deposits divided by 25, against the horizontal distance from the earthquake source ;
- Magnitude-peak acceleration criteria - this method involves the estimation of peak ground accelerations from seismic sources and attenuation models, taking into account the earthquake magnitude through magnitude scaling factors.

Past researchers have then derived liquefaction potential maps by the superposition of liquefaction opportunity and liquefaction susceptibility maps.

5 METHODOLOGY

A methodology was developed by the author for the liquefaction hazard study of the Wellington region. The methodology was tailored to suit :

- the requirement of the study to assess the hazard for two earthquake scenarios as discussed in section 3.2 above ;
- the variable ground conditions in the region, comprising soils which are clearly susceptible to liquefaction as well as many soils which are marginal ;
- the limited information available, both historical data on liquefaction and geotechnical data from site investigations ;
- the regional nature of the study, while requiring a reliability more than that which is achievable from geological mapping alone ;
- the constraints of resources and time available for the study.

Given the above requirements, a methodology was developed for the study, and comprised :

- compilation and review of records of liquefaction during historical earthquake events affecting the Wellington region ;
- use of surface geology maps for five study areas of the region prepared by the Department of Scientific and Industrial Research for the Wellington Regional Council ;
- compilation of accessible geotechnical information from site investigations carried out for various projects in the region, and carrying out some additional site investigations to fill in any gaps in the information ;
- selection of key points with adequate information for a detailed liquefaction assessment ;
- assessment of the liquefaction susceptibility based on the site investigation information collated and the surface geological map ;

- ❑ the ground shaking hazard maps with associated peak ground acceleration estimates prepared by the Department of Scientific and Industrial Research (1992) during previous studies for the Wellington Regional Council were used to represent liquefaction opportunity. This takes into account attenuation with distance, and amplification based on the ground conditions ;
- ❑ evaluation of the potential for liquefaction at the key points, using geotechnical methods ;
- ❑ extrapolation of the liquefaction assessment at the key points, based on the available site investigation information and the surface geology, to derive liquefaction hazard zones ;
- ❑ verification of the liquefaction assessment based on the available historical records of liquefaction ;
- ❑ assessment and mapping of ground damage due to liquefaction.

This methodology uses the combined information from historical liquefaction records, surface geology mapping, and geotechnical assessment. This is a better and more reliable approach than using only one of the three techniques used in overseas studies as discussed in section 4. This method particularly suits situations where only a limited amount of information is available.

The liquefaction opportunity from the ground shaking hazard study ensured that the magnitude, distance and soft soil amplification effects were taken into consideration. The liquefaction induced ground damage was assessed and shown on a separate map.

6 REVIEW OF HISTORICAL RECORDS

Given the uncertainties in the assessment of the potential for liquefaction, it is important to look at any evidence of liquefaction in the past. Further, overseas studies indicate that sites which had liquefied in the past could liquefy again during future earthquakes. Therefore, a comprehensive search and review of recorded instances of liquefaction was carried out.

6.1 Past earthquakes

Previous work on liquefaction case histories in New Zealand showed that liquefaction events were associated with a MM intensity of at least VII. The earthquakes which gave an MM intensity of VII or more in the Wellington Region, since 1840, are listed in Table 2. There appears to be little information prior to 1840 because most of the large scale settlement in New Zealand took place after this time. Many of the larger earthquakes in the Wellington region occurred before significant and widespread human settlement and development of the region.

Table 2 Earthquakes felt in the Wellington region with MM intensities of VII and above

Earthquake	Magnitude (Richter)	Max felt intensity (MM)
1848 Marlborough	7.1	X
1855 Wairarapa	8	XI+
1904 Cape Turnagain	7.5	IX
1914 Cook Strait	6	VIII+
1934 Pahiatua	7.5	VIII-IX
1942 June Masterton	7	VII-VIII
1942 August Masterton	7	VIII

6.2 Past records of liquefaction

Various sources of historical information were searched, collated and reviewed. Since liquefaction was not a recognised phenomenon until recent times, it was necessary to look for reports such as sand boils, subsidence and lateral spreading described by various people in different ways. Some 30 cases of liquefaction were identified from the records. Because many of the large earthquakes occurred in the last century, many instances of liquefaction would have either been not observed, or not recorded.

Some of the records are indicative of liquefaction, while others clearly show that liquefaction did occur. For example, records such as,
" *In the lower part of the valley of the Hutt, numerous hillocks of sand were thrown up, forming cones, varying from 2 to 4 feet in height, and in many parts of the valley large fissures were formed, with partial subsidence in many places.*" (1855 Wairarapa Earthquake) and
" *Opposite this building on the road, a considerable opening emitted slimy mud, and the main street was overflowing by inundation*" (1855 Wairarapa Earthquake), are clear indications of sand boils observed during liquefaction. The reports such as,
" *Between Paraparaumu and Waikanae 19 lengths of rails subsided, and between Waikanae and Te Horo 10 lengths had subsided*" (June 1942 Masterton Earthquake) suggest liquefaction induced subsidence.

6.3 Geological evidence of liquefaction

In addition to the records of liquefaction observations during past earthquakes, there was an interesting geological observation of possible liquefaction (Brown - pers comm). It is reported that during excavation of trenches adjacent to the Melling

Railway Station in Lower Hutt, a surface silt layer underlain by sandy gravel was found to have about 5 mm diameter vents through the silt, filled with sand. This suggests that the underlying sandy gravel may have liquefied during some earthquake in the past, during which sand and water were ejected through the silt layer to the surface.

6.4 Conclusions from review of past liquefaction

The review of the historical records of liquefaction clearly demonstrated the occurrence of earthquake induced liquefaction damage.

Some of the reports of liquefaction (see Section 6.2 above) refer to ejection of "slimy mud" during the 1855 Wairarapa earthquake. While the descriptions in 1855 may be inaccurate, these reports do suggest that fine grained materials such as silt have liquefied in the past, substantiating such occurrences quoted in overseas published literature.

The geological evidence of liquefaction in the Hutt Valley (see Section 6.3 above), suggests that coarser grained soils such as sandy gravels could also liquefy.

There has been widespread development, including reclamation and development in areas such as Porirua and the Wellington harbour front, since the large earthquakes in the Wellington region occurred. Therefore, more widespread liquefaction, and hence ground damage can be expected from future large earthquakes.

7 COMPILATION OF GEOTECHNICAL DATA

Geotechnical information on the ground conditions were collated from site investigations and laboratory tests carried out for various projects in the region. The locations of the site investigation information collated were plotted, and where there were significant gaps in the information in areas considered to be susceptible to liquefaction, some additional investigations were carried out.

The information collated was reviewed, and together with the surface geology, was used to choose key locations. These key locations represent areas with different ground conditions which appeared to be vulnerable to liquefaction. The density of the key points depended on the variability of the ground conditions and the degree of surface development.

8 LIQUEFACTION HAZARD ASSESSMENT

8.1 Liquefaction susceptibility

The liquefaction susceptibility was assessed based on the groundwater level, soil type, particle size distributions, and a general consideration of the density of the soils as indicated by the geology and SPT 'N' values. Sands and silty sands below the groundwater level, with SPT 'N' values less than about 25 were generally considered to be susceptible. As discussed in section 6.4 above, historical evidence suggests silts to be also susceptible to liquefaction, and hence sandy silts and silts of low plasticity and SPT 'N' values less than about 20 were also classified as susceptible to liquefaction. These soils, when present within a depth of about 15 m of the ground surface were classified as being susceptible to liquefaction. Liquefaction susceptibility maps were prepared by considering this assessment and the surface geology.

8.2 Liquefaction opportunity

The liquefaction opportunity was directly obtained from the results of the ground shaking study carried out by the Department of Scientific and Industrial Research (1992).

8.3 Liquefaction potential

The liquefaction assessment was carried out mainly using the Seed and Idriss simplified procedure for evaluating liquefaction potential (National Research Council, 1985). This method takes into consideration the peak ground accelerations, the magnitude of the earthquake (which also allows for the effect of duration of shaking), the effective overburden stress, and the groundwater levels. The peak ground accelerations are based on the results of the ground shaking hazard study, which is summarised in Table 1, for the Wellington city study area. The cyclic stress ratio caused by a given earthquake scenario at a given location can be calculated.

For each key location, the cyclic stress ratio was calculated for the layers of soils which are susceptible to liquefaction, and the calculated values for the most critical layer at each key location were plotted against the SPT 'N' value corrected as described by the National Research Council (1985) to allow for the effect of overburden pressure. An example plot for a Scenario 1 event in the Wellington city study area is shown on Figure 2. Seed and Idriss have developed curves separating liquefiable and liquefaction resistant soils for different magnitude earthquakes, based on empirical correlations. A family of curves for fines contents of 5%, 15% and 35%, for a magnitude 7 event are shown on Figure 2.

The plot of cyclic stress ratio v corrected SPT 'N' values for the Wellington city study area shows that even a magnitude 7 earthquake at a distance of 100 km, can cause liquefaction in a number of areas of the Wellington city. This is because the soft soils could amplify the ground shaking from a distant earthquake, giving significant peak ground accelerations of up to about 0.1g to 0.3 g. Depending on the fines content, and the location of the plotted point relative to the curves, each key location is classified as liquefiable or liquefaction resistant, for each scenario.

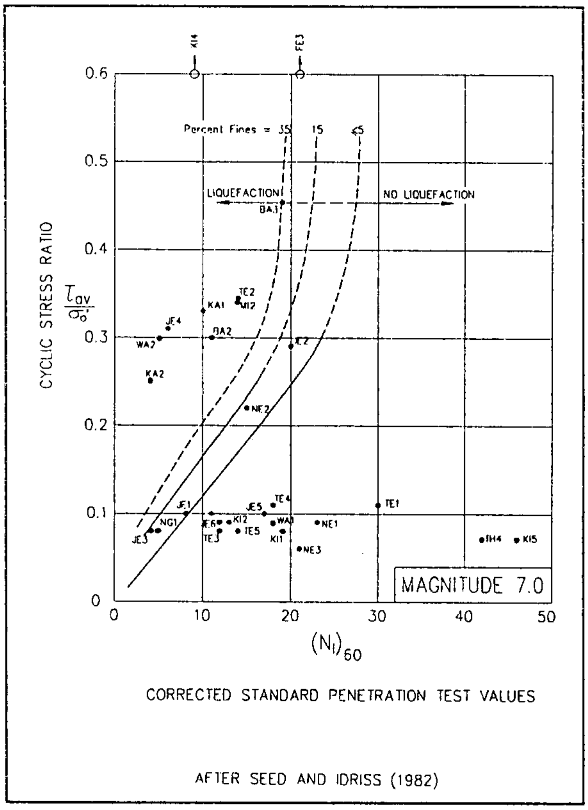

Figure 2 Liquefaction assessment for Wellington city

Where Static Cone Penetration Test results were available instead of SPTs, the method proposed by Sugawara (1989) was used.

The potential for liquefaction is classified as :

- ❑ High — liquefaction during both Scenarios 1 and 2
- ❑ Moderate — liquefaction unlikely or marginal during Scenario 1
- ❑ Low — liquefaction may occur only during Scenario 2
- ❑ Variable — liquefaction potential varies from low to high due to variable ground conditions

The liquefaction assessments at the key points were used to map the liquefaction potential zones, using the information from other site investigations, and the surface geology map. As an example of the hazard maps prepared, the liquefaction potential map for part of the Wellington city study area is shown on Figure 3.

9 GROUND DAMAGE

The likely type and order of magnitude of ground damage due to liquefaction were assessed. The estimates are considered to be only a rough order assessment of likely ground damage, as accurate methods of prediction are not currently available. The ground damage will depend on the ground conditions as well as the topography at each site. The main types of liquefaction induced ground damage are subsidence, slope failure, and lateral spreading of natural banks and embankments built on liquefiable ground. The presence of a liquefaction resistant surface layer could minimise ground damage, depending on the thickness of the surface layer relative to the thickness of the liquefiable layer below. However, where lateral instability is likely, the presence of a surface layer may not necessarily preclude ground damage.

The likely order of subsidence due to liquefaction was estimated from a chart proposed by Tokimatsu and Seed (1987) for a magnitude 7.5 earthquake. Therefore, the estimates may be somewhat conservative for a Scenario 1 event of magnitude 7.

Most of the areas susceptible to liquefaction in the Wellington region are in relatively flat areas between the foothills and the present coastline. Lateral spreading is likely to affect areas adjacent to stream / river banks and coastal areas.

In the absence of analytical methods which can be easily applied to assess the lateral spreading, the following simple rules have been formed to enable ground damage maps to be prepared :

- ❑ liquefiable areas within 50 m of water fronts and banks could be affected by lateral spreading during a Scenario 1 earthquake ;
- ❑ liquefiable areas within 200 m of water fronts and banks could be affected by lateral spreading during a Scenario 2 earthquake.

10 PREPARATION OF HAZARD MAPS

Hazard maps for liquefaction susceptibility, liquefaction potential and ground damage were prepared for the Wellington City, Hutt Valley, Porirua Basin and Kapiti Coast study areas. Maps were not prepared for the Wairarapa study area, because there were only localised areas which are susceptible to liquefaction.

The maps were produced to a scale of 1 : 20,000 to 1 : 25,000. For the Wellington City centre, a larger maps was also prepared to a scale of 1 : 10,000. The maps were digitised and produced using computer aided draughting techniques, and installed in the Wellington Regional Council's geographical information system.

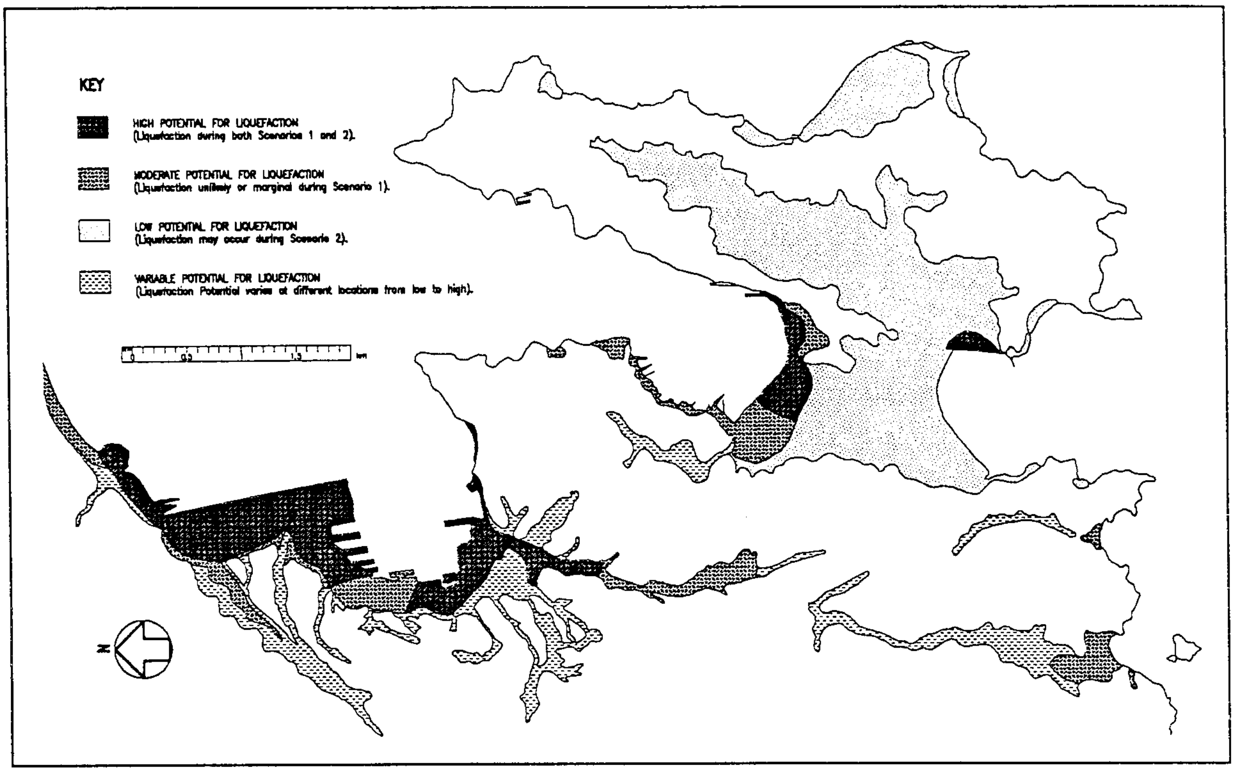

Figure 3 Liquefaction potential map, Wellington city

11 SUMMARY

A liquefaction hazard study has been carried out as part of the Wellington Regional Council's strategy for achieving an acceptable level of seismic and geological hazards in the region. The hazards were mapped using a method specifically developed, for two earthquake scenarios, a large distant event producing MM V-VI shaking in Wellington, and a large magnitude 7.5 event on the Wellington fault which runs through the region.

The liquefaction hazard was mapped using surface geological maps, historical records of liquefaction and geotechnical engineering methods. The study has confirmed significant liquefaction hazards in the Wellington Region. The maps have been published by the Wellington Regional Council, and will help in the recognition and mitigation of liquefaction hazards.

ACKNOWLEDGEMENTS

The author wishes to acknowledge the initiative of the Wellington Regional Council who have undertaken the task of identifying the seismic and geological hazards in the interest the community, and thank them for permission to publish this paper. The support of the Earthquake Commission (EQC) and the New Zealand Geomechanics Society through a Young Geotechnical Professionals Award is also gratefully acknowledged. The author also wishes to thank Works Consultancy Services Limited for permission and support to publish this paper.

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Engineering geology and remedial works design, Brewery Creek Slide, Clyde Power Project, New Zealand

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ABSTRACT:

Remedial measures at the Brewery Creek Slide are part of works to stabilise a number of large schist landslides along the shores of Lake Dunstan, Clyde Power Project, New Zealand. Remedial measures for part of the slide comprise a low level pumped drainage drive/drillhole network in conjunction with a zoned earthfill blanket and grout curtain. The works were designed to, and have successfully, lowered and controlled groundwater at the toe of the slide. Comprehensive investigations provided data for geological and geohydrological models on which the design was based. The design was independent of model uncertainties yet enabled an area which developed adverse conditions during lake filling to be readily and successfully treated.

1 INTRODUCTION

The Clyde Power Project is a hydro-electric development on the Clutha River, South Island, New Zealand. The project which is owned by the Electricity Corporation of New Zealand (ECNZ) comprises a 102m high concrete gravity dam, powerhouse, and a 35km long reservoir up to 60m deep. Approximately 20km (25%) of the shoreline of the reservoir (Lake Dunstan) is bordered by existing large landslides (Figure 1). Extensive stabilisation work was carried out on the Brewery Creek Slide and six of the other landslides to offset the destabilising effects of lake filling. A general overview of the landslides on the project is presented by Gillon and Hancox (1992).

The Brewery Creek Slide, situated 15km upstream from the dam (Figure 1), is approximately 2.4km wide, 1.3km long, up to 150m deep and covers an area of 220 hectares. The slide mass contains an estimated volume of 150Mm^3 of gravitationally displaced material. Reservoir filling has raised the water level at the toe by 35m.

Although the slide was dormant, geological precedent indicated that lake filling would result in a more adverse groundwater regime than that during the last phase of movement (< 18,000 years ago). This together with stability analysis and the overall strategy adopted for stabilisation works on the project established the need for remedial works to offset the destabilising effects of lake filling. The works comprise a buttress and gravity drainage drive system at the upstream end (Zone 1) and a low level pumped drainage drive/drillhole network with associated grout curtain and low permeability earthfill blanket across Zones 2 and 3 (Figure 2).

The general nature of remedial works constructed to stabilise the Brewery Creek Slide have been outlined in previous papers (Graham et al, 1991; Gillon et al, 1992) so this paper focuses on the interaction between engineering geology and the Zone 2/3 remedial works.

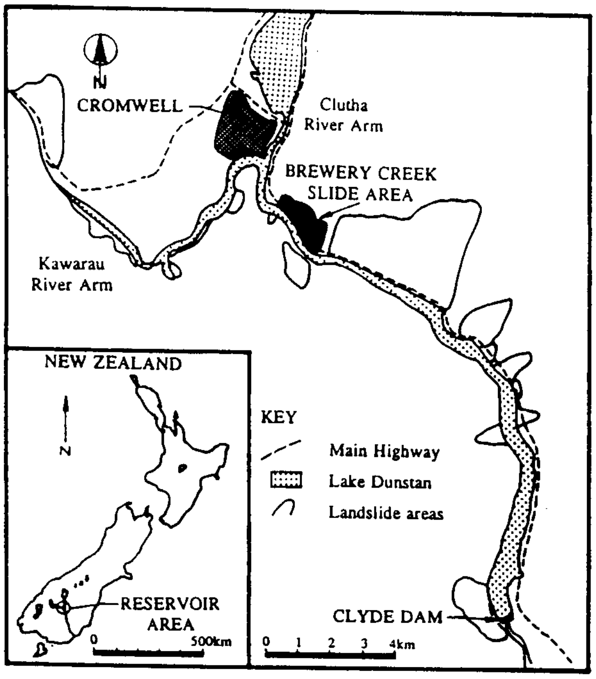

Figure 1. Locality plan.

2 GEOLOGY AND GEOHYDROLOGY

At the time of remedial work design the geology and geohydrology models of the landslide were based on a comprehensive investigation programme comprising:

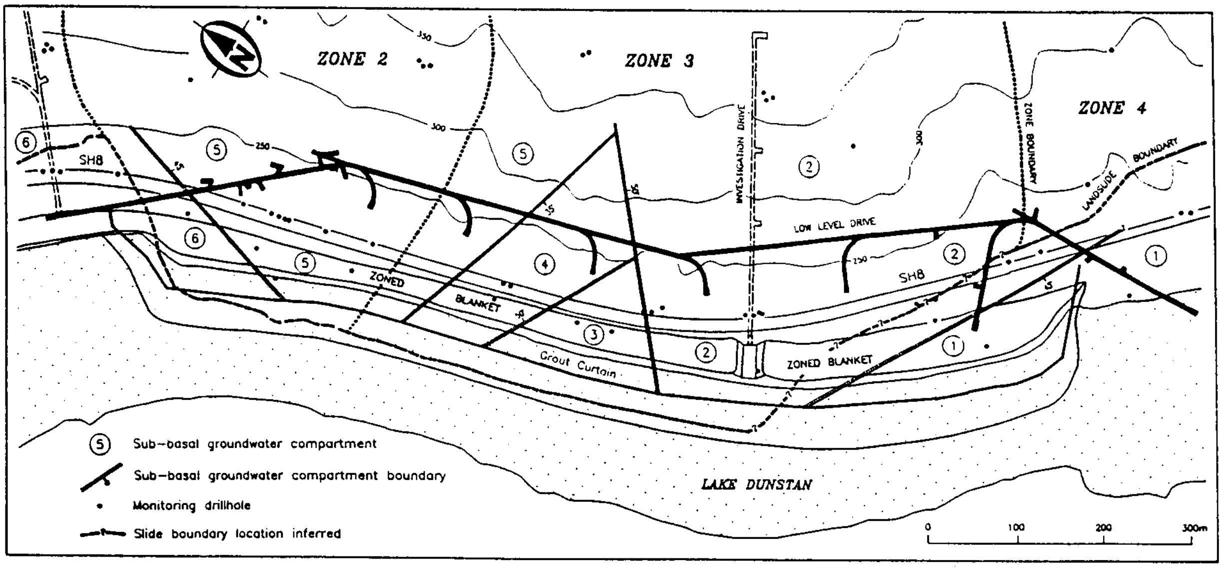

Figure 2. Plan of Zone 2/3 remedial works.

- Engineering geological mapping.
- Trenching.
- 9666m of cored and non-cored drilling.
- Two investigation drives (717m).
- 11.6km of surface and underground seismic refraction surveys.
- Laboratory testing.

2.1 Bedrock geology

Bedrock comprises predominantly unweathered to slightly weathered, grey, foliated, quartzofeldspathic mica schist. The schist is typically moderately strong to strong but is highly anisotropic with marked strength differences parallel and perpendicular to foliation. Foliation under the slide dips gently (5-15°) into the slope except in the toe where it dips gently (0-10°) out of the slope due to anticlinal folding (Figure 3).

The bedrock is cut by sets of tectonic defects comprising variable proportions of crushed, sheared and gouged schist 200mm-5m thick. The major defect sets dip at moderate to steep (45-60°) or low (20-35°) angles and generally strike obliquely to the lake axis (Figure 2). One low angle set which dips parallel to the slope (cross-cutting foliation) is inferred to be the major factor controlling landslide initiation and slide base geometry.

2.2 Landslide model

Subdivision of the landslide into six zones was based on geomorphological evidence and the nature of the slide mass which varies from highly disrupted "chaotic debris" through less disrupted "blocky debris" to least disrupted "displaced schist". The amount of disruption is inferred to be related predominantly to the amount of downslope movement the material has undergone.

In the mid and upper slopes the slide base geometry is relatively planar reflecting the nature of the inferred controlling defect. At the toe the slide base forms a broad partly rotational breakout zone which typically dips back into the slope indicating an upthrust component (Figure 3).

Due to the comprehensive investigation programme the model for slide base geometry made available for remedial works design had a high degree of confidence. The main area of uncertainty in the model was the location of the downstream margin of Zone 3 in the area between the river and highway (Figure 2). To facilitate remedial work design in this area the downstream margin was shown as a zone covering the area in which the actual margin could be located. Design was based on the most downstream and deepest (most conservative) possible location.

2.3 Groundwater model

At the time of remedial works design the Zones 2/3 groundwater model comprised sub-basal and perched aquifers.

The sub-basal semi-confined aquifer was coincident with river level up to 700m back from the river. The aquifer exerted uplift pressure on the slide base over a relatively small but critical area in the toe of the slide (Figure 3). Further upslope the piezometric level was well below slide base due to the almost flat gradient back from the river. The only subdivision of the semi-confined aquifer possible prior to drainage drawdown was at the upstream end of Zone 2 (later to be identified as Groundwater Compartment 5) where the piezometric level was 6m higher than the remaining area (Figure 2). All other piezometers showed piezometric levels coincident with the river. In addition most piezometers in the toe

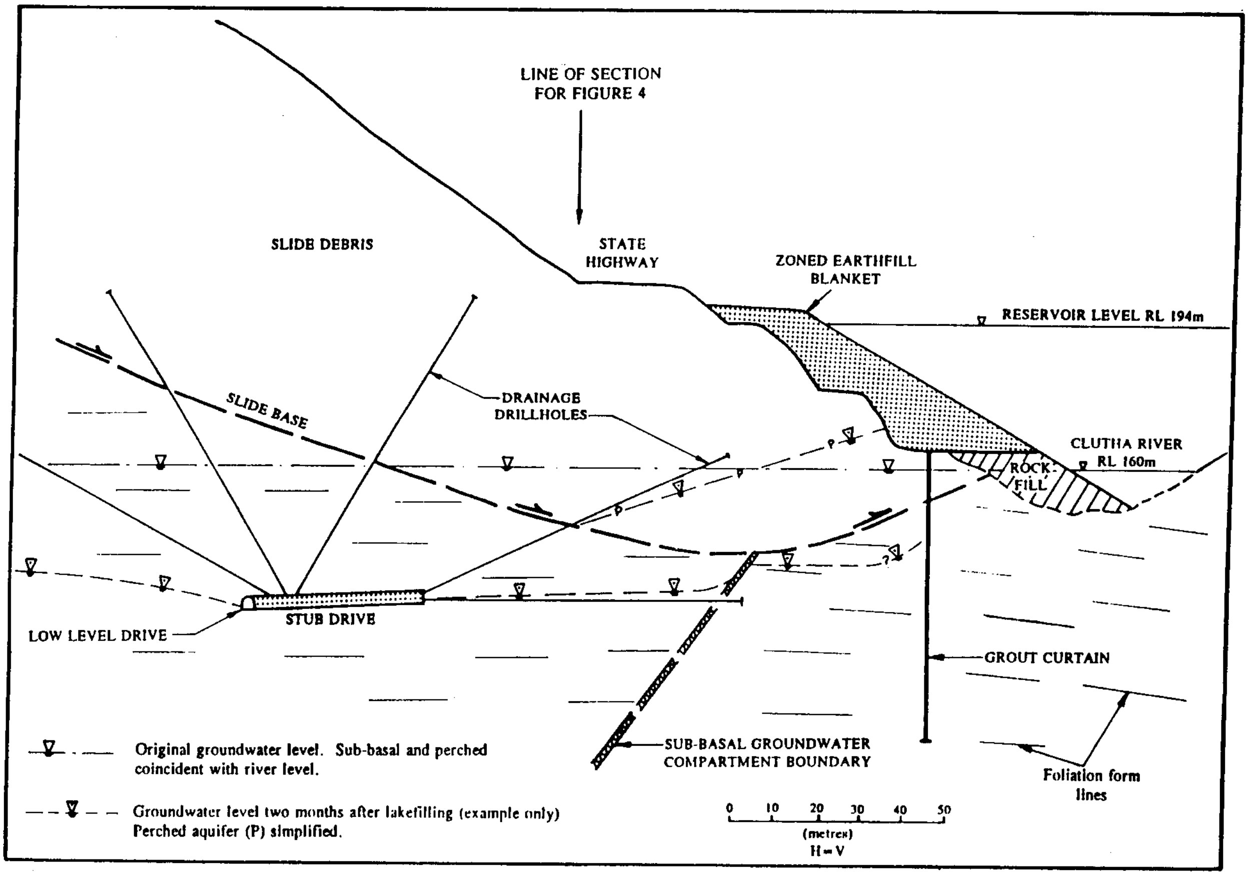

Figure 3. Toe cross section.

and some further upslope showed a rapid and positive response to changes in river level indicating a strong hydraulic connection with the river.

The perched aquifer was mainly restricted to the trough formed by the slide base in the toe of the slide (Figure 3). Like the semi-confined aquifer the perched aquifer was coincident with river level in all but the upstream end of Zone 2 and most piezometers showed a positive response to changes in river level. Due to the paucity of data points the perched aquifer was not subdivided but experience from other landslides indicated it would be highly compartmentalised.

3 REMEDIAL WORKS DESIGN

3.1 General

Low level pumped drainage was chosen to stabilise Zones 2/3 of the slide. Gravity drainage to reservoir level was not feasible because there was no drainable head and buttressing was not possible because of the narrow river valley. The chosen option comprises a low level pumped drainage drive/drillhole network together with a zoned earthfill blanket and grout curtain (Figures 2 and 3). These works consist of:

- 1.9km of tunnelling (up to 30m below original river level) and a 70m deep shaft,
- 12km of drainage drilling,
- 1.3km long, 140,000m^2 grout curtain (100km of drilling), and
- 2Mm^3 of earthworks

The works are designed to lower and control both sub-basal and perched groundwater levels in the toe of the slide so that post lake filling stability is greater than or equal to that prior to remedial works. The key effect of the works is the stabilising thrust of the reservoir generated by drainage at and behind the toe of the slide.

The low level drainage drive acts as a drainage element and provides ongoing access for maintenance and, if necessary, additional drainage drilling or tunnelling. Water collected in the drives is pumped back into the reservoir. The combined effect of the drives and drilling is to increase the rock mass permeability both sub-basally and within the landslide. The low permeability zoned earthfill, which includes core and filter zones, and grout curtain impede flow from the reservoir to the drive/drillhole system.

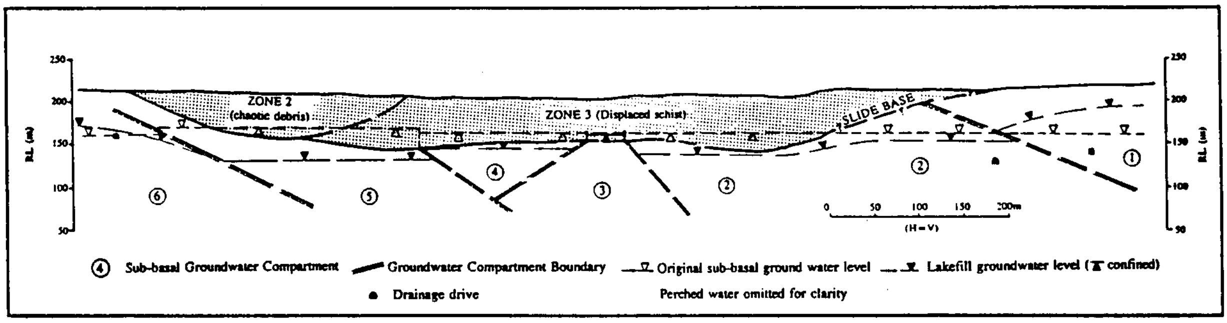

Figure 4. Toe transverse section.

3.2 Geological factors influencing low level drive design

The low level drive was located beneath the landslide and away from the disturbed rock underlying the slide base. The position of the declines (access to the low level drive) was constrained by the Zone 2/3 lateral boundaries and the location of the low level drive by the deepest section of the slide. Five stub drives were constructed perpendicular to the main low level drive to establish drilling chambers close to the slide base (Figures 2 and 3). The length of the stubs was dependent on the slide base geometry and optimization of tunnelling and drilling costs. An array of drainage holes was drilled from the stubs to provide drainage beneath and within the landslide. The orientation of the drilling arrays was dependent on slide base geometry in the stub area.

3.3 Design and model uncertainties

The remedial works were designed/located to be independent of model uncertainties but also enable unforeseen problem areas to be treated.

A significant advantage of the layout of the low level drive was that it provided a network of easily accessible drives from which additional drilling and/or tunnelling could be carried out if geological and/or groundwater conditions developed which resulted in design objectives not being met.

4 MODEL REFINEMENTS FROM CONSTRUCTION AND LAKE FILLING INFORMATION

4.1 Slide base model

Excavation of the decline and shaft excavation at the downstream end of the low level drive system enabled the landslide margin in Zone 3 to be more accurately delineated. However due to poor definition of the slide base in this area and conflicting data from drillhole information the location of the margin is still not known with as much confidence as further upstream.

Groundwater response to drainage drawdown and lake filling confirmed the modelled slide base location in the remainder of Zones 2/3. The slide base was also found to be a significant aquitard. Although this was previously assumed, the coincidence of perched and sub-basal groundwater (with river level) had made it very difficult to confirm prior to drainage drawdown.

4.2 Groundwater model

Stressing of the groundwater systems during drainage drawdown and lake filling revealed significantly more compartmentalisation in the sub-basal semi-confined aquifer than had been inferred prior to drawdown (Figure 4).

The semi-confined aquifer groundwater compartments (GWC) were identified by analysing piezometric response to groundwater stress events such as tunnel excavation, drainage drilling and lake filling. These response diagrams often enabled piezometric response in one or many drillholes to be attributed to the penetration of a discrete crushed zone. Response diagrams and other methods used for determining groundwater characteristics on the Clyde Power Project landslides are outlined by Macfarlane, Pattle and Salt (1992)

In contrast to the apparent homogeneity (except for GWC 5) indicated prior to remedial works, the identified compartments of the semi-confined aquifer showed significant variability in drained and post lake filling water levels (Figure 4). The different water levels are due to factors such as:

- Rock mass storativity and transmissivity,
- Length and effectiveness of drainage elements intersecting the GWC's,
- Area of grout curtain frontage vs GWC volume (dictated by GWC boundary defect orientations) and,
- Relationship between grout curtain and drainage element effectiveness per unit area.

5 REMEDIAL WORK TO ACCOMMODATE MODEL REFINEMENTS

5.1 Slide base model

Confirmation of the slide base at a higher level at the Zone 3 margin enabled final groundwater levels in GWC 1 to remain higher than initially planned (i.e. less drainage drilling was required to achieve design groundwater levels).

5.2 Groundwater model

In general additional work was not required as a result of compartmentalisation of the semi-confined and perched aquifers because adequate drainage drawdown and control was achieved by the original drainage/cutoff element design. However after raising the lake to RL 177m higher than expected groundwater levels occurred in the area of GWC 3 (Figures 2 and 4). This indicated that unacceptably high uplift pressure could be acting on the slide base in a relatively small area of the slide toe where there was no suitably located instrumentation. Two new piezometer holes were drilled and 8m of uplift pressure on the slide base was confirmed, as were revised boundary defects for GWC 3. The previously inferred GWC 3 was larger and was intersected by the drive. The newly defined GWC had a large grout curtain "frontage" and minimal drainage element penetration (6 drainage holes and no tunnel) compared to its total volume.

The drive layout enabled additional drainage drilling to be carried out promptly and the GWC 3 groundwater level was lowered 15m with a series of ten holes. If necessary it would also have been possible to extend DR 583 into GWC 3 to provide additional drainage.

6 CONCLUSIONS

Comprehensive engineering geological investigations enabled development of geological and geohydrological models with a high degree of confidence on which to base remedial works design for a large schist landslide. The adopted design layout was purposely independent of the identified model uncertainties yet still enabled adverse conditions which developed in one area during lake filling to be readily treated to achieve the overall stabilisation objective. The remedial works have successfully lowered and controlled groundwater levels in the toe of the slide.

ACKNOWLEDGEMENTS

The author is part of a team which has developed solutions for landslides around the shoreline of Lake Dunstan for Electricity Corporation of NZ Ltd, Clyde Power Project. Information gained, interpretations made and design solutions reached by members of the Brewery Creek Landslide team have made the presentation of this paper possible.

The permission of the Electricity Corporation of NZ Ltd to publish this paper is gratefully acknowledged.

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Prediction of Liquefied Strata from the 1987 Edgecumbe Earthquake

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ABSTRACT: On March 2 1987, an earthquake of magnitude (M_L) 6.3 occurred near the town of Edgecumbe, in the North Island of New Zealand. Extensive ground damage in the form of liquefaction and surface faults resulted from the shock, especially in the recent sediments of the Rangitaiki Plains surrounding Edgecumbe. This paper focuses on a major lateral spreading site, at the Landings Road Bridge, which lies on the eastern side of the Rangitaiki Plains. The liquefaction beside the bridge produced both sand boils and lateral spreading of the overburden towards the river. Data from the probes and the use of empirical and semi-empirical models enabled the prediction of which strata liquefied during the earthquake. A three-dimensional model of the lateral spreading in this area was established, and a verification of lateral spreading models was accomplished.

1 INTRODUCTION

This paper provides background information on the events of the 1987 Edgecumbe earthquake and then looks at the effects at the Landing Road Bridge site in detail. Firstly, a general overview of the geology of the area is given, which is then followed by a summary of the earthquake events and resulting damage. Details of the effects that the earthquake had on the Landing Road Bridge site are then examined.

1.1 Background information and geology

The subduction boundary between the Pacific Plate and the Australian Plate, see inset on Figure 1, makes the central North Island of New Zealand a seismicity active region. Within this region, the Taupo Volcanic Zone is an area of marked high seismic and volcanic activity. The Edgecumbe earthquake sequence occurred in the Whakatane Graben, which lies to the northern end of the Taupo Volcanic Zone. The Whakatane Graben is composed of the alluvial Rangitaiki Plains surrounded by hills and is crossed by three major rivers the Whakatane, Rangitaiki and the Tarawera Rivers. At the time of the earthquake in excess of 25 000 people lived throughout the plains, concentrated mainly in the towns of Whakatane, Edgecumbe and Kawerau (Pender and Robertson, 1987).

The hills to the west of the Rangitaiki Plains are composed of late Quaternary rhyolitic volcanics while the hills to the east are composed of mesozoic greywackes. Within the graben mesozoic greywackes have been down-faulted to a depth of two kilometres below the current sea level (Nairn and Beanland, 1989). Infilling of the basin has occurred with Quaternary volcanics and sediments from the catchments to the south. Subsidence along the graben axis has been occurring at a rate of 2-3 mm/year for the last 5 500 years, whereas the hills to the sides of the graben have been rising by in excess of 0.5 mm/year for the last 120 000 years and the graben is widening at 7 mm/year. Ground water logs across the plains indicate lithological variability both vertically and horizontally, with deep bores indicating alternating layers of pumice derived alluvial sand and gravel, tephra, marine silt and sand.

A mixture of dextral and sinistral faulting implies either local reversals of the stress field or that the zones widens via a complex system of mainly dextral faults in the west, sinistral in the east and normal faults in the middle (Pender and Robertson, 1987). The basement structure dips north-east from Kawerau into the Whakatane graben. The Edgecumbe fault which pre-existed before the 1987 earthquake (but was unrecognised), dips to the west at $45^\circ \pm 10^\circ$ and swings upward from 100 m deep to become vertical at the surface. Fault traces are controlled by surface geology, which is indicated by their non-linearity as seen in Figure 1.

The present surface of the Rangitaiki Plains was formed around 7 000 years ago when the sea attained its present level. The shoreline has since prograded about 10 km with major sedimentation episodes following the Whakatane (c.1850 years BP) and Kaharoa (c.800 years BP) pyroclastic eruptions. Heavy forest covered the Rangitaiki Plains 150 years ago, but the land is now used for pastoral farming.

1.2 1987 Edgecumbe earthquake sequence

Seismic activity started in the region one week prior to the main event and culminated in a foreshock of magnitude (M_L) 5.2 seven minutes before the main event. The main event of magnitude (M_L) 6.3 occurred at 01h 42m 34s UT (1.42 pm local time), with a focal depth of 8 ± 1 km (Pender and Robertson, 1987). Strong ground motion (>0.05g) lasted 8-9 seconds, with a maximum of 0.33g being recorded at Matahina Dam, which was the closest accelerograph to the epicentre (within 15 km of the epicentre). This is the strongest ground motion ever recorded in New Zealand. Response spectra of the recording at

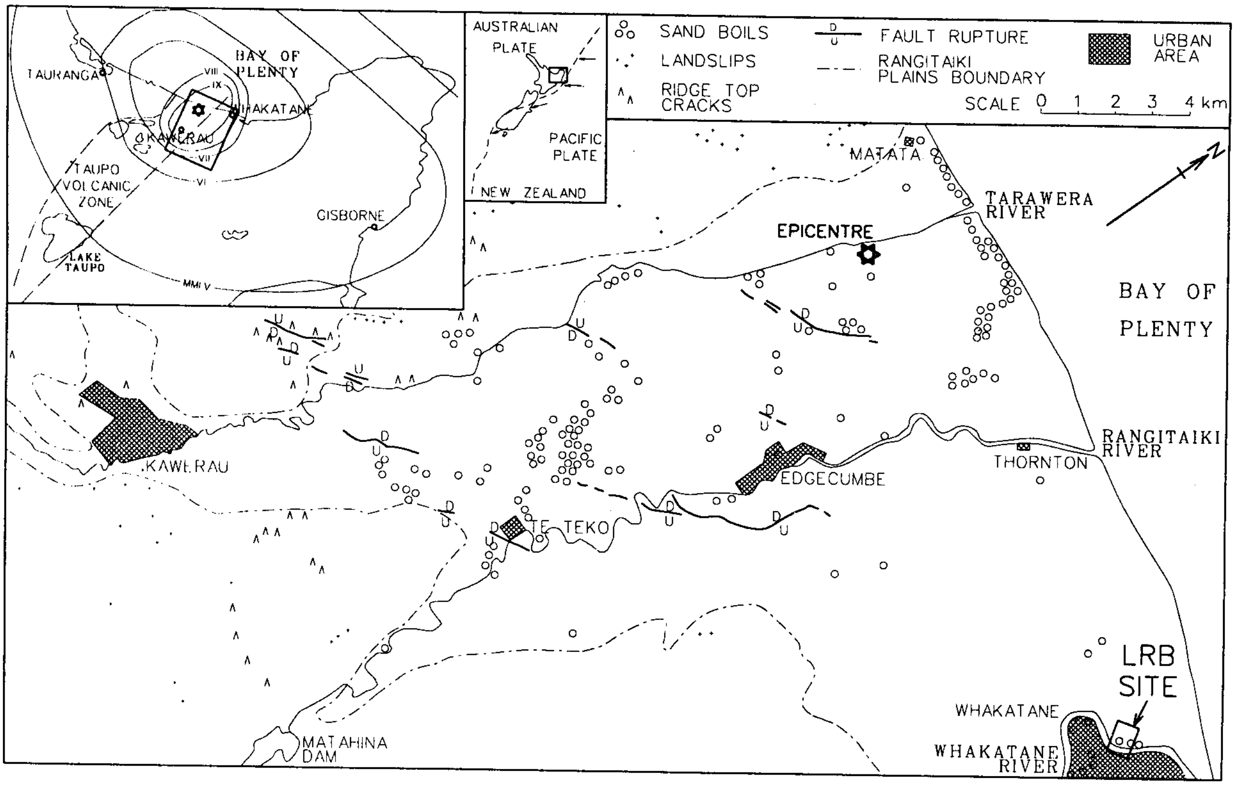

Figure 1: Map of the Rangitaiki Plains showing the ground damage caused by the 1987 Edgecumbe earthquake. Large inset showing Bay of Plenty region and isoseismal from the earthquake.

Matahina Dam, are similar to that of El Centro NS 1940 in periods less than 0.5 s and greater than 1.2 s. In the hours following the earthquake four aftershocks of magnitudes between (M_L) 5.1 and 5.6 were recorded. In total 600 shocks of magnitude (M_L)>3.0, in an area of 70 x 50 km, were noted throughout the earthquake sequence.

This earthquake produced Modified Mercalli Intensities of up to MMI X in the Edgecumbe area, this high epicentral intensity being due to the shallow focal depth. Rapid diminution of intensity with distance was due to high attenuation in the near surface sediments (Pender and Robertson, 1987).

1.3 Observed damage of the earthquake

Heavy structural damage resulted in the large dairy factory in Edgecumbe, while paper and paper board mills at Kawerau and Whakatane suffered damage to a lesser extent. Other structural damage included: compression buckling of railway tracks and pavements; shell failures and tops pushed off liquid containers; a locomotive and trucks overturned; intensive breakage of pipes in Edgecumbe; destruction of a shopping mall in Edgecumbe; toppling of unreinforced concrete block walls; cranes and electricity transformers pushed off rails; chimneys were toppled and a plastic hinge formed in one of the pier-pile interfaces on the Edgecumbe rail bridge. Damage to structures in Whakatane was mainly restricted to areas adjacent to the Whakatane River.

Ground damage was mainly confined to the recent soft sediments of the Rangitaiki Plains. Ten fault traces, a mixture of pre-existing and new, accounted for total surface fractures of 18.4 km. The largest fault, the Edgecumbe fault was 7 km long with a average vertical displacement of 1.5 m extending to an estimated 2.1 m at the maximum. Along with the normal faults and compressional rolls, there was regional subsidence of up to 2 m at Edgecumbe, which varied gradually to a few centimetres uplift in Whakatane. Level ground liquefaction was widespread throughout the plains and lateral spreading often occurred next to the rivers. In excess of 26 km of stopbanks required repairs and 13 (6 serious) of 39 drainage schemes were damaged. Stopbanks adjacent to the Whakatane River, made of well graded material, suffered only longitudinal cracks, whereas Tarawera River stopbanks (made of coarse pumice sand, 90 years old) were damaged due to slumping (Franks et al.). Repairs for the extensive damage to drainage works due to lateral spreading, fault traces and general subsidence cost in excess $NZ 10M. Other ground damage included ridge top cracks up to 150 mm wide and slope stability problems due to slumping of cohesionless lapilli in the hills.

Liquefied ejecta ranged from medium sands to fine sands, in general carrying pumice lapilli up to 20 mm in diameter; in some areas coarse silts were ejected. Ejecta were quite stratified with coarse sands on the bottom and fine sands on top or vice versa. The liquefaction which occurred followed generally accepted patterns of behaviour (Jennings et al., 1988).

2 LANDING ROAD BRIDGE SITE

This site is of particular interest due to the intensive degree of liquefaction and lateral spreading that occurred in result of the 1987 Edgecumbe earthquake. This site, labelled LRB in Figure 1 and shown in detail in Figure 2, lies beside State Highway 2 (SH2) on the approach to the town of Whakatane. SH2 crosses the Landing Road Bridge which traverses the Whakatane River on the southern side of the site.

The Landing Road Bridge site lies beside a very active river area. At this location beside the river, the water table depends on the tide which can vary by 2 m at the bridge. This site was on the edge of the Whakatane River estuary at the time of the Taupo Pumice eruptions (c.1800 years BP). But more recently the river has prograded towards the south 300 metres in the last 100 years. The site comprises medium to coarse pumiceous sands, overlain by approximately one metre of sandy silt. In general the sands are loosely compacted to a depth of six metres below the ground surface, and from this point they are some what denser. In situ saturated density of the soil averages 1.1 t/m^3 (Jennings and Smith, 1991). The average yearly rainfall for the area is approximately one metre and during the dry summer months, the water table often drops to 1.1 metre below the ground surface. The site is used as pastoral farming land.

The Landing Road Bridge was constructed in 1962 and consists of 13 x 18.3 m non-continuous spans which support two concrete traffic lanes plus two footpaths. The superstructure comprises 5 precast post-tensioned concrete I-beams which are interlinked with linkage bolts through diaphragms over the piers and through the abutment backwalls. The bearings consist of 16 mm rubber pads under the beams which are tied down using holding bolts at all piers and abutments. The substructure is made up of 12 concrete slab piers on 8 x 406 mm square raked prestressed concrete piles. The abutments are supported by 406 mm square raked prestressed concrete piles (2 rows of 5 and 3 piles). The abutment backwall is tight packed and bolted to the beam diaphragm. There are no approach slabs. Five river piers were additionally underpinned with two extra 1.1 m diameter concrete cylinders around 1985 after flooding had undermined one of the piers.

2.1 Observed damage at the Landing Road Bridge site

Modified Mercalli Intensities in the area were estimated at between MMI VII & VIII.

Cracks and sand boils associated with lateral spreading occurred as indicated in Figure 2 and continued in the same manner a few hundreds of metres further downstream of this site. The cracks tended to be parallel with the river's edge at all places except immediately beside the bridge, where they swung around to approximately 45° to the bridge. In total there was about five major cracks sequences, which were in excess of 200 mm wide, in the 300 m beside the true left bank of the river. There was no observed evidence of liquefaction on the true right bank of the river. Particle size distributions of 13 samples of ejecta taken from the left bank (both upstream and downstream) beside the bridge, D_{50}'s from these samples ranged from 0.2 to 0.5 mm. One sample of ejecta obtained from approximately 100 m downstream of SH2 and 100 m on the land side of the stopbank had a D_{50} of 0.1 mm. Some of the ejecta contained pumice particles while others did not contain any pumice. The stopbank contained longitudinal cracks which were in excess of 100 m in length in places. Intensive cracking and subsidence of the north-west approach to the bridge from the abutment to past the intersection of SH2 with Keepa Road. An eye witness report indicates that this approach was passable immediately after the main earthquake, but on returning one hour later it was no longer passable in a vehicle. Gaps between the piers and the soil of up to 600 mm appeared on the river side of the piers on the berm of the left bank of the river. Soil had also piled up behind the piers that were located on the berm of the river. At the time of the earthquake the tide was approaching low tide, and the river level was particularly low due to the preceding dry months.

The bridge superstructure did not undergo any significant distress. Deck joints over the berm on the left bank generally showed tension while the joints over the river piers varied between tension and compression (H. Chapman, pers. comm.). Opening in the joints over the piers may have induced tension in the linkage bolts. Excavation to about one metre, at the northern abutment showed that the front raked piles were cracked on the river side. These cracks extended through 75% of the width of the piles, with there being no indication of cracking on the bridge approach side of the piles. The cracks were repaired with epoxy resin at the time of excavation. Soil at the northern abutment settled 300 to 500 mm exposing the piles. The northern abutment appeared to have rotated (pers. comm. L. McCallin). The southern abutment was not inspected after the earthquake. The tops of the first two piers from the northern abutment were leaning towards the river by about 1°. The rest of the piers appear vertical. Horizontal cracks in piers H & J (which are the first two piers additionally underpinned from the left bank of the river) were not

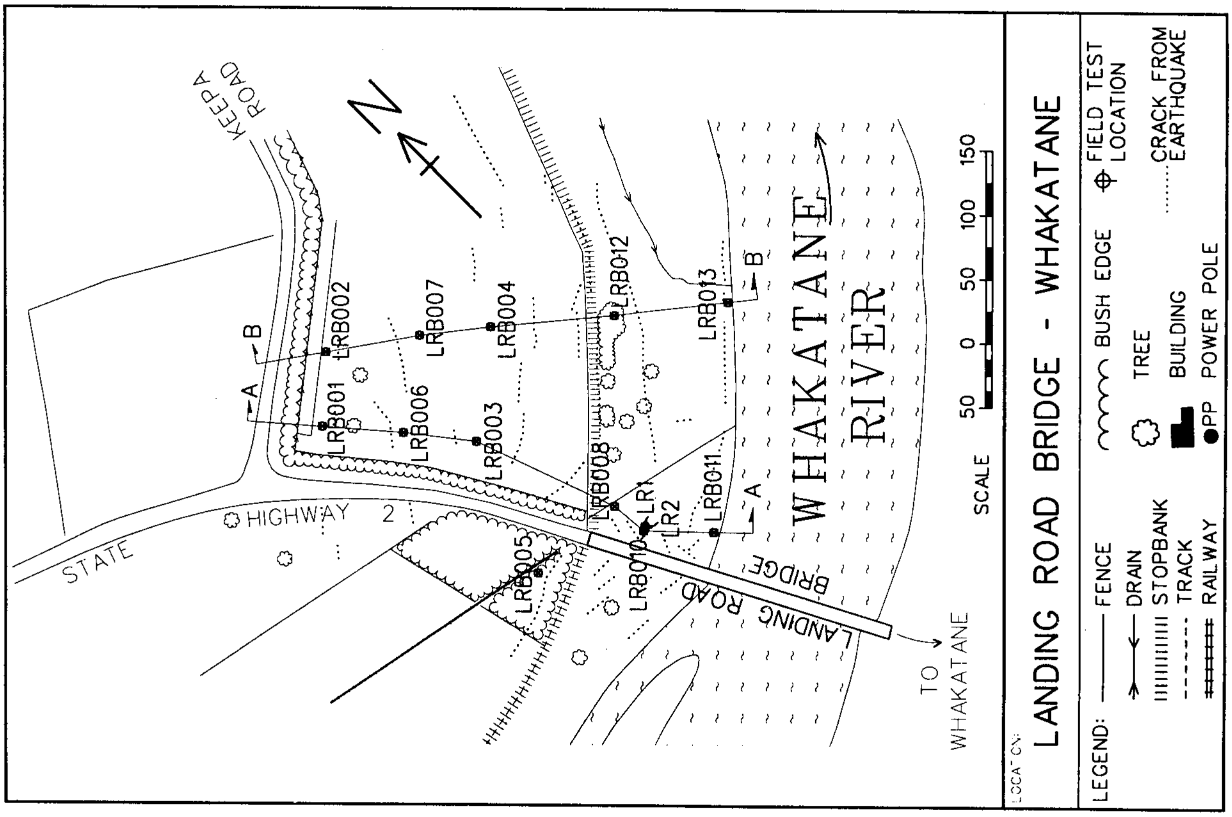

Figure 2: Map showing detail of Landing Road Bridge site.

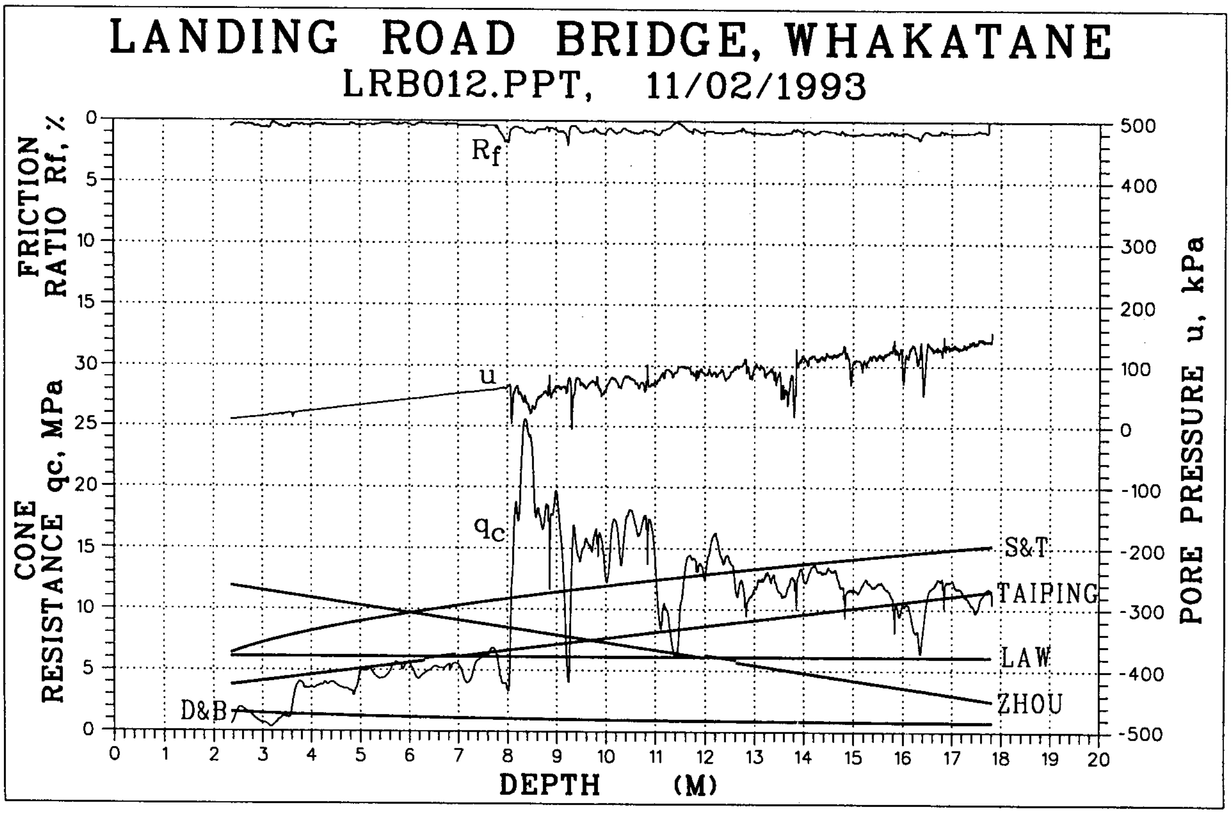

Figure 3: Typical data from CPT with prediction methods superimposed.

noticed until some years after the earthquake, but were considered to have occurred as a result of the earthquake in 1987. These cracks were repaired with epoxy resin in 1992.

3 COLLECTION AND EVALUATION OF DATA

Soil profiling using a truck mounted piezocone was undertaken across the Rangitaiki Plains in 1993. In total 55 probes were made at 15 sites, ranging from intensive investigation of the lateral spreading sites to investigation of sites with only a single sand boil. In addition 5 rotary borings with Standard Penetration Tests (SPT) were completed. At the Landing Road Bridge site 12 piezocone penetrometers (CPT) and one boring were completed.

Measurement of the extent of horizontal and vertical displacement at the Landing Road Bridge site was attempted using photogrammetry. The photogrammetry used photography from January 1982 (scale 1:26000) and photography flown one week after the earthquake (scale 1:11400). There was difficulty in gaining reliable measurements of the displacements due to the lack of fixed detail within the site.

3.1 Estimation of liquefied strata

Five well known methods were used to determine whether a soil is susceptible to liquefaction. These were then plotted along with the CPT data to both ascertain the viability of the method and to determine which soil liquefied during the earthquake. The methods used were: Shibata and Teparaksa, 1988; Zhou, 1980; Taiping et al., 1984; Law et al., 1990; and Davis and Berrill, 1982. The widely conflicting results of these prediction methods are shown in Figure 3. For these results, when the cone resistance, q_c, is less than the predicted critical cone resistance, then this soil layer is deemed to have liquefied.

Due to the lack of consistency obtained by the prediction methods, other factors were introduced to help determine which layers liquefied during the Edgecumbe earthquake. One such important factor is the matching of the Particle Size Distribution (PSD) of the ejected soil with that of the retrieved soils from the SPT tests. Due to the lack of detail as to the location the sand boils from which the PSD of the ejecta were gained, a photograph showing a three layered sand boil was used to try and match ejecta with soil layers from the SPT. From this, the starting point of the liquefaction front was established, and using methods outlined below, the extent of the liquefaction throughout the CPT probe was established. Using this first probe as a guide to the liquefiable soils, the other CPT probes at this site were then analyzed to determine which soils liquefied during the 1987 Edgecumbe earthquake.

Scott and Zuckerman (1973) showed in laboratory tests, that an overlying relatively denser soil layer that would not liquefy under normal loading conditions, can be induced to liquefy from below by the behaviour of the underlying layer. This means that a soil layer that has a cone resistance in excess of the predicted critical cone resistance can liquefy under the correct conditions, if the underlying soil layer is liquefied.

Vreugdenhil (paper this conference) proposes that if a thin dense layer overlies a relatively softer layer then the cone resistance, q_c, may be underdeveloped. This means that a thin soil layer under these conditions, will have a higher cone resistance than measured by the CPT test. This higher cone resistance may then exceed the critical cone resistance, as predicted by the five methods above, which would make the soil layer less likely to liquefy.

Combining the results of the estimated liquefied layers, for each of the CPT probes, gave a three dimensional insight into the mechanism of the liquefaction and lateral spreading at this site. These results are displayed in cross-sections AA and BB, Figures 4 and 5.

3.2 Estimation of displacements

Measurements of the settlement due to the liquefaction and lateral spreading was estimated by photogrammetry to be in the order of 400 mm, which was confirmed by the settlement measured at the northern abutment (300-500 mm) shortly after the earthquake in 1987.

Estimation of the horizontal displacements was not able to be accomplished with any accuracy using photogrammetry. But using a crude technique of adding estimated crack widths, an estimation of the horizontal displacement was made at 1.5 to 2.0 m. Using Hamada's method outlined in Bartlett and Youd (1992), gave a estimate of the horizontal displacement of between 1.2 and 2.0 m for the soils in Figure 5, soils in Figure 4 have both a smaller angle on which to move and movement is some what restrained by the bridge structure.

3.3 Liquefaction model for Landing Road Bridge

Soils that are estimated to have liquefied during the 1987 Edgecumbe earthquake are shown in Sections AA and BB, Figures 4 and 5 respectively.

Section BB liquefied soils have a greater angle of the bottom of the liquefied layer and hence gravity will induce larger displacements than that of Section AA. The bottom of the liquefied layer rises in Section BB close to the river, which would have reduced the overall angle of the bottom of the liquefied layer, and hence reduce the maximum possible horizontal displacement (some residual shear strength is assumed in the liquefied soil). There is still a net downwards

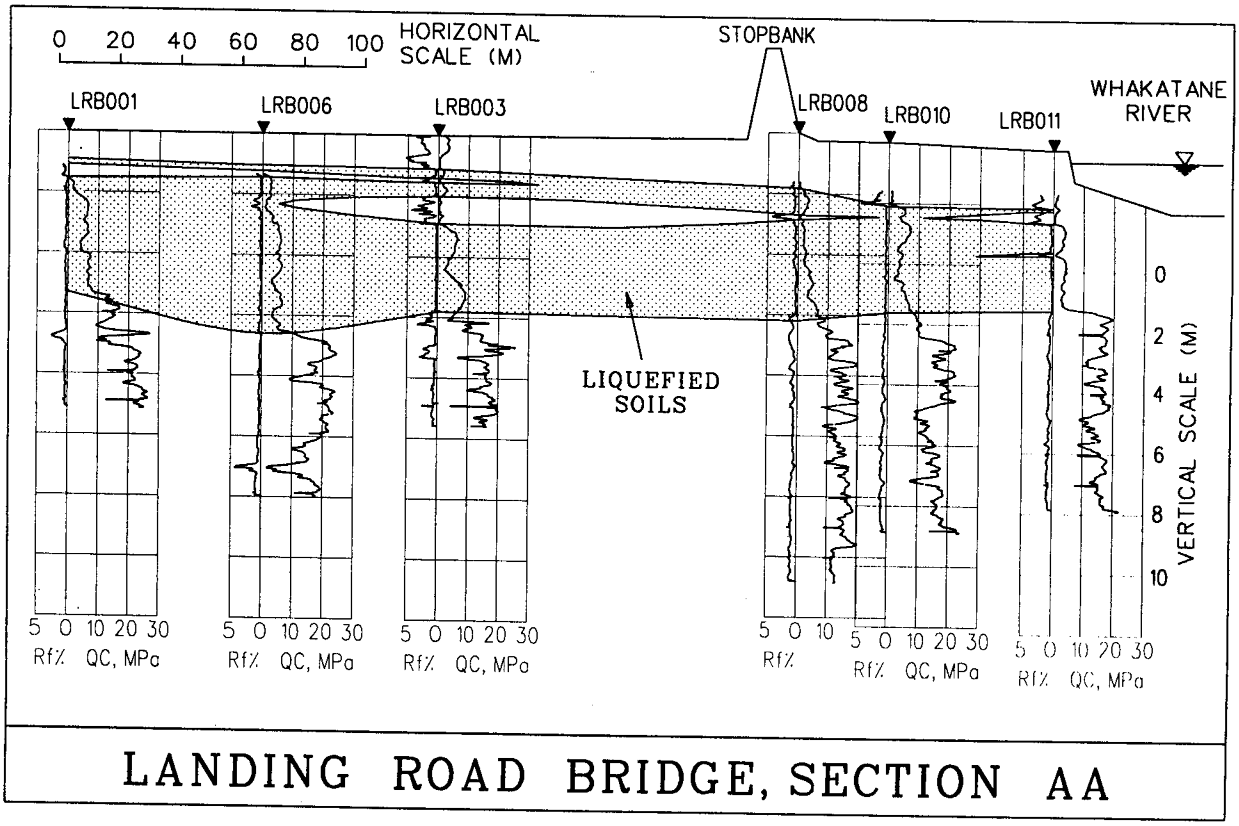

Figure 4: Section AA at Landing Road Bridge site showing CPT data and estimated liquefied soils.

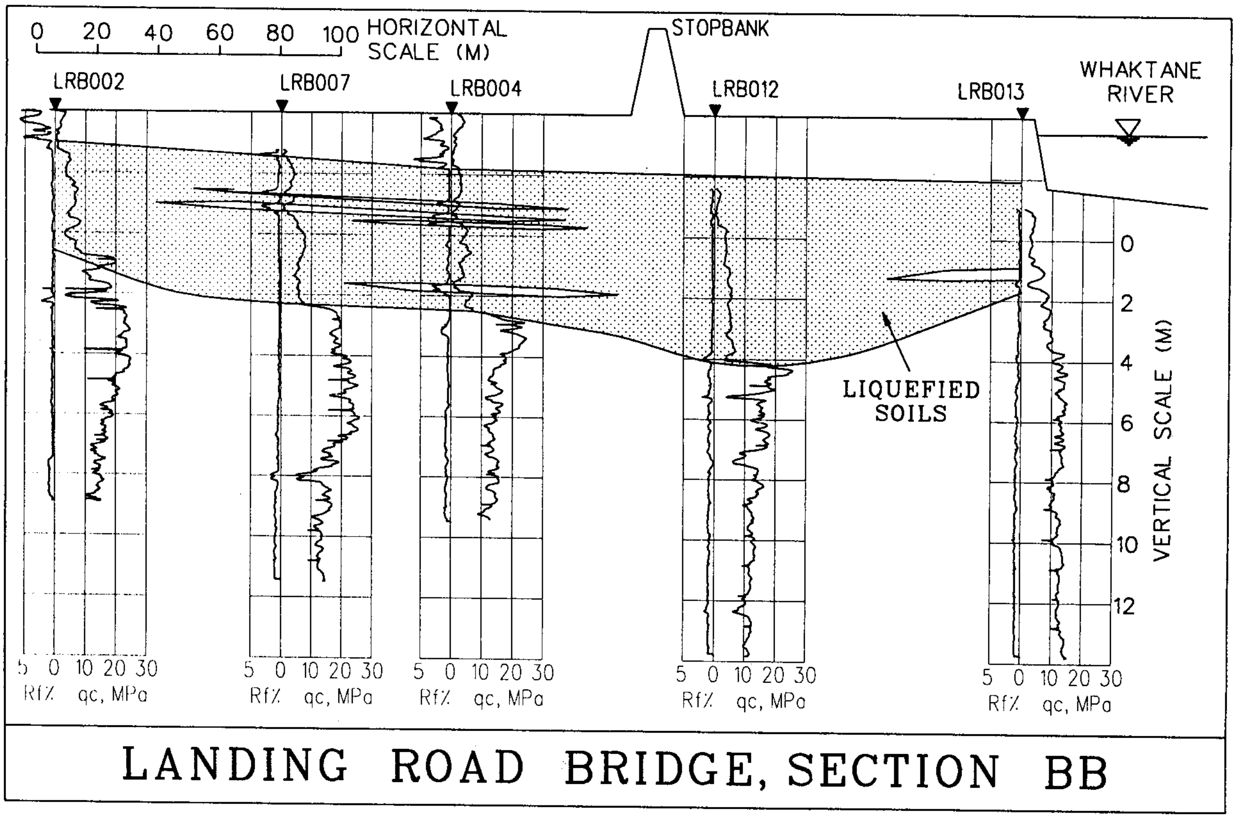

Figure 5: Section BB at Landing Road Bridge site showing CPT data and estimated liquefied soils.

slope towards the river on which the unliquefied overburden was displaced. Although there is a relatively thick liquefied layer(s) (about 5 m), the severity of the ground distortions may have been reduced because of the large grain size (0.3-0.5 mm) which would give rise to faster settling and drainage times than smaller sized particles, this would reduce the time in which lateral spreading could take place.

Section AA provides a more complex picture due to the influence of the bridge structure on the liquefied soils and because of a complex bottom slope of the liquefied layer. Although there is not an obvious slope of the bottom of the liquefied layer towards the Whakatane River, lateral spreading would still be driven towards the river due to the greater total stresses acting on vertical sections beneath solid ground than beneath the river, which would give rise to lateral spreading towards the river.

Structural damage to the northern abutment piles was probably due to the piles being pushed towards the river by the unliquefied soil and fill behind the abutment moving on the liquefied soil. The bottom of the piles being firmly embedded in soil that did not liquefy, forced the rotation of the abutment and cracking in the tops of the piles on the front face. Cracking on only one side of the piles indicates monotonic loading caused by lateral spreading of the piles and not cyclic loading caused by the earthquake.

4 CONCLUSIONS

In general liquefaction occurred to a depth of 6 to 8 m deep and as shallow as 1 m deep, with thickness of the liquefied layer(s) being 4 to 5 m in total. Lateral spreading occurred on gradients of about 1%, and extending to gradients of 2% in places. Soils which liquefied typically had cone resistances, q_c, less than 7 to 8 MPa. Soils below 6 to 8 m, which are denser than the overlying soils, are probably older and cemented to a degree.

More work needs to be put into the refinement of the liquefaction prediction procedures. This would enable the methods themselves to be used more as a standalone technique for the prediction of liquefied layers. These results show that reliance on only one prediction method is unwise.

ACKNOWLEDGEMENTS

I would like to thank The Earthquake and War Damage Commission for providing financial support for this project. Also, Dr. J. Berrill, my supervisor, for his support and help in preparing this report.

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The First ANZ Young Geotechnical Professionals Conference, Mostyn, Khalili & Small (eds), Feb. 1994, Sydney, Australia

SUBDIVISIONS IN THE GREATER AUCKLAND AREA

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ABSTRACT: Residential, industrial and rural subdivisions straddle a diverse range of geologic settings in the greater Auckland area. Typically, Geotechnicial reports address slope stability, bearing capacity and settlement, but depending on local geology, often more unusual features require investigation. These can include piping pumiceous silts, elevated groundwater levels, highly variable volcanic deposits and uncontrolled filling. In addition, topography, vegetation cover and location can often present substantial challenges for fieldwork. Occasionally, despite the best efforts of the investigation team, problems are only revealed once earthworks are underway.

1.1 Geology

Residential, Industrial and Rural subdivisions straddle a diverse range of geologic settings in the greater Auckland area. These include deeply weathered, highly fractured and jointed Mesozoic Age 'hard rock' Greywacke, 'soft rock' Miocene Age Waitemata Group sandstones and siltstones, overlain and interfingered with highly weathered Waitakere Group andesitic/basaltic lava flows, volcanic debris sandstones and breccia conglomerates, together with a series of relatively thin, Pliocene to Holocene Age alluvial, marine and estuarine terrace deposits, incorporating considerable thicknesses of organic silts and peats, as well as areas of both welded and reworked, loose pumiceous silts and sands.

In addition, basaltic volcanism over the last 150,000 years from some 48 volcanic centres has blanketed much of central and southern Auckland with volcanic ash and lapilli, extensive areas of lithic tuff, as well as some 75 km^2 of lava fields.

2.1 Geotechnical Considerations

Geotechnical considerations in subdivisional development around Auckland fall into three main categories, these being:

1. Slope stability (generally on steeper sites, cliff top sections or adjacent to banks and gullies).
2. Adequate bearing capacity and settlement potential for foundation loads.
3. Other, more site specific or localised problems such as piping pumiceous silts or collapsible basalt caverns, etc.

2.2 Slope Stability

Slope stability (or more correctly, instability) features encountered in the region range from the classic 'circular' style failures in highly weathered overburden and alluvial terrace deposits, block/wedge/planar type failures on cliff sites, subtle (and very difficult to investigate) features controlled by tectonic flexural slip clay seams, through to localised failures caused by elevated groundwater pressures in pumiceous silt beds or similar.

In very general terms the slope instability features as related to geology are summarized in table 1.

2.3 Bearing Capacity

Problems with adequate bearing capacity and settlement are generally restricted to terrace deposits, usually where extensive deposits of organic silts and peats are present. Some problems may occur on deeply weathered ash deposits, or where benching operations expose bands of soft weathered silt within Waitemata Group soils, but otherwise these soils and the residual Greywacke soils generally have at least 100 kPa safe bearing capacity.

Pumiceous silts, organic soils and peat present more challenging subdivisional problems and where possible 1.5 to 2 metres cover of naturally occurring clays and silts are left or Engineered filling put in place to ensure that an adequate raft of suitable overlying soil is available. Where this is not possible, lots are tagged for specific foundation investigation and design, the general solutions being either pile foundations, (driven timber or drilled and concreted) or alternatively stiffened raft designs. Driven timber piles ahave also been used beneath concrete strip and pad foundations.

Settlement problems can occur where marked lateral and vertical variations are present in underlying soils, natural soils exhibit marked seasonal moisture change characteristics, or where buildings are constructed partly on natural soils and partly on fill.

Solutions include adequate detailing for building movement (especially with brittle wall claddings), deeper footings or piled foundations.

2.4 Other Geotechnical Problems

Several problems additional to the above encountered on recent subdivisional investigations have included the following:

1. Highly erodible piping silts, both in pumiceous soils and weathered Waitemata Group soils (major design implications for underfill and buttress drain construction, generally controlled with geotextiles).

2. Extremely soft, saturated infill gully deposits (colluvium and slump debris). (Typical problems have included thoroughly bogged drilling rigs and excavators, as well as general earthworks difficulties such as installation of underfill drainage and trafficability).

3. Discovery of 5 skeletons in a lava cave on a 14 unit development site (potential for major contractual delays; averted through consultation with local Maori, leaving them undisturbed and sealing off mouth of cave with several metres of concrete, reinforcing steel and rock).

4. Extremely soft, marine silt deposits on a coastal subdivisional development (too soft to register any friction on cone penetrometer (CPT) cone; major design implications for bearing capacity and settlement - estimated 550mm over 8 metres thickness for 55 kPa surcharge).

5. Loose to very loose, uncompacted refuse fill on proposed industrial subdivision (seismic velocities <300 m/sec; potential for up to 2 metres of settlement; landfill gas problems, difficult piling conditions due to hardfill and inorganic refuse as well as elevated sulphate levels and generally aggressive soil conditions).

6. Environmentally 'impaired' sites including sites with imported, dirty fill through to development on former industrial sites such as timber treatment yards, etc.

7. Steep, hilly sites requiring development of extensive bulldozer tracks prior to investigation with bulldozer winches used to pull rigs into place or at worst use of heliportable rigs.

3.1 Investigation Methods

General investigation methods used during subdivision investigations can include any or all of the following:

1. Extensive field mapping and walkover surveys, coupled with aerial photographic interpretation; very useful as a planning tool for investigations, as well as slope stability assessments.

2. Hand auger boreholes; 50mm diameter to depths of up to 6 metres, in conjunction with field shear vanes and scala penetrometer tests.

3. Machine boreholes; both conventional open barrel and triple tube drilling, as well as HQ and NQ wireline have been used to depths of up to 30 metres, together with SPT testing, recovery of 100mm and 65mm diameter tube samples and bulk core samples for effective stress, consolidation and compaction testing. Hollow stem, 150mm and 300mm flight augers have also been used in conjunction with SPT and field vane testing.

4. Excavator pits: 12 to 20 tonne machines with reaches of up to 7 metres. Very useful tool for assessing infill gully deposits and general soil characteristics for earthworks.

5. CPT's in soft alluvial ground and marine terraces; useful stratigraphic mapping tool, relatively cheap costs for wide coverage.

6. Percussion boreholes, especially on basalt and scoriaceous sites for soakage and groundwater investigations.

7. Seismic methods, including refraction, gravity, resistivity and ground penetrating radar. Often used in conjunction with environmental investigations.

4.1 Case Studies

Two contrasting sites will be described in the following section, these being:

1. An 83 lot subdivision at Murrays Bay, on the North Shore, currently under construction on steep, dissected Waitemata Group sandstones and siltstones.

2. The Tamaki Farms Estate, involving both industrial and residential development at East Tamaki, on a mixture of volcanic, marine terrace and Waitemata Group soils capped in places with basalt rock.

4.2 Hauraki Heights Subdivision, Murrays Bay

The Hauraki Heights Subdivision is currently under construction (as at December 1993). Eighty three lots are planned, which, together with roading, accessways and reserves, will cover an area of approximately 12.7 hectares. The site has been the subject of several geotechnical investigations and proposed subdivisional schemes, the current development proceeding only after an extensive series of Town Planning Hearings.

The terrain comprises hill and valley topography, covered with regenerating bush, scrub and pine trees. One main ridge runs north-west/south-east between the top of Bellbird Rise and the Pinehill Reservoir, with a series of sub-parallel flanking ridges lying between the main ridge and East Coast Road to the west (fig. 1).

Total earthworks volumes are in the order of 70,000m^3, with cuts and fills of up to 10 and 15 metres respectively aimed at levelling off the ridges and filling in the flanking gullies.

The site was investigated by ourselves in December 1992 as a follow-up to earlier work undertaken in 1982, 1985 and work completed in the mid-1970's by another Consultancy.

The latest investigation involved a detailed walkover field survey and review of aerial photographs, coupled with a relatively intensive machine and hand auger borehole programme as well as laboratory testing and extensive slope stability analyses.

Briefly, problems identified were extensive slope movement and soil creep on the slopes flanking the main gullies, coupled with areas of elevated groundwater and a distinct range of lithology. The heavily vegetated site made field operations quite difficult. Apart from installing bulldozer tracks to allow machine rig access, several hand auger borehole sites needed to be cleared with a chainsaw prior to drilling.

In summary, the main ridges were considered to be 'intact ground' with the dominant slope features being relatively shallow movements down into the flanking gullies. Deep seated instability was not ruled out, but no conclusive field evidence was seen.

Following site clearance however, it became apparent that the front of the central ridge area consisted of an old, relatively thick (5 to 10 metre) landslide mass sliding over bedrock. It appeared that this slide had travelled somewhat laterally across the front of the ridge, starting from an area to the north of the tempoary silt detention dam, and moving south-west with most of the slide debris underlying an adjacent council reserve.

The slide is currently being sucessfully buttressed through infilling the detention dam gully, coupled with the installation of extensive, geotextile wrapped AP7 scoria buttress and underfill drains dug down and keyed into the underlying bedrock. All slump debris was excavated from the gully and replaced with engineered filling to ensure that the gully fill was benched onto competent sandstone.

In addition, some 750 metres of geotextile wrapped, 50mm diameter PVC screen is currently being installed in 6 horizontal drainage lines to maintain long term control over groundwater levels across the lower half of the site. Current water flows have ranged from an initial (and impressive) 6,000 litres per hour, on a 130 metre long line, stabilising to a constant 2,600 litres per hour over a week to more typical flows of between 100 and 200 litres per hour. Total flows from the 6 lines are currently running at 3,600 litres/hour, or around 86,000 litres/day.

Due to the fact that the lowest point of the landslide is situated in the adjacent reserve amongst well developed native bush at least 100 years old, the North Shore City Council requested that equipment and material were flown in and out using a helicopter, with full removal of drill cuttings and all drilling operations planned to have minimal impact on the environment.

Some 20 loads were flown in, including diesel, drilling rods, casing, slotted screen, power packs, drilling rig and pumps, as well as 30 bales of hay to provide silt control.

Piezometers installed prior to the horizontal works have shown marked changes as the lines are drilled, indicating that the drains are performing as intended.

The bulk earthworks for the site are scheduled for completion in February 1994.

The main point to note on this subdivision is that an ancient landslide moved sideways relative to present day topography, with subsequent downcutting through the slump debris by streams and extensive vegetation cover completely masking its presence. However, close site supervision detected the problem as soon as it became apparent and the works described have ensured that the development will be sucessful.

3.4 Tamaki Farms Estate, East Tamaki

The Tamaki Farms Estate, adjacent to the Tamaki Estuary in East Auckland, has been developed in several stages since 1986. Currently Stage 4 residential and Stage 2B Industrial are under construction, with earthworks due for completion early in 1994.

Briefly, the site consists of an old marine terrace, partly cut into underlying Waitemata Group soils, with relatively extensive deposits of overlying, loose, reworked pumiceous silt, several areas of organic silts and minor peat deposits, as well as typical inorganic, moderately plastic clayey silts and silty clays. Some of the site is covered by up to a metre of airfall volcanic ash, with the central third of the site capped with a basalt lava field up to 10 metres thick.

Investigations have involved both hand auger and conventional machine boreholes, excavator pits and field mapping, as well as percussion rock drilling to define areas of basalt (initially for a proposed quarry, but later used for location of service lines).

Earthworks operations have generally been confined to levelling off the site, mucking outsoft pockets of soil in and between lava flows and backfilling with compacted spoil, as well as placing up to a metre of filling over the basalt to allow conventional residential construction.

Apart from the usual problems of bearing capacity and differential settlement on the more organic soils (generally fixed using over-length driven timber poles and control joints on brick veneer walls), more unusual problems have been localised instability on esplanade reserve slopes caused by elevated groundwater levels in confined pumiceous silt layers (generally concentrated in old, buried stream channels), the extreme erodability of these materials under concentrated water flows, often complete loss of trafficability in these soils on haul roads (a sort of liquefaction failure effect), difficulties in achieving adequate compaction with some of the volcanic ash soils, and in monitoring and certifying fill areas using excavated basalt rock.

5.0 Summary

In conclusion, we have found that a detailed knowledge of local geological conditions and their problems has been useful in assessing subdivisional sites, especially where budgetary and time constraints dictate a relatively modest investigation. However, there is always the potential for unusual, or unexpected problems to appear once site works commence.

Acknowledgments.

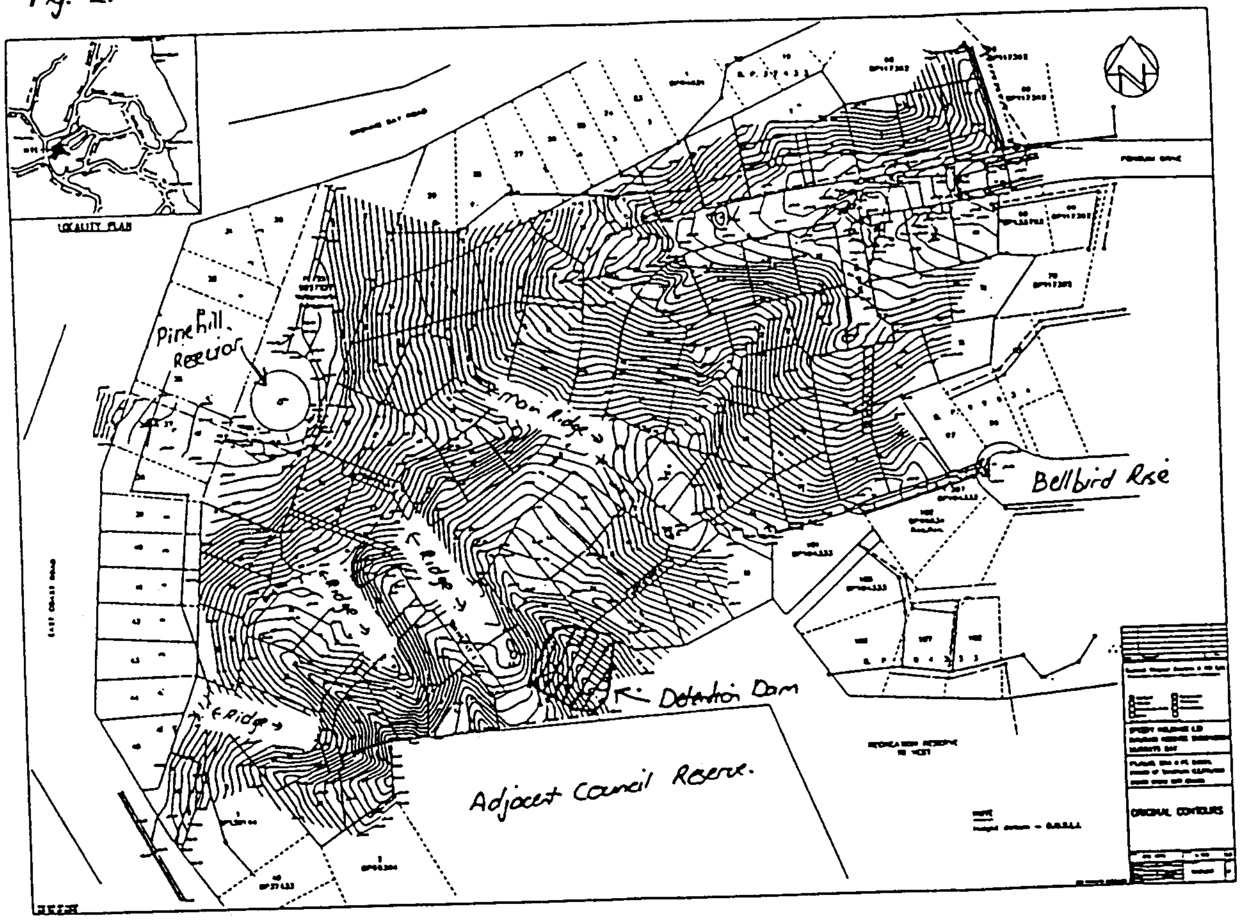
The writer would like to thank Speedy Holdings Limited and Neil Construction Limited for permission to publish details on Hauraki Heights and Tamaki Farms Estate Subdivisions respectively. Also, grateful thanks to Elaine Appleton for typing this paper.

Table 1.

Geology	Typical Instability Features	Site Settings
Indurated 'hard rock' Greywacke. Weathered soils typically firm to stiff, non-plastic, red/orange brown clayey silts and sandy silts. Unweathered rock typically dense, dark-grey sandstone alternating with grey to black argillite. Hard, typically closely to moderately closely fractured.	Circular and planar failures - slumps, slides, flows	Developed in highly to completely weathered overburden. Planar failures common where 'greasybacks' (soft overburden in conjunction with elevated groundwater) occur over less weathered rock.
	Wedge/block failures, rock falls, rock slides	Common in unweathered to moderately weathered rock on steep country. Failures usually controlled by joints, shear zones and fractures. Majority of gullies and valleys have developed due to some form of underlying structural control.
	Shallow soil creep	Common on steep slopes where relatively thin 'skin' of weathered overburden has developed over underlying bedrock.
'Soft rock' Waitemata Group sandstones and siltstones. Weathered residual soils range from soft to stiff, non to very plastic, grey/yellow/brown/cream silts, clayey silts, sandy silts and silty clays, reflecting their depositional lithology. Often extensive soil softening around standing groundwater levels. Unweathered rock is typically alternating, weak to moderately strong sandstone, siltstone and mudstone.	Circular and planar failures, soil creep	Circular failures usually restricted to completely weathered overburden, generally on slopes above about 20 degrees. Often occur in conjunction with elevated, sometimes artesian groundwater conditions. Planar failures somewhat less common, tending to develop along bedding planes coupled with thin clay seams and elevated groundwater or where 'weathering front' of overburden parallels slope and is exposed as a result of erosion. Both 'down dip' and 'cross dip' planar failures have been encountered on recent subdivisions. Often difficult to recognise in field.
	Clayseam controlled block slides	Relatively common within the more muddy or silty faces of the Waitemata Group, especially in East Tamaki/Flat Bush areas.
	Wedge/block failures - rock falls, rock slides, toppling failures	Most obvious along the extensive cliffs fronting the Waitemata and Manukau Harbours, but often present in inland, deeply incised gullies (usually well obscured). More subtle manifestations include some 'stress relaxation' features where blocks and beds of more resistant rock dilate laterally as a result of downcutting streams in adjacent gullies (often found with bedding parallel clay seams and clay filled joints. Toppling failures common where bedding dips inward on cliff sites, often in conjunction with wedging tree roots or similar. Block falls often develop where siltstone beds fret and spall faster than overlying sandstone.
Alluvial/estuarine terrace deposits. Diverse range of soft to stiff, non to very plastic, normal to well over-consolidated, inorganic and organic silts, clayey silts, sandy silts, silty clays and clays often interbedded with pumiceous silts, organic peat and gravels. Commonly overlain with volcanic ash deposits.	Circular and planar failures, soil flows, piping failures.	Circular failures common where areas are subject to active erosion such as in stream gullies or along estuary banks. Planar features rare, except where soil moves 'en masse' over underlying bedrock. Soil flows occur under extremely wet (usually concentrated runoff) conditions. Piping failure common in pumiceous deposits.

Volcanic materials - tuff rings, scoria cones, lava flows. Deposits typically exhibit the following: - rapid lateral and vertical variation. - emplaced over wide range of underlying sediments. - lava flows occupy former river valleys, locally cavernous.	Circular failures	Relatively common in highly weathered ash, or where volcanic detritus has been reworked into terrace deposits
	Planar failures	Occur in situations where ash or tuff is deposited onto sloping, weathered Waitemata group soils. Often potential failures not recognised on immediate site as problems usually well buried and can extend over wide areas.
	Block failures	Restricted to frontal lobes of basalt flows, or in quarry areas.

Fig. 1.



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SEISMIC HAZARD EVALUATION OF AN ACTIVE FAULT.

T.J. McMORRAN

D.J. DOUGLAS & PARTNERS PTY LTD

ABSTRACT

Seismic hazard evaluation for the Hope Fault in New Zealand has been carried out based on detailed mapping and reviews of historical earthquake data in several recent studies. Methods used in evaluating seismic hazard and examples from recent studies are presented here.

1. INTRODUCTION

In areas of active faulting the seismic hazard evaluation of faults or fault segments is important. This evaluation involves estimation of earthquake magnitudes and the determination of anticipated displacements associated with the larger events. Such evaluation requires data from various approaches such as analysis of historical events, structural mapping of faults, slip rate determinations, displacement per event determination and dating of previous earthquakes.

This paper outlines some of the approaches that have been used by the author and other workers to collect such data from the Hope Fault in New Zealand.

2. STRUCTURAL ASPECTS OF THE HOPE FAULT

New Zealand lies across the boundary between the Pacific and Australian Plates. Oceanic crust of the Pacific Plate is being subducted westwards beneath the North Island and northern South Island, along the Hikurangi Trench. The velocity of relative plate convergence at the southern end of the Hikurangi Trench has been estimated at 54 ± 9 mma^{-1}, (Figure 1). In the northern South Island, the obliquely convergent relative plate motion is accomodated by the Marborough Fault System which comprises four major active faults, the most southern being the Hope Fault.

The Hope Fault extends 230 km from the Alpine Fault to the Kaikoura coast (Figure 1). A single major fault zone trace is visible for the central 150 km only, with each end consisting of many diverging splays where dextral displacement is dissipated as oblique slip and thrusting movements (Freund, 1971; Van Dissen, 1989).

For most of its length, the Hope Fault cuts Mesozoic Torlesse Supergroup basement rocks, consisting mainly of indurated greywacke and argillite with the active fault plane striking between 070° and 085° and usually dipping steeply to the NNW, (Freund, 1971; McMorran, 1991; Van Dissen 1991). The fault zone consists of a corridor of intensely crushed and sheared basement rocks up to 2 km wide (Freund, 1971; McMorran, 1991). Faults that have been active in the Late Quaternary are generally readily mapped as they displace Late Quaternary glacial and fluvial gravels.

The Hope Fault can be divided into discrete segments on the basis of fault continuity, with segment boundaries being indicated by releasing stepovers, substantial bends in the fault, or splaying of subsidiary faults (see Figure 1). The Hope River segment, which ruptured causing the 1888 Glynn Wye earthquake (Cowan, 1990) is the best example and is probably the best defined segment.

Perhaps the most striking structural feature of the Hope Fault is Hamner Basin, a 15 km wide releasing bend or stepover that separates the Hope River segment from the Conway segment. Such releasing structures tend to reduce the earthquake intensity by limiting the length of fault that can rupture in a single event.

The structural geometry of the Hope Fault varies along its length, from nearly pure strike-slip or transtensional movement (Cowan, 1989) in the central section of the fault, to transpression east of Hamner Basin (McMorran, 1991), and a progressively stronger oblique component towards the coast (Van Dissen, 1991). This change in structural style reflects the varying orientation of the fault plane to the relative plate motion vector, and the proximity to the Alpine Fault in the west and the subduction zone in the east. Variations in the estimated late Quaternary slip rate for the Hope Fault along its length reflect the change in faulting style.

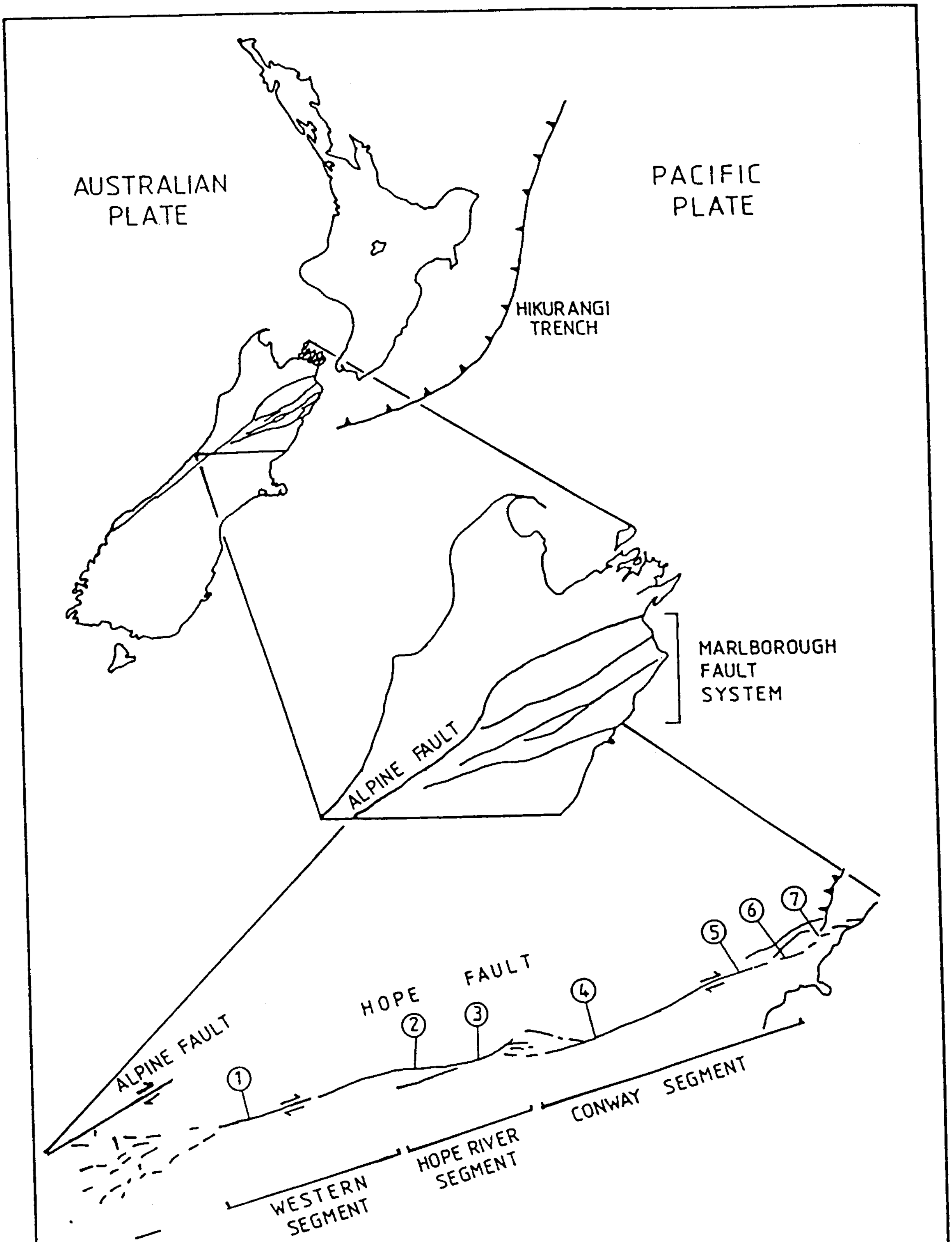

FIGURE 1. SCHEMATIC MAP OF THE HOPE FAULT SHOWING POSSIBLE SEGMENTATION. LOCALITY NUMBERS 1-7 REFER TO TABLE 1 FROM GREGG (1964); FREUND (1971); COWAN (1990); VAN DISSEN (1991)

Offset of geological and geomorphological markers across the Hope Fault varies from several metres for single event displacements to a total offset of 20 ± 2 km for displaced stratigraphic markers in the basement rocks (Freund, 1971; McMorran 1991). Displacement of geomorphological features such as river terraces and stream channels present an opportunity for estimating the Late Quaternary slip rate of faults, (Seih and Jahns, 1984; Berryman 1990). A summary of slip rate data for the Hope Fault is presented in Table 1, with methods described in Section 3.1.

Although the errors are often large, due to poor matching of degraded features or imprecise age determination, some significant trends are evident. The Late Quaternary dextral slip rate varies over relatively short distances along the Hope Fault. This variation is due to the effect of changes in fault geometry along its length as discussed above. Cowan, (1989, 1991), discusses the variation of displacement in a single earthquake event (Glynn Wye 1888) and demonstrates how these displacements relate to bends or stepovers in the fault trace. The displacement during that earthquake dropped by approximately 40% across a releasing feature (Lake Glynn Wye Graben) over a distance of only 2 km.

The ratio of horizontal to vertical displacement also varies along the fault. This ratio is also directly related to fault geometry, with a high ratio being indicative of a relatively oblique fault motion. The variations shown in Table 2 are consistent with the structural model of an increasing oblique motion component towards the east coast. The variations in obliquity and rate of displacement along the Hope Fault show that generalisations about fault behaviour, based upon limited data, should be carefully evaluated.

3. EVALUATING SEISMICITY OF THE HOPE FAULT

Evaluation of seismic hazard for a fault or fault segment often involves assessment of the length of fault segment that can rupture to generate an earthquake. This length can be related empirically or mathematically to probable earthquake magnitude (see Section 3.5). The assessment of Late Quaternary slip rate, displacement per event, and time elapsed since the last event on a fault all relate to the seismicity model. Examples from recent studies of the Hope Fault are used in the following sections to summarise the seismic hazard of the Hope Fault.

3.1 1888 Glynn Wye Earthquake

The only earthquake associated with surface rupture of the Hope Fault since European settlement is the 1888 Glynn Wye earthquake. Recorded in some detail by Alexander McKay (1890), this earthquake constituted one of the first detailed descriptions of a surface rupture earthquake and the first report of a strike slip earthquake (Sylvester, 1988).

Recent studies of the historical data and interpretation of the resulting geomorphological expression, suggest that the 1888 event caused a 30 ± 5 km long surface rupture of the Hope River segments (Cowan, 1989, 1991). During the earthquake (M_S7.0 - 7.3) local felt intensities of MMIX were experienced along the ruptured fault. Shaking attennuated rapidly perpendicular to the fault suggesting a shallow focus for the event.

TABLE 1 - SLIP RATES CALCULATED FOR THE HOPE FAULT.

Locality	Source	Slip Rate (mma^{-1})	h/v Ratio	Remarks
1. Harper Pass	Hardy & Wellman (1984)	14	-	28000 yr BP riser? displaced 200 m
2. Glynn Wye	Cowan (1990)	14 ± 3	-	17000 ± 2000 yr BP Moraine displaced 230 ± 20 m
3. Manuka Ck	Cowan (1989)	10.6 ± 0.3	15-17	3482 ± 77 yr BP riser displaced 36 ± 0.5 m
4. Hossack Stn	McMorran (1991)	11 - 26	30-33	2330-3690 yr BP stream channel displaced 40 - 60 m
5. Sawyers Ck	Van Dissen (1989, 1991)	20 - 35	14	2780 ± 560 yr BP riser displaced 78 ± 2 m
6. Goldmine Ck	Van Dissen (1989)	12 - 20		14375 ± 2000 yr BP fan displaced 230 ± 20 m
7. Hapuku R.	Van Dissen (1989)	2.1-7.5	1.5	6290 ± 950 yr BP boulder bar displaced 12.6 m and 8.5 ± 0.3 m

Note that variations in both slip rate and horizontal to vertical displacement ratio occur along the fault. These variations relate to the change in faulting style both locally and regionally. Locality numbers refer to Figure 1.

Severe shaking was also experienced on Hamner Plain during the earthquake, and for more than one month aftershocks were experienced, possibly relating to extension within the basin allowing stress concentration to be re-equilibrated .

Several offsets measured by displaced fencelines at the time of the earthquake showed dextral displacements of up to 2.6 m. Variations in offset were related by Cowan (1989, 1991) to changes in fault geometry as discussed in the preceding sections of this paper.

Recent evidence suggests that the 1888 Glynn Wye earthquake may have been a characteristic event for the Hope River segment and that similar events may occur every 80 - 200 years (Cowan and McGlone, 1991).

3.2 Late Quaternary Slip Rate

The displacement of Quaternary features such as stream channels, river terraces, fans etc., by active faults is a common way to assess slip rate. At Hossack Station, the Conway segment has offset a small stream and created a series of abandoned channels presenting an opportunity for estimation of slip rate. Trenching within two of the abandoned channels revealed a stratigraphy that could be simply divided into coarse grained channel deposits and fine grained flood overbank deposits, the latter presumably accumulating after abandonment of the channel (McMorran, 1991). Radiocarbon dating placed limits on the age of channel abandonment, and slip rates determined at this location are in the range 11 - 26 mma^{-1} for the late Holocene. Large uncertainties associated with timing of channel abandonment are reflected in this range.

A series of post glacial degradational river terraces at Hossack station show a dextral offset of at least 260 m (McMorran, 1991). Downcutting associated with the end of the last glaciation began approximately 13000 years BP suggesting a minimum late Quaternary slip rate of 20 mm per year.

A slip rate at Glynn Wye Station of 14 ± 3 mm per year was determined using offset of a terminal moraine of approximately known age (Cowan, 1990) by glacial chronology. Dating of Late Quaternary geomorphological features has also been carried out by weathering rind methods (Van Dissen, 1989). This technique involves measuring the thickness of weathering rinds on Torlesse sandstone cobbles which weather at a known rate (McSaverney, 1992). Dating of geomorphic features has been carried out with an accuracy of better than ± 10%. Another technique that has been used for the dating of geomorphic surfaces is lichenometry. Yellow rhyzocarpon lichen grow at a known rate such that diameter of lichen can be related to the age of Torlesse sandstone boulders on which it grows (Bull, pers. comm, 1991).

3.3 Displacement Per Event

The estimation of how much displacement will occur on a fault when it next ruptures can be important to the design of engineering projects that are located near to or across that fault. Return period for earthquakes occurring on a fault is often calculated from slip rate and average displacement per event. Also, the magnitude of earthquakes has been empirically related to maximum fault displacement associated with those earthquakes (Bonilla, et al., 1984). Estimating single event displacement can be done by historical review [as was the case with the 1888 Glynn Wye earthquake for the Hope River segment (Cowan, 1991)]. Other techniques include measuring offsets associated with single prehistoric earthquakes (Berryman, 1990; Seih and Jahns, 1984) or estimating offsets based upon rupture of other known faults (Van Dissen, 1991). For the Conway segment, no reliable single event displacements have been recorded. Some offsets of approximately 10 m of dextral displacement have been noted (Pettinga, pers.comm.) and these probably relate to two displacements of about 5 m each. Further mapping is required to evaluate the likely displacement per event for the Conway segment.

3.4 Dating of Previous Earthquakes

Age determination of previous fault rupture events on active faults enables an indication of the timing of the next earthquake generated by that fault. The dating of geomorphological features such as slope failures, depositional events of fault gouge material and tree damage (Cooper & Norris, 1991) can be used to date previous earthquakes. Cowan and McGlone (1991) dated five earthquakes for the Hope River Segment based on silt layers within a peat swamp. It was postulated that silt horizons were associated with abnormally high sediment supply caused by landsliding in the swamp catchment areas following earthquakes. The techniques of weathering rind dating (McSaverney, 1992) and lichenometry discussed earlier have also been used to date paleoseismic events by determining multi modal ages for boulders on or near fault scarps. The last earthquake on the Conway Segment may have been approximately 1838, which coincides with a modal peak for yellow rhizocarpon lichen (Bull et al., 1991). Many potentially datable landslides occur along the Conway Segment including several at Hossack Station. Dating of these features may help to improve the paleoseismicity record for the Hope Fault.

TABLE 2 - ESTIMATED EARTHQUAKE MAGNITUDES

	Length (km)	Surface Wave Magnitude (M_S)	Average Displacement (m)	Moment Magnitude (M_W)
Western Segment	50 ± 5	7.3	3 6	7.0 7.2
Hope River Segment	30 ± 5	7.0 - 7.2	2	6.8-6.9
Conway Segment	70 ± 5	7.4	3 6	7.2 7.4

NOTE: Expected earthquake surface wave magnitudes calculated using empirical formulae from Bonilla et al (1984). Moment magnitudes calculated using formulae from Hanks and Kanamori (1979), with estimated average displacements for western Hope Fault and Conway segments. For discussion and return periods refer Section 3.5.

3.5 Probable Earthquake Magnitude

Studies of active faults have shown that earthquakes can occur at regular intervals (return periods) with similar magnitudes. The Hope Fault may rupture in this way as studies of the Hope River Segment suggest (see Section 3).

For active faults, expected earthquake magnitudes and exceedence probabilities can be empirically determined from statistical analysis of historical earthquakes. Earthquake magnitude can be related to rupture length or maximum fault displacement (Bonilla et al., 1984; Slemmons, 1982). From the fault segment lengths shown in Figure 1, earthquake magnitudes of M_S 7.0 to M_S 7.4 are derived from the relationship of Bonilla et al., (1984), see Table 2. These figures have a 50% exceedence probability, for smaller exceedence probabilities, the distribution of historical earthquake events has a standard deviation of 0.3 M_S units.

Hanks and Kanamori (1979) relate fault rupture area to a scale of moment magnitude (M_W), where moment magnitude approximately equals surface wave magnitude (M_S) for rupture events between M_S 5.0 and M_S 7.5. For the Hope Fault moment magnitudes of between M_W 6.8 and 7.4 are predicted from the above method.

Return periods for earthquake events on the Hope Fault are determined from slip rates and known or estimated single event displacements (Van Dissen 1991; McMorran, 1991), or from indirect dating of prehistoric earthquakes, (Cowan, 1989, 1991; Cowan and McGlone 1991). No single event displacements are documented for the Conway segment and both Van Dissen (1991) and McMorran (1991) calculated return periods of between 90 and 600 years based upon estimated displacement during an earthquake. Single event displacements for the Hope River segment are based upon the historical observations of the 1888 Glynn Wye earthquake (McKay, 1890; Cowan, 1991). The return period calculated by Cowan and McGlone (1991) for the Hope River segments is 81 to 200 years. Few data for the western Hope Fault are recorded and return periods based upon estimated fault displacement of 3 m to 6 m are between 210 and 430 years for earthquakes of magnitude M_S 7.0 to 7.4.

4. CONCLUSIONS

From the above studies many aspects of seismic hazard evaluation, for areas where active faults are present, are discussed.

As a major active dextral strike slip fault, the Hope Fault must be considered a major contributor to the seismic hazard in the northern South Island. The structural model for the Hope Fault includes the subdivision into segments based on geometric and kinematic changes in the structural style of the fault along its length. Variations in both slip rate and obliquity of relative fault motion along the fault relate to proximity to the Alpine Fault and subduction zone to the west and east respectively. Fault segments are likely to rupture independently, and may do so with 'characteristic' earthquakes of relatively uniform magnitude and return period. Rupture of the three major segments are likely to generate earthquakes of M_S 7.0 - 7.4 with recurrence intervals of between 80 and 600 years. Earthquakes of that magnitude may cause local felt intensities of up to MMX along the ruptured fault. It is important to emphasise that evaluation of seismic hazard presented by an active fault must be carried out based on a sound knowledge of the structure and movement history.

Geotechnical classification of ignimbrite and prediction of engineering behaviour from simple index tests

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ABSTRACT: Ignimbrites are typically weak rocks of low density and high porosity, and undergo considerable softening on saturation. Extensive systems of open, continuous, vertical joints occur in many ignimbrites, while others are effectively non-jointed. Large changes in strength and jointing may occur within a single profile. Two broad categories of ignimbrite are recognised: durable ignimbrites with $I_{d2} \geq 90$ %, and non-durable ignimbrites with $I_{d2} \leq 30$ %. Durable ignimbrites behave as weak rocks for which the rock mass characteristics exert the primary influence upon slope stability and engineering behaviour. Non-durable ignimbrites are typically non-jointed and are the weakest materials; they are classified as soft rocks, and the intact rock characteristics are the dominant control on their behaviour. Second-cycle slake durability index and effective porosity together allow classification of ignimbrites and prediction of likely material characteristics.

1 INTRODUCTION

Ignimbritic materials are widely distributed throughout the Central North Island of New Zealand as a result of Quaternary eruptions from the Taupo Volcanic Zone, together with eruptions from older volcanic centres in the Coromandel region. These materials are increasingly being encountered in engineering investigations, sometimes less than successfully (for example, Anon., 1982, 1983).

Many site-specific investigations of the engineering behaviour of ignimbrites have been undertaken (for example, Maloy and Lowe, 1945; Yamanouchi *et al.*, 1981; Price, 1983; Price *et al.*, 1985; Nappi and Ottaviani, 1986). However, site-specific studies measure only those properties relevant to the particular development and hence data from different studies are not necessarily complementary.

In this paper, the results of a systematic series of measurements of a variety of geotechnical properties of ignimbrite materials are used to develop a simple classification system for ignimbrites which gives a ready indication of the likely engineering behaviour of the materials. Index tests which allow classification of ignimbrites, and hence prediction of their characteristics, are identified.

2 DEFINITION OF IGNIMBRITE

Ignimbrites are well defined from a geological point of view as materials deposited by a pyroclastic flow. This genetic definition provides useful geological information about the origin and likely composition of the material. From an engineering viewpoint however, this definition covers an enormous range of materials which display quite different geomechanical properties.

At one extreme are the hard, jointed materials which form steep bluffs, and which have successfully been used as the foundation rocks for a number of hydro-dam projects, such as the Whakamaru, Maraetai, and Waipapa dams. The definition of ignimbrite also incorporates somewhat softer material which may show excellent columnar jointing, or very soft material that is characterised by few joints, can be dug with a spade and might generally be referred to as "non-welded" or "poorly welded". At the furthest extreme are deposits consisting of essentially individual clasts, such as the Taupo Ignimbrite.

3 GEOMECHANICS

3.1 Intact rock properties

Table 1 presents geomechanical data obtained from a variety of unweathered ignimbrite specimens from the Central North Island, New Zealand. Geomechanical tests were undertaken following the standard procedures recommended by Brown (1981). Where appropriate, measurements were made in both saturated and dry conditions to give the range of likely geomechanical properties over all moisture contents.

Table 1 includes the maximum and minimum measured values for bulk density, porosity, a variety of strength parameters, and the second-cycle slake durability index (the percentage of the original mass remaining after two standard cycles of wetting and drying and mechanical abrasion). Softening values for strength measurements are also given: these are the ratios of dry to saturated strengths.

The following points summarise the geomechanical behaviour of ignimbrite:

(1) Ignimbrites have low bulk densities and high porosities, with a significant proportion of the total porosity accounted for by pores which do not contribute to the effective porosity.

(2) Some ignimbrites are essentially unaffected by the slake durability test, whereas others break down almost completely.

(3) Ignimbrites are weak in both compression and tension; their cohesion is weak, but the angle of internal friction is relatively high and uniform. Saturation leads to considerable softening of the

Table 1: Range of a variety of geomechanical properties of intact ignimbrite specimens (from Moon, 1993a).

	minimum	maximum
bulk density		
saturated, ρ_{sat} (kg m^{-3})	1644 ± 10	2290 ± 2
oven-dry, ρ_{dry} (kg m^{-3})	1212 ± 10	2124 ± 2
porosity		
effective, η_{eff} (%)	14 ± 1	43 ± 1
true, η_{true} (%)	17 ± 1	51 ± 2
compressive strength		
saturated, $\sigma_{c,sat}$ (MN m^{-2})	0.23 ± 0.01	36 ± 5
oven-dry, $\sigma_{c,dry}$ (MN m^{-2})	0.73 ± 0.07	54 ± 4
softening factor, $\sigma_{c,soft}$	1.3	5.9
tensile strength*		
saturated, $\sigma_{t,sat}$ (MN m^{-2})	0.12 ± 0.03	7 ± 2
oven-dry, $\sigma_{t,dry}$ (MN m^{-2})	1.3 ± 0.4	7 1 ± 0.7
softening factor, $\sigma_{t,soft}$	1.0	10.8
cohesion		
effective, c' (MN m^{-2})	0.06 ± 0.01	9.0 ± 0.5
total, c (MN m^{-2})	0.14 ± 0.06	13 ± 2
softening factor, c/c'	1.2	8.4
angle of internal friction		
effective, ϕ' (°)	31 ± 4	38 ± 4
total, ϕ (°)	27 ± 5	35 ± 2
slake durability index		
second-cycle, I_{d2} (%)	30	99

* Note that reliable tensile strengths could not be obtained for the weakest specimens.

materials, with softening factors of 3 to 5 being common.

(4) Although weak, the strength of ignimbrites is highly variable. Indeed, the materials are characterised by very rapid vertical variations in strength, which can lead to two orders of magnitude change in the measured strengths within one vertical section.

(5) Considerable deformation prior to reaching the peak strength is typical. Fracture surfaces show evidence of crack initiation at inhomogeneities in the rock (crystals and pumice clasts), and propagation through the groundmass between these inhomogeneities.

3.2 Jointing

The primary jointing features in ignimbrites are tension fractures formed during cooling and compaction of the mass. However, classical columnar jointing, such as seen in basalts and other lavas, is surprisingly rare in New Zealand ignimbrites.

Typical columnar jointing in ignimbrites is comprised of very wide, somewhat curved, irregular columns. The margins of these columns are generally marked by narrow zones of joints, rather than a single, discrete joint plane. Many ignimbrites have horizontal joints superimposed on the columnar jointing pattern; occasionally these are sufficiently well developed that they split the material into a blocky structure.

The other common situation is for the jointing to be so widely spaced that the materials are best treated as non-jointed for practical purposes, although some, approximately vertical joints may exist. In general these joints occur in groups separating truly non-jointed zones of up to 15 m width. Unlike the columnar joints, individual joints in this material are tightly closed.

Finally, many ignimbrites show a more complex jointing pattern. Dominantly vertical jointing exists, but the joints are curved along both horizontal and vertical axes and are more closely spaced than their columnar counterparts. They thus form complex joint blocks, often developing a tabular form, with large, vertical plates running parallel to eroded cliff faces. This complex jointing pattern is believed to represent cooling towards a complex cooling surface following emplacement in a deep, narrow valley.

4 GEOTECHNICAL CLASSIFICATION

Possibly the most significant geotechnical property of ignimbrites on which to base a classification of the materials is the slake durability index. The marked dichotomy in response to a weak slaking process allows a ready subdivision of the materials into two principal groups: non-durable ignimbrites ($I_{d2} \leq 30$ %), and durable ignimbrites ($I_{d2} \geq 90$ %). Based on the data described above, the ranges of geotechnical properties associated with these groups are given in Table 2; for most properties the ignimbrites fall into two distinct groups, with no overlap between the ranges given for the two groups.

Although an apparently arbitrary distinction, this classification provides a ready differentiation between ignimbrites which respond to stress in a way generally associated with "normal" rocks (durable ignimbrites), and those non-durable ignimbrites which behave as "soft rocks" - materials which exhibit many properties transitional between engineering rocks and soils (Johnston, 1993). For engineering and slope stability purposes recognising this distinction is critical for the prediction of material behaviour.

4.1 Non-durable ignimbrites

Non-durable ignimbrites are those which break down under a weak slaking process. They are typically low density, highly porous ignimbrites, are very weak in compression, and undergo considerable plastic deformation prior to failure. Softening factors are not strictly related to the durability classification as they tend to vary between individual ignimbrites, but in general the non-durable ignimbrites also undergo the most softening on

Table 2: Geotechnical classification of ignimbrites based on second-cycle slake durability index. Ignimbrites falling into the non-durable category behave as soft rocks where the intact rock properties are an important control on engineering behaviour; in highly durable ignimbrites the rock mass properties of prime importance (from Moon, 1993b).

	non-durable ignimbrites	highly-durable ignimbrites
slake durability		
I_{d2}	≤ 30 %	≥ 90 %
density and porosity		
ρ_{dry}	≤ 1300 kg m^{-3}	≥ 1500 kg m^{-3}
ρ_{sat}	≤ 1700 kg m^{-3}	≥ 1800 kg m^{-3}
η_{eff}	≥ 40 %	≤ 35 %
η_{true}	≥ 50 %	≤ 40 %
compressive strength		
$\sigma_{c,dry}$	≤ 5 MN m^{-2}	≥ 15 MN m^{-2}
$\sigma_{c,sat}$	≤ 2.5 MN m^{-2}	≥ 10 MN m^{-2}
$\sigma_{c,soft}$	> 3	≤ 3
tensile strength		
$\sigma_{t,dry}$		≥ 3 MN m^{-2}
$\sigma_{t,sat}$		≥ 1 MN m^{-2}
$\sigma_{t,soft}$		≤ 2
shear strength		
c	≤ 1 MN m^{-2}	≥ 6 MN m^{-2}
c'	≤ 0.1 MN m^{-2}	≥ 2 MN m^{-2}
c_{soft}	≥ 2 MN m^{-2}	≤ 3 MN m^{-2}
ϕ, ϕ'	27 - 38 °	27 - 38 °
jointing	non-jointed (spacing ≥ 10 m)	very to extremely wide columnar with blocky or complex variants
engineering problems	sensitive materials, piping and gully erosion	high cleft water pressures in jointed rock mass

saturation.

In the field these ignimbrites are non-jointed, as thermal stresses during cooling are released by plastic deformation rather than brittle failure, so joint development is minimal. Intact rock characteristics are thus an important control on slope stability and engineering behaviour, and the materials typically behave as very stiff soils which derive the bulk of their strength from frictional effects between groundmass glass shards.

The relatively high cohesion and the very high angles of internal friction means that these materials can maintain steep slopes for long periods of time (almost vertical artificial cuttings of greater than 40 years age still maintain their initial form). Slope stability analysis (using Culmann wedge and infinite slope analyses) suggests that steep, stable slopes of ≥ 10 m can readily be supported in these non-durable ignimbrites. Indeed, natural slopes in these materials are steep, and maintain a steep profile as they erode, though gully erosion may be a problem (Yamanouchi *et al.*, 1981).

Conversely however, past engineering experience on similar materials suggests that these non-durable ignimbrites may be susceptible to loss of strength caused by structural changes or elevated pore water pressures, resulting in sensitive and dispersive behaviour. The failure of the Ruahihi headrace canal was contributed to by seepage of canal water into non-durable ignimbrite, which resulted in piping of this material, elevated pore water pressures in the non-durable ignimbrite and adjacent artificial fill, and eventual rapid breakdown of the ignimbrite structure with associated loss of strength; these characteristics of the material led to major slope failure (Anon., 1982). Likewise, piping in non-durable ignimbrites is seen as an important problem in these materials for engineering purposes in Japan, especially where they overlie jointed, durable ignimbrites (Okamoto *et al.*, 1981), and piping of non-durable ignimbrites and associated pyroclastic fall deposits lying above a jointed, durable ignimbrite, assisted in the development of major leakage into the foundation rocks of the Wheao Power scheme (Anon., 1983).

4.2 Durable ignimbrites

At the other extreme are ignimbrites which are highly resistant to slaking. These ignimbrites have high bulk densities, low porosities, and high strength. They exhibit some elasticity under an applied load, and most closely approach a brittle failure. Softening factors for strength tend, in general, to be lower than those for the non-durable ignimbrites. The high strength and brittle nature of these materials mean that they develop extensive systems of open, continuous joints during cooling. Their engineering behaviour is thus dominated by the jointing, giving them properties most akin to "normal" rocks in an engineering sense, and making rock mass characteristics of prime importance in terms of slope stability and engineering behaviour.

The most common jointing pattern is the very widely spaced, vertical columnar jointing described previously, though blocky and complex variants may also exist. Overall, the most important aspect of the jointing is the tendency for approximately vertical joint planes to predominate. As a result, in these durable ignimbrites, precipitous faces and narrow gorges represent the normal slope patterns as the materials erode by failure of jointed blocks; dominantly vertical jointing thus results in steep faces. The very wide joint spacing typical of the columnar jointing zone, in particular, produces large blocks which are insensitive to breakdown by the action of water (high slake durability), and these may accumulate as coarse talus at the base of the slopes.

Engineering applications of these materials must recognise the open, continuous nature of the jointing, together with the deformable nature of both the intact rock and the rock mass. Considerable deformation of the country around the Maraetai Dam, for example, was noted by James (1955) following the installation of a grout curtain; this was attributed primarily to closure of the joint apertures (James, 1955). The Arapuni Dam suffered a near toppling failure of a large joint block, upon which the powerhouse was founded, following an increase in cleft water pressure along an open joint plane (Hornell and Werner, 1930; Marshall, 1930), and more recently, the collapse of the headrace canal of the Wheao power scheme occurred after water was allowed to leak into a jointed rock mass (Anon., 1983). Likewise, seepage due to high permeabilities in jointed, durable ignimbrites is quoted as an important problem in these materials for engineering purposes in Japan (Okamoto *et al.*, 1981; Iida *et al.*, 1981).

4.3 Ignimbrites with intermediate characteristics

A few ignimbrites cannot be readily classified as durable or non-durable materials on the basis of the classification given here. This may either be due to their properties being intermediate to the ranges given, or because they have some properties appropriate for each range and hence do not clearly fall into either category. The ignimbrites that cannot be readily classified appear to have undergone some unusual post-depositional alteration processes, such as case-hardening or zeolitization, which have changed their mechanical behaviour.

Consequently, special consideration must be given to ignimbrites which do not readily fall into one of the two main categories. In particular, one or two critical properties, especially the slake durability, may allow the field expression and slope development of the materials to indicate a highly durable ignimbrite, but if subjected to a change in the stress or groundwater conditions the materials may well behave as non-durable ignimbrites with very low strength. Localised hardening must also be considered when anomalously high durability and strength measurements are obtained.

5 PREDICTION OF IGNIMBRITE BEHAVIOUR

As the original basis for the classification, the second-cycle slake durability provides an excellent index property which allows classification of ignimbrite materials. This test can be readily undertaken with minimal material and very simple laboratory equipment, and in almost all cases it gives a good indication of whether the materials behave as stiff engineering soils which may show sensitive and dispersive behaviour, or whether they can be treated as weak, jointed rocks in an engineering sense. Borehole specimens or material from areas of limited exposure are easily tested for slake durability.

For the few specimens recognised where the slake durability did not readily indicate the overall poor engineering response of the materials, the porosity did indicate that the ignimbrites fell into the non-durable category. Hence, the effective porosity in conjunction with the slake durability provides a reliable means of classifying the materials. This test (from saturation methods described by Brown, 1981) can also be simply carried out using a minimal quantity of material and with little specimen preparation.

6 CONCLUSIONS

Ignimbrites are characteristically weak rocks under all forms of stress, but the strengths vary over a wide range due to the genesis of the materials. This range extends across the boundaries traditionally associated with engineering soils and rocks, thus many ignimbrites fall into the category of soft rocks - materials which show properties transitional between those of soils and rocks.

Second-cycle slake durability index data show a remarkable dichotomy, with some ignimbrites remaining almost completely intact, whilst others undergo almost total breakdown. This provides a ready means of classifying the geotechnical properties of ignimbrites into durable and non-durable ignimbrites. This classification provides a clear distinction between ignimbrites which respond to stress as "normal" rocks (durable ignimbrites) and for which the jointing, or mass characteristics, provide the principal control over engineering behaviour, and "soft" rocks (non-durable ignimbrites), for which the intact rock characteristics are of the greatest concern. Recognition of this distinction is critical for any engineering application.

Durable ignimbrites characteristically have systems of open, continuous, vertical joints. Access of groundwater into the rock mass is thus an important engineering consideration, as the materials may develop high cleft water pressures; historical failures of structures founded in these ignimbrites have been largely attributable to this mechanism. Non-durable ignimbrites are subject to rapid breakdown of the structure when subjected to changes in the moisture regime or to weak applied loads. These ignimbrites may thus behave sensitively and are susceptible to gully erosion on exposed slopes or piping within the rock material, leading to major failures if uncontrolled.

The slake durability index provides the best indicator of the overall material behaviour for ignimbrites. However, some materials which have undergone case-hardening or other localised induration processes will give spurious results, so the effective porosity should be used in conjunction with the slake durability to classify the materials. These two tests can both be undertaken with a minimum quantity of specimen, and with simple laboratory apparatus.

ACKNOWLEDGEMENTS

This paper represents a condensed version of those published in *Engineering Geology* by Moon (1993a,b).

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Geotechnical Aspects of the Maui B/A Pipeline

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ABSTRACT: The Maui B/A pipeline is a 15.3 km Pipeline connecting the Maui A and Maui B Platforms off the Taranaki coast of New Zealand. It is buried at least 1 m below the seabed in water depths exceeding 100 m. The pipeline conducts the product at elevated temperatures, inducing thermal instability related problems in the pipe. A number of issues needed to be examined during the design and construction of the pipeline including trench stability, backfill selection and design, thermal properties of soils, and uplift resistance of the backfill materials. The predictions made during design are compared to the observed performance of the pipeline.

1.0 INTRODUCTION

Shell Todd Oil Services Ltd (STOS) are further developing the Maui Gas Field by installing a second platform (Maui B) to service a lobe of the field. For economic reasons it was decided to install this second platform as an unmanned satellite platform controlled from the existing Maui A platform. The Maui B platform will have only sufficient facilities to manifold the wells together and enable maintenance of the facility.

The raw wellstream fluid, comprising a mixture of liquid and gaseous hydrocarbons, carbon dioxide and water, is produced at temperatures above ambient. Of concern was that ice-like gas hydrates formed from methane and water at high pressures and at ambient temperatures, could accumulate and block the pipeline as the fluids cooled due to the lower sea temperature.

2.0 GEOLOGIC SETTING

The site is located some 35 kilometres off the South Taranaki coast (Figure 1). The site is contained within the so-called Taranaki Basin, and is situated within an area of deposition described as the "central mud belt". Silts have been accumulating in this area for the last 6000 years and the average rate of deposition has been on the order of 1 mm/year.

The muds on site vary in thickness between the Maui A and Maui B platforms, but are always over 6 m thick. A layer of sand lies under the muds.

While on the surface, the sea can get very rough, the on-bottom conditions are mild. The currents are less than 1 m/s even under nominal 100 year conditions, and the oscillatory motions due to wave action are similarly slow.

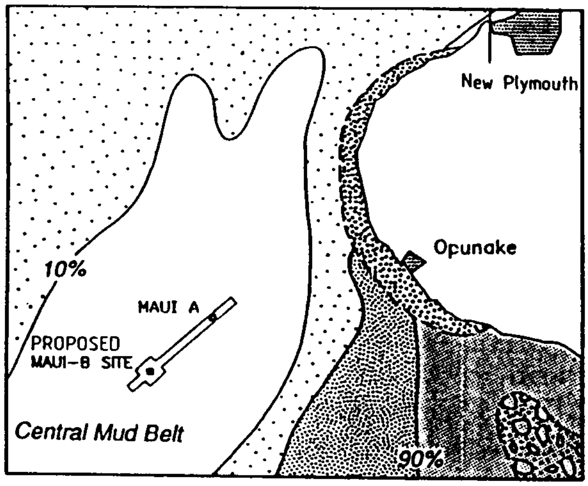

Figure 1 - Site Location

3.0 INVESTIGATION

3.1 Investigation

The general area is well known, as it has been extensively drilled for exploration wells. In addition there was the body of experience developed during the installation of the original A platform and its subsequent operation.

A site specific survey was performed which took four samples around the Maui B site, and four samples along the route (NZOI,1989). CPT's were taken at each location, and thermal conductivity was measured. The survey also included a seismic survey to determine the thickness of mud, seabed topography, and to determine whether there were any major obstacles that would impede the pipe installation process.

The samples were taken by a gravity piston coring device which obtained a sample 60 mm in diameter to a minimum depth of three metres .

Laboratory testing was performed on these samples in order to determine undrained shear strength (via shear vane), atterberg limits, natural moisture content, particle size distribution, and effective stress parameters through consolidated undrained tri-axial testing, and also shear box testing.

A second phase of testing was performed after the installation of the backfill to determine the particle size distribution of the backfill and the width of the dumping profile.

3.2 Results

Figure 2 shows a plot of the vane strength (peak and residual) taken from the cores plotted against depth. A typical grain size distribution of the muds is 25% clay, 5% fine sand, and the remainder is silt. The atterberg limits are typically Plastic limit around 30, Liquid limit around 75, Plasticity Index around 44, and Liquidity Index around 1. The bulk density of the material is around 16 kN/m^3 The cone strength of the material varies from approximately 0.05 MPa at the surface increasing to 0.15 at 2.7 m depth. Sleeve friction varies from 0.005 MPa through to 0.0075 MPa. Some anomalies were noticed in the cone plots where there were areas of no strength recorded. Thermal conductivity was measured to be around 1.2 w/m/°K. The tri-axial data was inconclusive in terms of reliable effective stress data. These results show that material is a very soft to soft sensitive clayey SILT.

The seabed surveys showed that the seabed is essentially flat, with no obstacles along the proposed route. The survey picked up some old anchor drag marks from exploration work done in the 1970's.

4.0 DESIGN REQUIREMENTS

A number of severe geotechnical constraints were imposed on the project. These constraints were

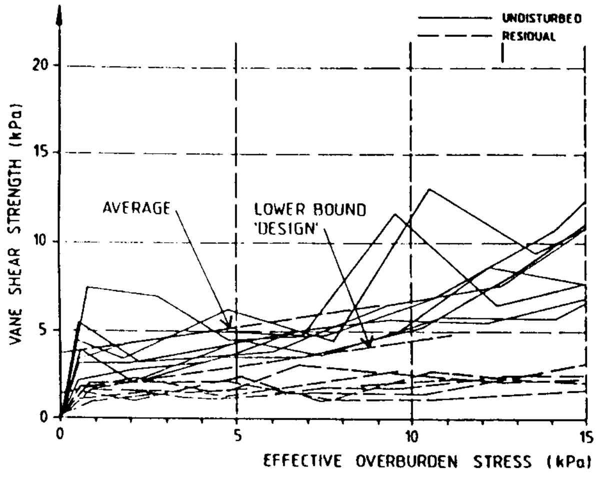

Figure 2 - Vane Shear Strengths vs Overburden Stress

briefly examined during the design phase, and were examined in greater detail during the post-design/pre-construction phase.

4.1 Trench Behaviour

Initial concerns about the trench behaviour were expressed by the contractor. The plough was designed to provide a trench with 30° side slopes and a depth of 2 m, and the soils needed to be stable under these conditions for up to 3 months. No previous trenching had been performed in the area, and the behaviour was unknown. Precedents from other parts of the world were mixed. Stand-up time, percentage of failure, and deformation characteristics were required to be predicted both under the open state, and also in the long term backfilled state. Also the ploughability, pipe sinkage and other construction considerations were frequently discussed.

4.2 Pipeline Behaviour

The pipeline has a design operating temperature of 78°C, and the average seawater temperature is 12.3°C (RJ Brown -Murray North, 1990). The pipe diameter is 0.508 m, with a wall thickness of 22 mm. Figure 3 shows a typical design section. Because the pipeline will be laid at the ambient temperature, and the trench backfilled before the pipe is heated, an axial compressive force will be developed that is proportional to the amount of restraint, and the operational temperature difference.

The pipe line becomes prone to buckling depending upon such factors as backfill uplift resistance, and the deviation of the pipeline from the straight. Construction tolerances required that deviations from the straight in the order of 0.5 m be designed for.

4.3 Backfill Behaviour

A number of backfill issues needed to be considered. The primary issues were those of uplift resistance, and of thermal conductivity. There was also a requirement that the material remained stable under storm conditions.

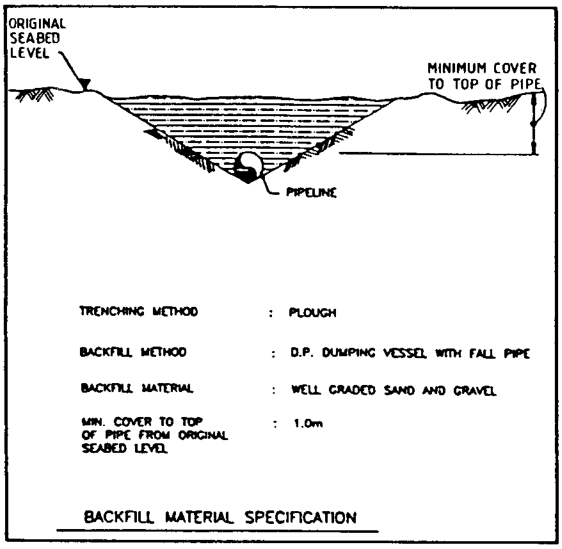

Figure 3 - Typical Design Section

5.0 INTERPRETATION AND ANALYSIS

The available information was required to be evaluated to provide information for the factors discussed above. The evaluated soil strength is shown by the dotted line. This line, while conservative, was chosen because it took into account concerns regarding the reliability of the soils information. These may be compared to the predictions made from the cone plots, and other information such as the anchor drag marks.

Soils modulii were calculated in a number of fashions, including the cone strength data, tri-axial data, as well as multipliers on the shear strength. These all indicated that the appropriate Youngs Modulus for the soil was around 1-2 MPa.

5.1 Trench Behaviour

Calculations based on the Taylor charts showed that the trench had marginal factors of safety of around 1.1-1.2, depending upon the assumed height of the spoil heaps. Given the paucity of the information and the variability, only a rough assessment could be made of the likelihood of failure along the trench length. The Vanmarke (1979) methodology was employed. This predicted that there would be approximately 1 failure per kilometre which would involve approximately 9 cubic metres each. An 'average' shear strength was employed for this analysis.

Some computer modelling was also employed in order to assess the likely deformation behaviour of the trench, both with and without the pipe. This indicated that there was some possibility of the trench sides flowing to the pipe level.

5.2 Pipe Behaviour

Shell's in-house buckling calculation procedures were employed to examine the likelihood of buckling occurring. A series of design charts were developed to assess the required amount of cover for a given degree of out of straightness, and length of out of straightness. These charts were developed initially for a 0.3 m, 0.5 m and 0.7 m out of straightness. The cover depths examined included 0.8 m, 1.0 m and 1.2 m for a gravel backfill cover. The uplift force for a critical 0.5 m imperfection is around 13 kPa.

5.3 Backfill Behaviour

Use of basic upheaval buckling formulae showed that the indigenous material did not have enough uplift resistance to prevent buckling. There were technical concerns about installing the indigenous material as a backfill material, because backplough technology was in its infancy. Hence a gravel backfill was preferred. A well graded material was needed to ensure the stability of the material under storm conditions, and to ensure that the material had a thermal conductivity of less than 2.5 W/m/°K. The contractor chose a material with D_{100} of 75mm, with a D_{50} of around 15mm, and D_{10} of 0.2mm.

The minimum required D_{50} was 3mm. Tests in the quarry showed that the material had a conductivity of around 2 W/m/°K

Theory was developed for use in the event of trench collapse (eg due to an in-filled anchor hole) utilizing a two layer backfill system. The bottom layer consisted of the flowed mud, with a gravel overlay.

Further probabilistic analysis was performed in order to determine the likely quantity of backfill that would be required. This showed that the likely quantity would be of the order of 75,000 cubic metres.

It was found that during the backfill operation that loss of material was occurring, raising the possibility that, if the fine material was being lost, then there was the possibility that thermal heat loss would occur due to convection within the fill as well as conduction. This involved the extension of existing theory (van Traa et al,1989) in order to determine the permeability at which convection would start becoming significant. It was predicted that the convection starts becoming dominant at a permeability of around 10^{-2} m/s for the situation at Maui, and that thermal heat loss increases rapidly once convection gets under way.

Various remedial options were developed in the event that those predictions were verified in the field. The one selected involved placing a remedial blanket over the existing backfill using a low-permeability material. The material finally selected was the indigenous sea bed material.

6.0 PERFORMANCE

6.1 Trench Stability

The trench remained stable after ploughing. There were some extremely minor failures only, which may in fact have been due to spoil material spilling back into the trench. Some spoil material that had clearly fallen was angular and had broken in a brittle fashion. In one area, the pipe had been pushed more than 1.5 m into the sea-bed during laying, leaving near vertical sides in excess of 1 metre high. Trench side deformation was minimal.

The fact that the material stood better than the analysis predicted suggests that sample disturbance weakened the samples considerably. Subsequent samples using a 100 mm tube showed that the real near surface shear strength was more in the order of 5 kPa. The degree of disturbance is assessed to be due to the diameter of the cores, and the transport used between time of sampling and time of testing. The sensitivity is clearly high, and this is well demonstrated in cores taken in the spoil heap, where the inter-clod strengths were very low.

The plough left a very straight and smooth trench, except for areas near each end, where there was some additional imperfections due to starting and finishing the plough and also the pipe push in.

6.2 Pipeline Buckling

The pipeline is behaving in an acceptable fashion, despite the backfill cover being less than designed in a few locations, and the greater than expected imperfections at the two end points.

6.3 Backfill Performance

The probabilistic analysis underestimated (119,000 cubic metres was required) the amount of backfill material for two reasons. The first was that it did not take into account the amount of material lost due to segregation. The second was that too much concern was placed on the possibility that the trench sides would collapse during the analysis.

As already mentioned, the backfill had segregated during the placement operation (through a fall pipe). This meant that the majority of the material left in the trench appeared to be quite coarse. This was confirmed by implication during the initial commissioning of the pipe, when the heat loss was very high, leading to unsatisfactory performance of the pipe. Model tests of the remedial blanket have since been carried out. It is likely that it will be installed to ensure satisfactory thermal performance of the pipeline

7.0 ACKNOWLEDGEMENTS

Thanks are given to those people who have supported the author throughout this project. Special thanks is given to Mr Peter North of Woodward-Clyde for review of work, and editing of this paper, Mr. Richard Press of RJ Brown, for review of work, and also to Mr John Pitman of Shell Todd Oil Services for reviewing and permitting publication of this paper.

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On the interpretation of cone penetrometer data

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ABSTRACT: A simple approximate analysis is presented for interpretation of cone penetration results when the cone resistance is affected by layering of soils with different stiffnesses. It is argued that the cone resistance senses the presence of a nearby layer elastically, and an approximate elastic analysis is developed to quantify the effect. Good comparisons with calibration chamber experimental results are found.

INTRODUCTION

Often when comparisons are drawn between cone penetration tests (CPT) and standard penetration tests (SPT), an advantage is attributed to the CPT due to the continuous nature of the test results. The CPT gives a continuous measure of penetration resistance while the SPT can yield only a mean resistance over a soil depth of 30 cm. SPT results cannot therefore adequately define thin layers or lenses of soil which in certain circumstances may be quite important.

In fact, the CPT itself may not precisely define thin soil layers, even though a continuous measure of penetration resistance is obtained. Many workers in the field have recognized that CPT resistance may be significantly affected by soil layering. As the cone approaches a stiffer layer for example, it will "sense" the presence of this layer some distance before actual penetration of the stiffer soil occurs. Also, after the cone enters the stiffer layer, its response will continue to sense the softer soil above, and this will result in a lower resistance for some distance in the stiffer layer. These effects have been well documented in calibration chamber experiments (Canou, 1989; Foray *et al.*, 1988).

Now consider the case where the cone approaches a thin layer, as illustrated in Figure 1.

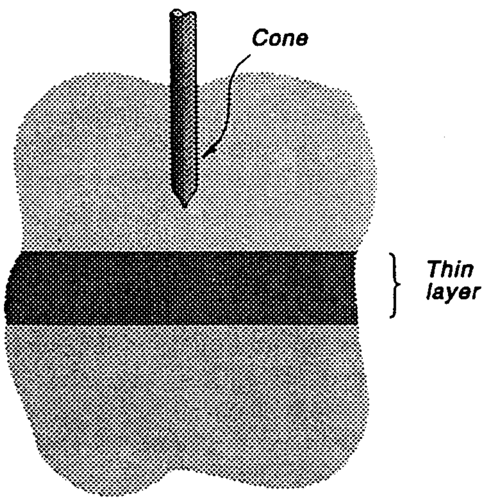

Figure 1. Penetrating cone approaching a layer of dissimilar soil.

The cone will sense the layer as it draws nearer. However, since the layer is thin the resistance will also be affected by the second stiffness change at the bottom of the thin layer. When penetration of the thin layer occurs, the CPT resistance will be affected by both the soil above the layer and the soil below. For quite thin layers, the penetration resistance may be greatly different from the value obtained in a homogeneous deposit of the same soil.

In this paper an attempt is made to model the effects of layering by way of an elastic analysis. At first glance, elasticity may appear to be an extremely poor model for a penetration problem in which large plastic strains must occur near the cone point. However, it is not the actual penetration process that is to be modeled, but the effect on the cone resistance of nearby layers of soil. This suggests that an elastic analysis may prove to be useful since the layering effect at a distance from the cone tip must be essentially elastic in nature. Moreover, the analysis produces results which compare favourably with calibration chamber test results. In fact, there may be potential application of the method to the interpretation of calibration chamber tests as well as to the interpretation of actual field CPT data.

ANALYSIS

Consider two linearly elastic, incompressible half-spaces in bonded contact as shown in Figure 2(a). The CPT will be represented by a disc-shaped region of radius a which supports a uniform applied stress p_o as shown in the figure. Let a be the radius of the cone, and let δ denote the vertical deflection at the centre of the loaded region.

The exact solution to this problem may be obtained by integrating the point-load solution of Plevako (1969), but it is more convenient to employ an approximate analysis based on the Boussinesq half-space, point-load solution. The reason for employing the approximate rather than the exact solution lies in the need to represent multiple layering, for which Plevako's analysis cannot be used, while the approximate solution can. The assumption of incompressibility is based on the expectation that the effects at a distance which are being modeled here will result primarily from undrained soil deformations (there is no difficulty in modifying the analysis to encompass compressible elastic materials).

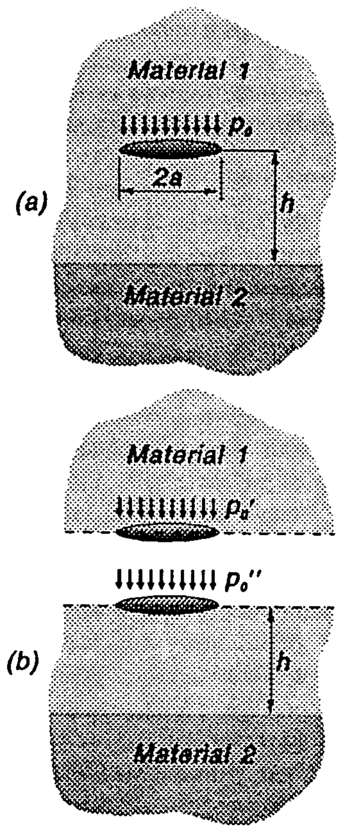

Figure 2. Method of analysis: (a) Representation of CPT by circular uniform load. (b) Decomposition of (a) into two half-space problems.

The first step in the analysis is to decompose the layered infinite space in Figure 2(a) into two elastic half-spaces as shown in Figure 2(b). Each half-space supports a uniform stress, p_o' or p_o'', over the disc-shaped region of radius a. The upper half-space is homogeneous, while the lower half-space is layered. For the upper half-space, Boussinesq's point-load solution may be easily integrated over the disc-shaped region to give the displacement δ' at the centre of the disc. For an incompressible material we find

$$\delta' = \frac{p_o' a}{2G_1} \tag{1}$$

Here G_1 denotes the elastic shear modulus of material 1. For the lower, layered half-space, an approximation can be used, based on Boussinesq's solution, in which the relative displacements in the two layers are combined (as proposed by Poulos, 1967). The displacement δ'' at the centre of the loaded region is given by

$$\delta'' = \frac{p_o'' a}{2G_1} - \frac{p_o''}{2G_1}\frac{a^2}{\sqrt{a^2+h^2}} + \frac{p_o''}{2G_2}\frac{a^2}{\sqrt{a^2+h^2}} \tag{2}$$

The three terms on the right hand side of this equation represent (i) the surface displacement of a homogeneous half-space composed of material 1, (ii) the displacement at depth h beneath the centre of the loaded disc in a homogeneous half-space composed of material 1, and (iii) the displacement at depth h beneath the centre of the loaded disc in a homogeneous half-space composed of material 2. The difference between terms (i) and (ii) represents the relative shortening in the layer of thickness h. Term (iii) represents the displacement of a half-space of material 2 below the depth h. Equation (2) is a well known approximation for the surface displacement of a layered half-space. It compares favourably with exact solutions based on Burmister's analysis (Poulos, 1967).

Now the two half-space solutions in Figure 2(b) must be combined to represent the infinite space problem in Figure 2(a). This is accomplished applying the conditions of compatibility of displacements ($\delta' = \delta'' = \delta$) and equilibrium ($p_o' + p_o'' = p_o$) to obtain

$$\delta = \frac{p_o a}{2G_1}\left[\frac{1-\lambda_1}{2-\lambda_1}\right] \tag{3}$$

where

$$\lambda_i = \left[1 - \frac{G_i}{G_{i+1}}\right]\frac{a}{\sqrt{a^2+h_i^2}} \tag{4}$$

In effect, the two half-spaces are being joined on the horizontal plane passing through the loaded region. This approach can be motivated by the well known fact that, for an incompressible material, Kelvin's solution for a point load in an infinite space gives exactly the same stresses and displacements in the half-space above or below the load point as does Boussinesq's solution for a half-space with a point load equal to 1/2 that used in Kelvin's problem. The approximate solution gives results very close to Plevako's exact solution for this single layer geometry, and it becomes exact in the limiting cases where $h \to \infty$ or $h \to 0$.

It is an easy matter to generalize this analysis to multilayered situations. In this way the displacement δ can be estimated at any point in a multi-layered infinite space. However, it will be convenient to recast the development in dimensionless form. The dimensionless parameter λ_i has already been introduced in equation (4). Two additional parameters are defined as follows, the dimensionless stiffness ratio:

$$k_i = G_{i+1}/G_1 \tag{5}$$

and the dimensionless penetration resistance:

$$\eta = \frac{p_o a}{G_1 \delta} \tag{6}$$

Thus for any number of layers, η may be defined by a functional relationship of the form

$$\eta = F(\lambda_1, \lambda_2, \ldots, k_1, k_2, \ldots) \tag{7}$$

The exact form of F depends on the number of layers involved and the position at which the load is applied. For example, in the simple single layer case of Figure 2(a), for load points above the layer interface

$$\eta = 2\left[\frac{2-\lambda_1}{1-\lambda_1}\right] \quad \text{for } h > 0 \tag{8}$$

while for load points below the layer interface

$$\eta = 2k_1\left[\frac{2+k_1\lambda_1}{1+k_1\lambda_1}\right] \quad \text{for } h \le 0 \tag{9}$$

Now using equations (8) and (9) the dimensionless penetration resistance η may be plotted as a function of position. Note the behaviour of η at points distant from the interface, and at the interface itself.

$$\begin{array}{lll} \text{for} & h/a \to \infty, & \eta \to 4 \\ \text{for} & h/a = 0, & \eta = 2(1+k_1) \\ \text{for} & h/a \to -\infty, & \eta \to 4k_1 \end{array}$$

It is apparent that η will move smoothly through the interface, varying between a value of 4 in material 1 and a value of $4k_1$ in material 2. Note also that at the interface the value of η is exact. For more complex layering, equations (8) and (9) must be replaced by more complex expressions, but this is a straightforward matter.

In what follows η will be used to model actual penetration resistance as a function of depth. This is not to suggest that the elastic analysis given here accounts in any way for the immediate effects of penetration and plastic deformation which occur near the tip of a penetrating cone. The analysis is intended solely to model the effects of layering sensed by the cone in its own vicinity. To accomplish this, when actual values of cone resistance, q_c, are considered, they will be made dimensionless in such a way that they will agree with the value of η when there is no layering in the near vicinity. Referring to the individual terms on the right hand side of equation (6), we assume p_o and a correspond directly to the measured penetration resistance q_c and the cone radius. The quantity $G_1\delta$ must be set equal to a constant reference value which is chosen so that η in the reference layer (material 1) tends to 4 at large distances from the layer 1 - layer 2 interface, that is

$$G_1\delta = \frac{1}{4}\bar{q}_c a \tag{10}$$

Here $\bar{q}_c$ is the mean value of measured penetration resistance in layer 1. In one sense this calibrates the model to the actual data. In another sense it completely avoids the problems associated with the actual penetration process and focuses attention on the elastic effects of layering.

COMPARISON WITH EXPERIMENTAL DATA

A number of calibration chamber experiments have been carried out using layered soil profiles by Canou (1989) at CERMES, Paris, and by Foray (1988) and co-workers at the IMG, Grenoble. The CERMES tests employed a mini-cone of diameter 11.3 mm; the calibration chamber having dimensions of 180 mm diameter and 400 mm depth. In the Grenoble tests, a Parez cone of 45 mm diameter was used in a chamber measuring 1.20 m in diameter and 1.50 m in depth.

Figures 3(a), 3(b) and 4 show dimensionless mini-cone resistance data (solid lines) and the corresponding theoretical resistance curves η for two calibration chamber tests. In each case the actual mini-cone resistance is made dimensionless in such a way that its average value in the upper layer away

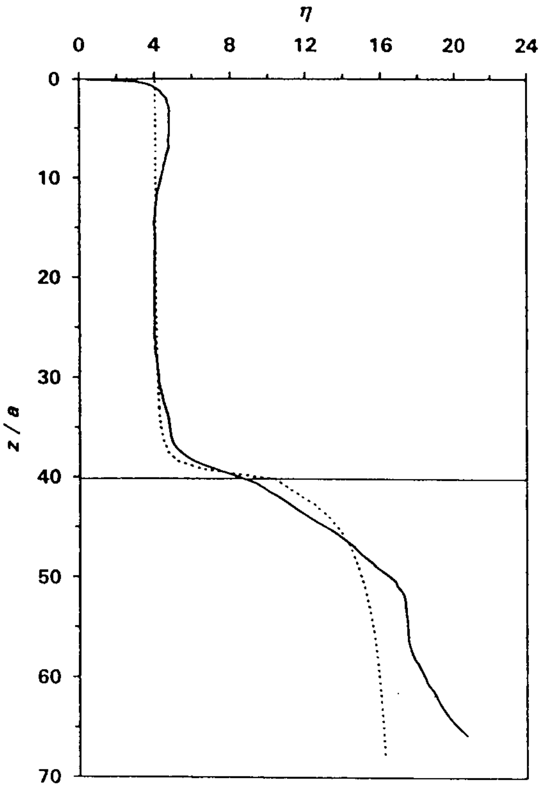

Figure 3(a). Comparison of theory with calibration chamber results (without chamber base).

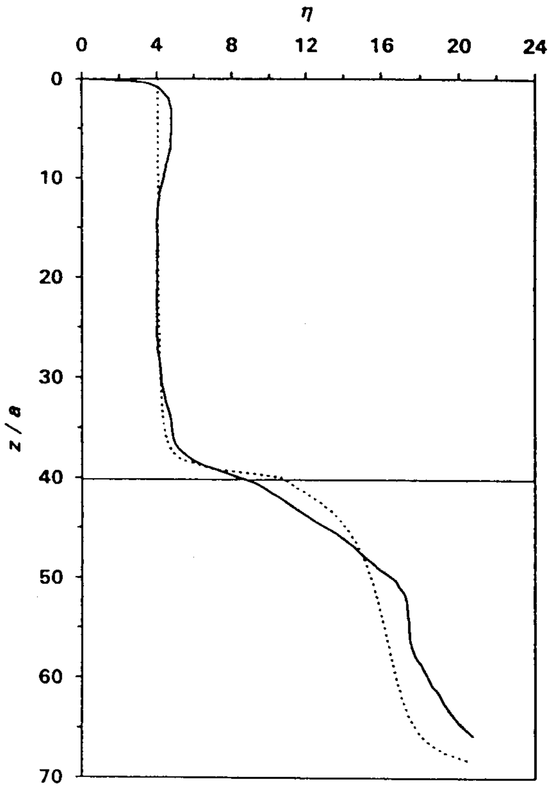

Figure 3(b). Comparison of theory with calibration chamber results (with chamber base).

from the interface is equal to 4. Vertical dimensions are shown measured from the upper soil surface in the chamber, and are made dimensionless by dividing by a.

Figures 3(a) and 3(b) show results from an experiment involving a soft layer over a stiffer layer. The experimental data (solid lines) in both figures are the same, but the theoretical results (dashed lines) are different. The theoretical dashed line on Figure 3(a) was obtained using equations (8) and (9), with k_1 set to 4.29. Calculations were performed without attempting to represent the chamber base, and consequently the experimental and theoretical curves bear only a reasonable likeness. In Figure 3(b) the base was included in the theoretical calculation by introducing a near-rigid layer at dimensionless depth 70.9. Figure 3(b) appears to model the experimental data remarkably well and the effect of the chamber base is seen to extend a significant distance upward into the sample.

Figure 4 shows an experimental plot in which three soil layers were involved: a soft layer sandwiched between two stiffer layers. The stiffness ratios used to model these results were $k_1 = 0.34$ and $k_2 = 2.51$. A rigid base layer was included in the calculation also.

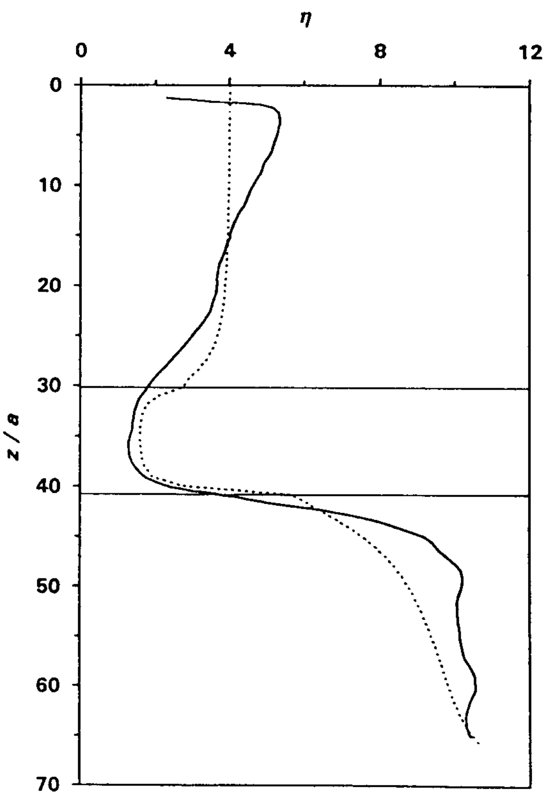

Figure 4. Comparison of theory with calibration chamber results. Three layer example.

Clearly, the theoretical results shown here do not agree exactly with the experimental data. Nevertheless, the theoretical penetration resistance appears to adequately model the layer effects observed in the experiments, and there is a remarkable degree of correlation despite the noisiness evident in the experimental traces.

Finally, it is evident from the figures that a soft layer requires only a relatively short distance of penetration for cone resistance to reach a steady state value. This seems reasonable since a soft layer would be expected to isolate the penetrating cone from materials it has yet to encounter, while a stiff layer will not have this effect.

APPLICATIONS

The most obvious use for this analysis appears to be in the interpretation of cone resistance data where thin soil layers are involved. As an example, consider the following actual field record. Figure 5 shows a section of a piezocone (CPTU) penetration resistance record obtained at a site near the Salinas River mouth in central California. There appears to be a thin layer of stiff soil embedded between two softer layers, within the segment between 8.5 m and 10 m depth.

This segment of the record will be modeled using the above analysis. The cone had a standard radius a of 17.84 mm. The raw value of $\overline{q}_c$ recorded in the soft soil was, on average, 0.8 MPa. This was converted into dimensionless form such that $\eta = 4$ in the soft material, and $G_1\delta$ was set equal to

$$G_1\delta = \frac{1}{4} \times 0.80 \times 17.84 = 3.58 \text{ MPa} \cdot \text{mm}$$

The positions of the two interfaces at top and bottom of the stiff layer were determined from the pore pressure record obtained simultaneously with the penetration record. Clear cut jumps in pore pressure occurred both at the top and bottom of the layer.

Based purely on the perceived values of q_c in the soft soil and in the layer, a value of k_1 equal to 5.4 was initially used to calculate the theoretical response. That calculation produced a theoretical curve which severely underestimated the resistance in the layer as shown in Figure 6. Returning to Figure 5 for a moment, one might suspect that the thin layer material could possibly be the same as that found above 8.5 m or below 10 m, but that its perceived resistance is diminished due to its small layer thickness.

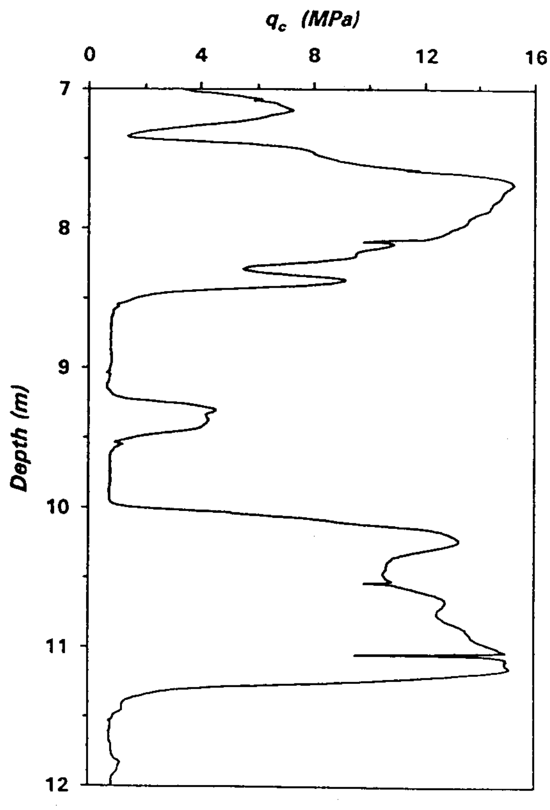

Figure 5. Measured penetration resistance at a site near the Salinas River in California.

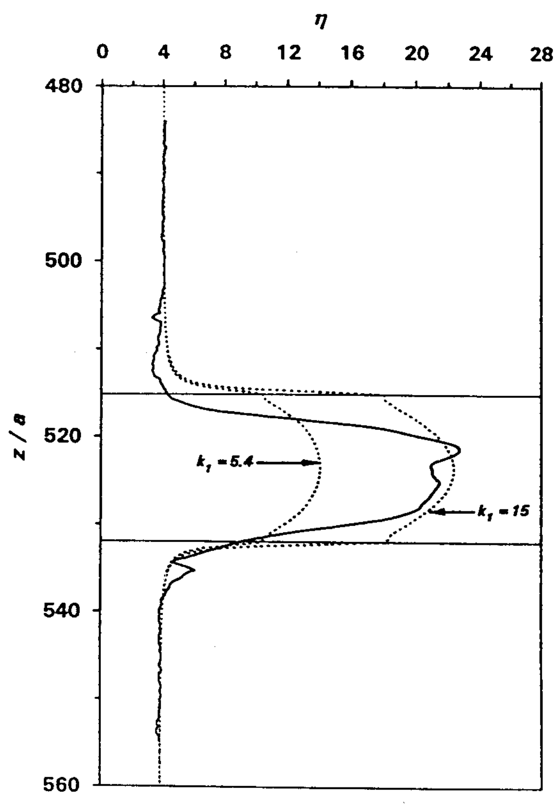

Figure 6. Enlargement from Figure 5 showing comparisons with theoretical response for two stiffness ratios.

The soils above 8.5 m and below 10 m both have mean penetration resistance roughly 15 times that found in the soft material. This suggests resetting k_1 to 15 and recalculating the theoretical response. When this is done the theoretical curve agrees with the field record much more closely as shown in Figure 6. One is led to conclude that the thin layer at depth 9.4 m is in fact identical to the soils found above 8.5 m and below 10 m. Clearly this conclusion would not be reached based on the raw data shown in Figure 5.

One final point can be made concerning Figure 6. In the figure, the location of the top of the layer appears to be slightly higher than the field resistance curve would suggest. This may be due to the fact that the standard procedure of referring all depths to the location of the cone tip has been followed. It may be that the experimental result should be compared with the analytical solution at the depth where the entire projected area of the cone is embedded, rather than at the tip. If allowance was made for this fact, the upper interface would be shifted downwards approximately $2a$, and the analytical solution would fit the experimental data even more closely. Of course, the lower interface would also shift downward by the same amount, and this might cause the theoretical data to fit somewhat less well at the base of the layer. It should be noted however that, at a dimensionless depth of $z/a = 534$, the field penetration was halted for a brief time in order that a new drill rod be added to the string. This would have resulted in an unload/reload situation at the cone tip, and possibly distorted the experimental results near the base of the layer.

CONCLUSIONS

In this paper a simple elastic analysis has been presented to explain how cone penetration resistance may be affected by the presence of nearby soil layers with different stiffness characteristics. It is suggested that the perceived penetration resistance may "sense" nearby layers. Comparisons with experimental data obtained in calibration chamber experiments and in the field have been presented which bear this out. The analysis has clear implications concerning the interpretation of cone data for soil classification, particularly where strength or liquefaction potential of thin layers happens to be under consideration. It is easily conceivable that a thin layer might be classified as susceptible to liquefaction based on perceived cone data, whereas, in fact, it is not. The simple elastic analysis presented here allows for a more reliable and realistic interpretation of any cone penetration data.

ACKNOWLEDGEMENTS

The author wishes to acknowledge the assistance of the New Zealand Earthquake and War Damage Commission (EQC) both for their direct support and for their general support of research into *in situ* testing of liquefaction sites. The calibration chamber results were obtained within the joint French-United States-New Zealand *Projet Piézocône*, funded principally by the French Ministry of Research and Technology, but also by the New Zealand Ministry of Energy, Transit New Zealand and the EQC. The Salinas River CPTU record results from a joint project with the United States Geological Survey, and the cooperation of the United States Geological Survey is gratefully acknowledged.

Finally, the author wishes to thank Dr Rob Davis and Dr John Berrill of the University of Canterbury for their assistance in the development of this analysis and in the preparation of this paper.

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Topics: Rock Mechanics and Waste Disposal, Laboratory Testing, Weak Rock Engineering, Slope Stability, Neotectonics, Seismicity, Mining, Rock Dynamics-Drilling and Blasting. Abstracts due August 26, 1994.

AUGUST 29 - SEPTEMBER 2, 1995
Beijing, China
10TH ASIAN REGIONAL CONFERENCE ON SOIL MECHANICS AND FOUNDATION ENGINEERING
Topics: Soil properties, Regional soils and their engineering behaviour, Deep and shallow foundations, Earth Structures and underground geotechnics, Ground improvement techniques; Natural hazard and environmental geotechnics.
Language: English

SEPTEMBER 23-29, 1995
Nakase, Chiba, Japan
8TH INTERNATIONAL CONGRESS ON ROCK MECHANICS
Keynote: Frontiers of Rock Mechanics towards the 21st Century. Main Session Themes: Geology, site exploration and testing; Physical properties and modelling of rock; Near surface excavations, Stability of slopes and foundations; Excavation and stability of underground openings; Heat, water flow and chemical transport in rock masses; Information system and artificial intelligence in rock mechanics.

NOVEMBER 14-16, 1995
Tokyo, Japan
INTERNATIONAL SYMPOSIUM ON DYNAMIC BEHAVIOUR AND DAMAGE OF GROUND CAUSED BY RECENT EARTHQUAKES
Topics: Laboratory and in-situ tests on dynamic behaviour of soils, including model tests. Case histories of recent earthquakes with emphasis on dynamic response of grounds, liquefaction problems, and ground failure. Abstracts: by 30 July 1994.
Language: English

DECEMBER, 1995
Cairo, Egypt
XI AFRICAN REGIONAL CONFERENCE ON SOIL MECHANICS AND FOUNDATION ENGINEERING

1996

JUNE 17-21, 1996
Trondheim, Norway
7TH INTERNATIONAL SYMPOSIUM ON LANDSLIDES
Topics: Analysis of Landslides Inventories; Landslide investigations; Monitoring and instrumentation; Stability analyses and geotechnical parameters; Shoreline stability and submarine slides; Assessments of landslide risk and hazards; Stabilisation and remedial works; Open-pit mine slopes and mine tailings; Slope instability in tropical and seismic areas; Landslides in sensitive soils.
Language: English and French

1997

SEPTEMBER 6-12, 1997
Hamburg, Germany
XIV INTERNATIONAL CONFERENCE ON SOIL MECHANICS AND FOUNDATION ENGINEERING

Footnote: For further details on contacts for any of the above conferences or symposia please contact the editor of NZ Geomechanics News.

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APPLICATION FOR MEMBERSHIP
of
New Zealand Geomechanics Society

A TECHNICAL GROUP OF THE INSTITUTION OF PROFESSIONAL ENGINEERS OF NEW ZEALAND

The Secretary
The Institution of Professional Engineers of New Zealand
P O Box 12-241
WELLINGTON

I believe myself to be a proper person to be a member of the N.Z. Geomechanics Society and do hereby promise that, in the event of my admission, I will be governed by the Rules of the Society for the time being in force or as they may hereafter be amended and that I will promote the objects of the Society as far as may be in my power.

I hereby apply for membership of the N.Z. Geomechanics society and supply the following details:

NAME: ______________________ in full in block letters, surname last)

PERMANENT ADDRESS: ______________________

____

QUALIFICATIONS AND EXPERIENCE: ______________________

____

NAME OF PRESENT EMPLOYER: ______________________

NATURE OF DUTIES: ______________________

Affiliation to International Societies: (All members are required to be affiliated to at least one Society, and applicants are to indicate below the Society/ies to which they wish to affiliate).

I wish to affiliate to:

International Society for Soil Mechanics for Foundation Engineering	(ISSMFE)	Yes/No	($16.00)
International Society for Rock Mechanics	(ISRM)	Yes/No	($16.00)
International Association of Engineering Geology	(IAEG)	Yes/No	($10.00) (with bulletin) ($37.00)

SIGNATURE OF APPLICANT: ______________________

DATE: ______________________

NB: Affiliation Fees are in addition to the basic Geomechanics Society membership fee of $36.00 which is reduced to $26.00 if member of IPENZ.

PLEASE DO NOT SEND FEES WITH THIS APPLICATION. AN ACCOUNT WILL BE SENT ON YOUR ACCEPTANCE INTO THE SOCIETY.

Nomination:

I ______________________ being a financial member of the N.Z. Geomechanics Society hereby nominate ______________________ for membership of the above Society.

Appendix

Proceedings of the Symposium on

Geotechnical Aspects of Waste Management

Supplementary Papers

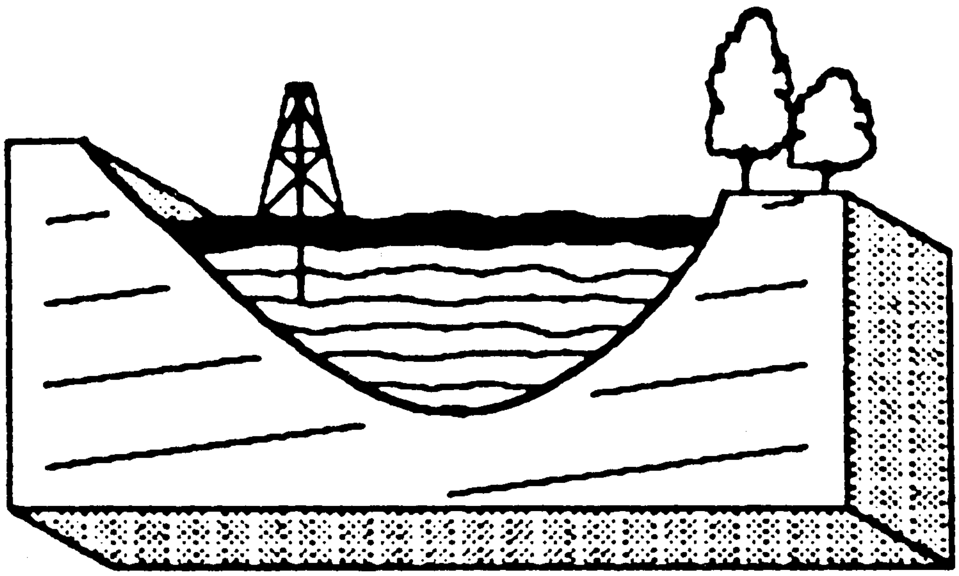
Wellington

May 1994

GEOTECHNICAL ASPECTS OF WASTE MANAGEMENT

VICTORIA UNIVERSITY OF WELLINGTON 13 - 14 MAY 1994

SUPPLEMENTARY PAPERS



This publication contains those papers which were presented at the New Zealand Geomechanics Society's Symposium on Geotechnical Aspects of Waste Management but were not available at the time of printing the Proceedings, (Volume 20, Issue 1(G))

Published by:
The Institution of Professional Engineers New Zealand
101 Molesworth Street, Wellington, New Zealand, 1994.

Proceedings of Technical Groups Volume 20
Issue 2(G)
ISSN 0111-9532

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PAPERS

☒ Papers published in the first issue of the Symposium Proceedings

MINIMISING ENVIRONMENTAL LIABILITY IN WASTE MANAGEMENT

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SYNOPSIS

The paper looks at firstly, the contract of service, which can be a key document determining environmental liability. Then the paper addresses some of the measures should be undertaking (at minimum) to ensure compliance with environmental legislation.

The paper then looks at environmental management systems as a way in which you can be more confident about your environmental performance and reduce the risk of non-compliance with environmental legislation. Adopting an environmental management system is also likely to assist in any enforcement proceedings or prosecution taken against you. The paper considers BS 7750:1994 Specification for Environmental Management Systems which many New Zealand companies are adopting or proposing to adopt.

INTRODUCTION

A concern to minimise environmental liability is understandable. It should not necessarily be seen in a negative light as it has a positive side. It should result in improved environmental performance by all those involved in waste management.

This paper assumes that most of the audience are professionals with a concern about how they can give advice on waste management, design and supervise the construction of landfills and other waste treatment facilities, and ensure their long term environmental security.

THE TRANSITIONAL AND NEW REGIME UNDER THE RESOURCE MANAGEMENT ACT

A down side of the Resource Management Act is the need to come to terms with the new plans and requirements for resource consents. It is risky to make assumptions about the implications of the Resource Management Act for the particular waste management activity you are engaged in.
Before undertaking any job consider the environmental implications and where appropriate find out about the environmental requirements. Relying on the advice given by council officers over the counter or the telephone can cause problems. It is advisable to put your requests in writing and to obtain a written response. Even then there are risks that another party can challenge the council's interpretation of its plan or a resource consent.

It will often be advisable to seek expert advice on the legal position under the Resource Management Act.

THE CONTRACT OF SERVICE

Many of you will work for clients. The basis of your working arrangements will be a contract between you and your company and the client (or a consultant if you are a sub-consultant). This contract will probably contain a clause requiring you or your company to comply with all relevant Acts, regulations, and bylaws. This sounds reasonable as everyone has to comply with the law.

Before agreeing to such a contract provision you should consider what these requirements will mean in the case of the services you are supplying.

There are two major issues to be aware of. One is the issue of obtaining consents for activities. The other is the issue of liability.

Obtaining Consents

You will need to clarify whether the client requires you to obtain any consents necessary for the activity you are advising on or undertaking. In many cases it will not be practicable for you to obtain the resource consents.

It is likely that the IPENZ Model Conditions of Engagement will be the basis of engagement. These explicitly specify that it is the responsibility of the client to:

> *"Obtain all approvals, authorities, licences and permits which are required from governmental, territorial, statutory or responsible authorities for the lawful implementation and completion of the Project, unless otherwise stated ... "*

[3.4 (iii)]

The IPENZ Short Form of Agreement for Engagement of a Consultant does not contain a similar provision which seems an unfortunate oversight. Specific requirements therefore need to be written into the short form or else the consultant may be responsible for obtaining the resource consents or other approvals. You should therefore clarify the question of who will obtain the necessary approvals with the client.

Pass it on ...

There is a growing trend for bodies to attempt to pass on their liability to consultants and contractors. This is an undesirable trend when it means that consultants and contractors must assume liability for actions which are outside their control. It is likely to result in poor environmental outcomes. The attitude the Courts will take to such clauses is unknown as yet.

You should therefore look out for clauses in draft agreements which attempt to pass on liability to you. You should consider how reasonable this is in the circumstances of the work you are being expected to do. Another aspect to it is the cost of the job to the client. Assuring compliance by people outside your direct control may cost you a significant amount. It may be more cost efficient for the client to do this.

ENSURING COMPLIANCE

Consultancy Advice

You should ensure that your client is aware of the implications of environmental legislation for the job. You should also obtain a copy of any consent, or conditions on an activity, where these may impact on the job, for example by affecting design parameters.

It is important to be aware of a client's responsibilities under the Resource Management Act in respect of a particular project. Not being aware of the client's responsibilities under environmental legislation can harm your business. It might mean that the client receives advice that cannot be acted on, or that results in enforcement action. This may result in a negligence action by the client.

Field/Site Work

Before undertaking field/site work check the position under the Resource Management Act and other environmental legislation. If the work involved requires a resource consent, ensure that you have a copy of that consent. It is likely to contain conditions which you must comply with.

Even if the activity is one which is permitted without a resource consent by a regional or district plan, it may be subject to performance standards or other conditions. You must know what the permitted activity status allows. It may be advisable to obtain a certificate of compliance for a permitted activity, particularly in cases where there is a new plan which is likely to be notified.

Check that your proposed programme of work complies with all standards, terms, and conditions. Compliance with some conditions may require monitoring, such as noise. If such is the case, then monitoring must be done. If you are unclear as to what the conditions might involve, consult with the consent authority.

You or your company can be the subject of enforcement action or a prosecution under the Resource Management Act or other environmental legislation in the event of non-compliance. Even if the client is the subject of enforcement action, the client can still take an action in negligence against you or your company.

Company Culture

It is essential to have a company culture that places importance on meeting its environmental responsibilities. This commitment must come from the top.

There are numerous ways a company could foster an environmentally responsible company culture. These include having a company environmental policy, staff training, having dedicated "environmental" staff, sponsoring environmental projects, and making environmental performance a factor in rewarding staff.

One way is to have an environmental management system.

ENVIRONMENTAL MANAGEMENT SYSTEMS

Overseas experience has shown that in the absence of a common and accepted understanding as to what constitutes "responsible environmental performance" there is no yardstick to determine whether a company is in fact being responsible. This has led to the development of the concept of what has come to be termed, an "environmental management system". What this concept does is to bring together into a systematic structure all the good ways of being environmentally responsible.

Systems Available

The British Standards Institute [BSI], building on its quality assurance standard [BS 5750, which is equivalent to the ISO/NZS 9000 series], introduced in 1992 a standard specification for environmental management systems. BSI piloted the standard and review it in light of the pilot study. The latest version of the standard is referred to as BS 7750:1994. BS 7750 is currently the only **standard** specification available for environmental management systems.

The European Community has developed and published a **regulation** [Council Regulation (EEC) No 1836/93 of 29 June 1993] which allows the voluntary participation of industrial companies in an eco-management and audit system. This regulation was being finalised during the time that BS 7750 was being reviewed and the Foreword to BS 7750:1994 states:

> *This standard has been produced with the express intention that its requirements should be compatible with those of the environmental management system specified in the European Community's Eco-Management and Audit (EMA) Regulation ...*

The International Standards Organisation [ISO] is in the process of developing an international standard for environmental management systems and publication is expected by December 1995. As Britain is providing the secretariat for the technical subcommittee charged with developing the standard it is highly likely that the ISO standard will bear a close resemblance to BS 7750:1994.

In 1992, Standards New Zealand and Standards Australia established a joint committee to look at producing a joint Standard. The committee has had problems determining just what was needed but the current intention is to produce an interim Standard. This would mean that the document is issued for public comment for a period of two years during which time the international standard should be available.

Even though there is no New Zealand standard on environmental management systems, it is possible to be certified to BS 7750 by certifying authorities in New Zealand such as TELARC or BVQI.

BS 7750:1994 Specification for Environmental Management System

To give an indication of what an environmental management system might involve, the paper describes the elements of BS 7750.

The implementation of BS 7750 requires an assessment of your current operations and their compliance with relevant environmental law. This is the function traditionally performed by "audits". A register must be compiled of relevant environmental laws and environmental effects caused by your activities. An important part of the system is allocating management responsibilities for environmental performance.

However, BS 7750 does not stop there. It requires the development of an environmental policy. An element of this is a commitment to continual improvement in environmental performance. This policy must be publicly available. Objectives and targets and a management programme to achieve them also follow.

Manuals and operational controls and procedures as well as staff training ensure that all staff know what is expected of them. Record-keeping is also important. Finally there is the requirement for regular audits of environmental performance against the environmental policy, objectives, and targets. Regular reviews of the system can result in changes to it.

Relationship to Quality Assurance

In the initial development and subsequent review of BS 7750 and in the work of ISO, there has been a lot of discussion as to the relationship between an "environmental management system" and a "quality assurance system".

The Foreword to BS 7750:1994 states:

> *The management system elements specified in this standard do not need to be established independently of existing management system elements. In some cases it will be possible to comply with the requirements by adapting existing management system components.*
>
> *This standard shares common management system principles with BS 5750 (EN 29000, ISO 9000), the European and internationally recognised quality systems standard, and organisations may elect to use an existing management system developed in conformity with BS 5750 as a basis for environmental management.*

In addition, Annex B to the standard, provides information on the links between the two standards. It contains a useful table which sets out the links in a matrix form.

The reason why Britain developed an environmental management standard separate from its quality assurance standard, BS 5750, was that not all companies interested in the environment necessarily wanted to develop a quality assurance system. It remains to be seen what happens at the ISO level.

The Benefits

There are many benefits of having an environmental management system, such as BS 7750, besides minimising environmental liability. However, this paper is not about them.

An environmental management system should assist you to identify areas where non-compliance with environmental laws is likely to be a problem. It should ensure that there are procedures in place to prevent violations of environmental laws. Where there are incidents of non-compliance the environmental management system should mean that there are effective procedures in place for dealing quickly and appropriately with the cause of the problem and handling clean-up.

All of these points are likely to assist you in the event of enforcement action under environmental laws. There are some interesting cases coming through the Courts which demonstrate the potential advantages of having an environmental management system.

Sections 340 and 341 of the Resource Management Act contain what commentators loosely describe as the "due diligence" defences to prosecution for having no consents or for non-compliance with consents. There are elements in these defences which an environmental management system may assist in proving.

In *Shell Oil New Zealand Limited v Cudby Motors Limited*, a case that involved a petrol leak at a service station, Shell, which owned the service station, successfully argued that it was environmentally responsible. I quote from the case:

> *I [that is the judge] am satisfied that the company is environmentally conscious and that the management and directorate take a keen interest in environmental matters running courses and generally overseeing the activities of its managerial staff ... I cannot find Shell New Zealand guilty of the offence as charged and consider it has discharged the onus placed upon it by the Resource Management Act in proving its defence.* [Pages 4 and 18 of the decision].

Contrast this with what the judge had to say about Cudby Motors Ltd, which leased the service station from Shell:

> *In effect the actions of its manager, which I hold to be actions sanctioned by the company through its directors ... is grossly negligent ... I hold the case against Cudby Motors (1985) to have been proven and that the evidence does not discharge the onus upon them concerning strict liability.* [Pages 18 and 20 of the decision].

The High Court made it very clear in the *Machinery Movers Limited v Auckland Regional Council* case that good environmental practices can minimise liability. The Court found that a factor relevant to sentencing was the extent to which the defendant had sought to comply with environmental legislation. In order to assess this, the Court noted pertinent factors may be:

- the adoption of appropriate in-house corporate environmental principles; and
- the existence of an internal environmental compliance programme.

The District Court followed the *Machinery Movers* approach to sentencing in *Taranaki Regional Council v Chrome & Chemicals (NZ) Limited* and *Auckland Regional Council v Bitumix Limited.*

A Warning

An environmental management system, such as BS 7750, leads to a much greater level of documentation of environmental performance. This documentation covers poor performance (as well as good performance) and could increase the risk of having easily obtainable evidence available in the case of enforcement action by regulatory authorities and others.

Implementation of an environmental management system requires a commitment to follow up actual or suspected non-compliance if the risks of legal action are to be reduced. An organisation could be worse off if it partially implemented an environmental management system, documented its non-compliance, and did not follow up with corrective action.

CONCLUSION

There are a number of ways that you as professionals can minimise your environmental liability. These are:

> Before undertaking any job consider the environmental implications and where appropriate find out about the environmental requirements.
>
> Ensure that any contract for services adequately recognises and provides for environmental requirements and does not seek to impose an unreasonable degree of environmental liability on you.

Put measures in place to ensure that you, and everyone working with you, comply with the environmental requirements of the legislation and the consent authorities.

Consider adopting an environmental management system which will help ensure that compliance.

EFFECTS OF THE RESOURCE MANAGEMENT ACT ON WASTE MANAGEMENT PRACTICES

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SYNOPSIS

The effects of the Resource Management Act on waste management practices is an evolving process. In a practical sense, the Act provides an excellent framework within which to develop and sustain workable waste management strategies in New Zealand. Timing has meant that waste management planning is often ahead of some of the statutory processes that govern it, but provided the plans are soundly based, and have taken account of effective consultation, risks of major reworking being required should be small.

The Act challenges us to consider alternatives. To do this effectively, we need to think laterally and not be constrained by personal experiences, expectations and prejudices, while retaining sufficient control to avoid going off on expensive tangents that lead nowhere.

The Act requires wide consultation, which brings with it the need for technical specialists to develop the ability to communicate using language that other non technical people can understand.

The Act is not specific on exactly what consents are required, leaving much room for differing opinions with regard to landfill projects. In the authors' view, up to eight consents could normally be required for most operating landfills.

It is possible that we are having to be conservative in our current designs for new landfills because of an incomplete understanding of all the processes that affect the performance of soils in containing and attenuating leachate produced in landfills. We need to work towards having greater confidence in defining the actual effects rather than the potential effects based on conservative theory.

In conclusion, it will be a very long time before we fully understand all aspects of the Resource Management Act, but if used wisely, it does provide an excellent framework within which to develop sound waste management strategies for the future.

INTRODUCTION

The effects of the Resource Management Act on waste management practices is an evolving process, and one which we are unlikely to fully understand for some time yet. There is also a grey area between the actual effects of the Act and effects resulting from differing, but not necessarily correct, perceptions of what the Act requires. Different users of the Act often interpret it differently, with the Ministry for the Environment, different Regional Councils, applicants for consents, submitters, specialist waste management advisers and lawyers all having their own views.

In a practical sense, the Act provides an excellent framework within which to develop and sustain workable waste management practices. As such, it provides us with a unique opportunity to put waste management in New Zealand on a sound footing. However, there are a number of very real threats that could jeopardise the opportunity if we are not careful. We need to recognise them and work towards overcoming them.

This paper takes into account experiences of working in the waste management field both before and after the Act was enacted. In many respects, the Act merely formalises what was starting to happen as a result of experience of what does and does not work. The paper also focuses on the broader issues of the Act and those most directly relevant to this Symposium, rather than going into detail.

The paper is based mainly on the authors' method of working with the Act. The authors' experience includes working for waste management operators, both local government and private, regulatory authorities and objectors, as well as in a peer review capacity. It is thus hoped that the comments made reflect a balanced view of the issues.

MAIN REQUIREMENTS OF THE ACT WHICH AFFECT WASTE MANAGEMENT

(a) Requirement to Comply with Plans

In one way or another, central, regional and local government all have either obligations or opportunities or both to prepare plans or implement policies that could affect waste management at the service delivery level. Few legally binding plans or policies prepared under the Act have yet been introduced, but many districts and cities have taken the initiative and developed their own waste management strategies in advance. Where these are well thought out, environmentally sound, and based on proper consultation, it is unlikely that they will require major modification to comply with any later plans that might apply to them.

While risks still remain, it is still better to continue the planning process and as a consequence, local government is now planning its waste management activities much better than at any time in the past. Where consents for future works will be required under RMA, and the works can be demonstrated to be in accordance with a soundly based waste management strategy, it places the applicant in a much stronger position. It is perhaps arguable if the position is legally stronger, but in terms of the practical realities of defining choices in waste management consent processes, the opportunities for serious threats to the case should be substantially reduced.

(b) Requirements to Consider Alternatives

The requirement to consider alternatives should form an essential part of developing any waste management strategy, or plan, at any level of government. The true benefits of considering alternatives can only be obtained if you are genuinely interested in exploring <u>all</u> the options and finding the best solution for the community concerned, regardless of personal experiences, expectations and prejudices, the last being particularly important to overcome.

To do this successfully, and to identify all the possibilities, you will need to think laterally. You may also need to spread your net wide and seek inputs from any party who might be able to provide a solution. If you are open-minded, you may find that solutions you might once have discounted without much serious consideration can be made to work and offer a better overall solution. On occasions, you can also get some very pleasant surprises.

Some caution is required, however, when considering alternatives, as a great deal of time, effort and money can be spent on options that lead nowhere. To have any chance of success any option must be:

- actually available and proven to work;
- politically, financially, commercially and environmentally sustainable;
- have access to the waste needed to make them work;
- without unacceptable social impact.

An initial evaluation of these key factors will generally eliminate impractical options.

To illustrate the benefits of a genuine willingness and/or desire to explore alternatives seriously, the following outcomes of investigations have resulted for or are under serious consideration by forward thinking organisations:

(i) Instead of a private company and a local authority continuing to oppose each other's landfill proposals in the same general locality, they put aside their differences and formed a Joint Venture to operate a common landfill, to the anticipated benefit of everyone involved.

(ii) By considering an existing landfill as a resource, developing environmentally sound management proposals, and purchasing two adjacent properties, a Council was able to increase available disposal capacity by over 1,000%. Other similar examples exist around New Zealand.

(iii) Existing tips or landfills in or adjacent to estuaries or the margins of lakes can remain in use, in some circumstances, with suitable upgrading.

(iv) Transporting rubbish over large distances can be the most practicable option. One Community already disposes of its rubbish at a site over 100 km away. This may be no where near the practical limit at which transport becomes a constraint for some communities in the future.

(c) Consultation

While some organisations actively consulted with potentially affected parties about projects before the RMA came into effect, many did not to any significant extent. Regrettably, some historical engineering projects, including waste management projects, are held up as prime examples of how not to go about consultation and this has lead to some criticism of engineers in the past.

The consultation requirements of the RMA should avoid this in future, and rightly so. Any party who could be impacted significantly by a project should reasonably expect to be consulted, and well in advance of finding construction has started next door.

There is no text book on how to ensure effective consultation, there are a few easy rides. However, for such consultation to have a chance of being effective there is a bottom line which includes:

(i) Being open and honest.
(ii) Listening and responding carefully to concerns.
(iii) Taking seriously the views of opponents to

the project, and using their often better local knowledge for the benefit of the project - no one knows land better than the person who has worked it.

(iv) Acknowledging the personal stress caused by such projects to those affected by them.

(v) Working through issues out of court as far as possible.

(vi) Allowing enough time for the process to work.

The requirement to consult is one of the most important effects of the RMA on waste management projects and one which has the greatest potential to minimise the conflict associated with them.

One challenge for engineers generally, and geotechnical engineers in particular, is to learn to write and talk in simple language that non-technical people can understand.

(d) What Consents Are Actually Required Under the Resource Management Act?

What consents are required under RMA for, say, a landfill, are not clearly defined. Most applicants seek advice from their regional council as the consent authority, but different regional councils have different views. The Ministry for the Environment appears to have a different view again.

This uncertainty leaves the potential for legal challenge by a party set on preventing or delaying a project going ahead and is therefore most unsatisfactory. However, in reality, provided the conditions of consent cover the matters that need to be covered, it appears unnecessary to have consents for every small activity and preferable to have the smallest number of consents that will allow the required activities to be undertaken.

It is the authors' view that the following consents are required for an operating landfill:

- to discharge contaminants onto or into land
- to discharge contaminants or water into water
- to discharge contaminants into air
- to discharge stormwater into water
- to take water
- to dam water
- to divert water
- an appropriate land use consent

As far as possible one of each type of consent should be used to cover all activities at the one site. One exception to this is that a separate consent should be obtained for each named tributary into which discharges occur.

For closed landfills the consent to discharge contaminants onto or into land could be replaced by a consent to discharge contaminants to groundwater via seepage. Some guidance is also required as to when a consent to discharge to air is no longer required. For small landfills, tips or dumps a period of five years after closure could be adopted as a practical cut-off point although in practice some discharges to air are likely to occur beyond this time.

A possible grey area in terms of responsibility for consents is who is responsible for placing conditions on the control of landfill gas migration via the ground. It is a regional council matter, a district council matter or both? In practice, it might be both.

(e) Effects on the Environment and Setting Appropriate Consent Conditions

From a geotechnical point of view, the two main effects to address are the effects of leachate loss on natural water and the effects of landfill gas on odour levels. In terms of the leachate issue, it is likely to differ for a new landfill compared to an existing one. In the latter case, you can measure the actual effects and develop systems to overcome them. Often, the effects are far less serious than most people's expectations, with nitrogen being the component that often causes greatest concern. With a new landfill, there is normally a requirement to prove beyond reasonable doubt that unacceptable environmental effects will not occur. Because of the limitations of theory and our overall understanding of all the processes that affect the performance of soils as a containment or attenuation medium, we often opt for a more conservative design that actual experiences in New Zealand suggests is necessary.

In assessing any new landfill site, it is desirable to consider risk from the outset of any site selection and/or assessment process, and try to minimise the risks as far as possible. This is effectively a requirement of the Resource Management Act through Section 17 - the duty to "avoid" adverse effects and Section 1(e) of the Fourth Schedule, which requires a risk assessment where any activity includes the use of hazardous substances and installations. The likely presence of landfill gas and the presence of household hazardous wastes are two reasons why a landfill comes into the category of requiring a risk assessment.

Effects on water quality will normally be measured in terms of USEPA criteria for the protection of aquatic life or the ANZECC guidelines. While some effects are measured in terms of stockwatering or drinking water requirements, these are becoming less frequent. Thus, we are seeing a gradual tightening of standards. In some cases this has reached the point where the need for such stringent standards is being questioned. One way of addressing this appears to be the use of

"Whole Effluent Toxicity" or WET testing where bioassay techniques are used to determine the actual effects of the discharge. Some results from overseas suggest that USEPA criteria might be able to be relaxed substantially without any significant adverse effects on the environment. This should result in less onerous, and hence less expensive, new landfill designs.

The containment requirements for landfill gas in the future could become onerous if odour controls currently being talked about by some regulators are adopted. The design of the cap may become critical, possibly requiring the use of a synthetic component. Based on current knowledge, the authors consider this would be highly undesirable, and thus see the issue of odour control conditions as being of considerable importance to waste management in New Zealand. While high standards of gas control are clearly justified in some cases, the economic impact of the proposed standards must be balanced against the actual effects of periodic exceedance of those standards.

(f) Requirement to Produce a Management Plan

While there is no specific requirement of the Resource Management Act to produce a landfill management plan, it seems to be more and more the accepted thing to do and has been made a requirement by the Planning Tribunal on some recent projects. While it was originally acceptable to the Tribunal to produce the plan after consents were granted, a recent case required the plan to be finalised before a decision was given.

This is consistent with the overall trend towards better understanding of the whole project before consents will be granted. A related trend is the use of peer review processes to check the investigation, design, construction and operation of larger landfills.

(g) Other Considerations

There are a wide range of other aspects of the Act that remain to be fully understood. These include:

- Are environmental trade-offs acceptable and/or practicable?
- How much emphasis can be placed on economic considerations?
- What power does the Act provide to minimise wastes, taking into account the requirement to avoid, remedy or mitigate adverse effects?

In conclusion, it will be a very long time before we fully understand all aspects of the Resource Management Act, but if used wisely, it does provide an excellent framework within which to develop sound waste management strategies for the future.

REMEDIATION OF GROUNDWATER CONTAMINATED WITH CHROMIUM WASTE

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A major U.S. industrial corporation conducted wood treatment operations related to cooling tower manufacturing at their former facility located in Northern California from 1965 until 1984. The wood treating process involved the preparation and use of chromated copper arsenate (CCA) and acid copper chromate (ACC solutions. Site assessment activities conducted at the site beginning in 1982 revealed the presence of elevated concentrations of chromium in the soil and groundwater beneath the site. The major operational source areas appear to have been several aboveground and underground storage tanks, the pressure treating vessel, and treated-wood drying and storage areas. Total dissolved chromium (Cr) concentrations in groundwater have been detected up to 200 milligrams per litre (mg/l); hexavalent chromium (Cr^{6+}) constitutes the majority of the dissolved concentrations, having been detected up to 150mg/l. The regulatory established clean up objective is 0.05 mg/l total dissolved chromium.

The results of aquifer testing indicated extraction wells were capable of producing a maximum of approximately 20 gpm per well. Model results indicated the aggregate capture zone of three shallow and one intermediate depth extraction wells would be adequate to address the chromium impacted portions of the shallow and intermediate zones. However, the capture zone extended onto the upgradient property where organic compounds were detected. To solve this problem, upgradient groundwater injection (between the chromium plume and the organic compound impacted area) was modelled in several scenarios. The scenario selected as the basis for remedial well field design optimised capture of the chromium impacted zone, and produced minimal hydraulic effects on adjacent properties. In this scenario, groundwater extraction is performed using three shallow zone extraction wells located approximately along the plume axis and one intermediate zone extraction well near the centre of the intermediate zone plume. Treated effluent is re-injected through three shallow zone and one intermediate zone injection wells located upgradient of the chromium plume. Flow rates necessary to maintain capture on the plume averaged approximately 10 gpm for each extraction and injection well. Hydraulic mounding upgradient of the injection wells was minimised by increasing the flow rate of the extraction wells closest to the injection wells to 15 gpm, and decreasing the flowrate of the most downgradient extraction well to 5 gpm. The designed net flow through the system was 40 gpm.

THE CONDITIONING AND COMPACTION OF SCHIST TO FORM LOW PERMEABILITY FILL

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SYNOPSIS

Many people involved in dam design are unaware of the wide range of materials that can be suitable for low permeability zones. This paper presents the example of dams associated with the Macraes Flat Gold Mine which are constructed largely from schist rock.

INTRODUCTION

Water and tailings storage dams at the Macraes Flat Gold Mine in Otago are designed as zoned earth/rockfill embankments with low permeability zones constructed largely from weathered schist. Many engineers dealing with dams have considered that clay was the best material for the low permeability (impervious) core of a dam. There are examples where clay has been hauled over great distances and at considerable cost (1). Clay was considered superior to cohesionless low permeability material because it is more plastic and able to withstand greater deformations without cracking. In addition, clay has a higher resistance to erosion, which is particularly important if concentrated leaks develop. However, Sherard (1) points out that with appropriate filter design, the "preference for clay over cohesionless impervious soils is not considered valid. The two materials act differently in the presence of a concentrated leak in a crack. Each has theoretical advantages and disadvantages, and it is not clear that either is superior". Sherard concludes that "the principal job of the core is to be impervious. Prevention of piping is the job of the filter".

Major dams have been constructed with impervious cores consisting of cohesionless silty sands, sandy silts and gravelly and sandy silts. These materials have been derived from decomposed granites, schists, glacial moraines and weathered sandstones. The example of water retaining and tailings dams associated with the Macraes Flat Gold Mine and constructed from schist rock is presented in this paper.

PROJECT DESCRIPTION

Macraes Flat Gold Mine is located at an elevation of about 500 m above sea level approximately 100 km north-west of Dunedin. The ore is located in a shear zone consisting of semi-pelitic and semi-psammitic schist. Deep (up to 75 m) gullies have been eroded in the schist. Slopes are mantled by relatively thin layers of loess, colluvium and solifluction material. The available source of fine grained low permeability zones is limited to these materials and slightly to moderately weathered schist from within the mine.

The project includes a number of large dams for storage of tailings, water and silt retention. The two tailings dams will be 78 m and 135 m high when complete. The two water storage dams are approximately 18 m and 35 m high. The two largest silt dams are approximately 10 m and 14 m high.

During the initial design, it was clear that the quantities of loess and colluvium were inadequate in terms of the volume of low permeability material required for the various dams. Also these materials were the only source of soil suitable for rehabilitation of the dams and waste rock stacks formed to dispose of excess material from mining. The designers therefore focused on the problems of processing the weathered schist into a well graded fill material suitable for construction of the low permeability zones.

Large quantities of material are required to construct the low permeability zones in the embankments. Weathered schist from the pit is available at very low cost, the only additional cost above transporting it is that of conditioning, watering and compacting the material. As a consequence, low permeability zones can be constructed with material from the pit for a cost of approximately 10-15% of that for borrowed material.

SITE INVESTIGATION AND TESTING DURING DESIGN

Initial design of the tailings dams associated with the Macraes Flat project was based on the assumption that cohesive materials (silts, clays) would be required for the low permeability (impervious) core zones. Investigations were undertaken to identify such materials both on and off the site. It soon became apparent that such materials were available in only limited quantities and that an alternative approach was required. The possibility of using partially weathered schist, which is the basement bedrock in the site area, was then investigated. Such material, when compacted, has limited fines content and is essentially a cohesionless material. Similar materials have been used in canal and road construction in Central Otago and some information on these projects was provided by Works Consultancy. This information, together with a review of the use of materials in an overseas dam (2) and the positive comments by Sherard (1) on the use of cohesionless materials, gave encouragement that such material had the potential for use in the dams associated with the Macraes Flat Project. To confirm the suitability of such material, an investigation programme consisting of field compaction trials and large diameter (225 mm) triaxial tests was instigated.

In 1989, material from close to the outcrop of the ore body was excavated and compacted in test strips using the largest type of locally available plant. Four test strips were formed at this time with weathered schist used in two (TS1 & TS2) while fresher rock was used in the other two (TS3 & TS4). TS1 was compacted with six passes of a 6 tonne sheepsfoot roller and TS2 was compacted with six passes of a 10 tonne vibrating roller. The material in the test strips was spread in loose lift heights of 250 mm and the total thickness of each compacted test strip was approximately 1.5 m. Large *in situ* density tests (1.65 m diameter water replacement) and permeability tests were conducted to assess the performance. The results of the compaction trials are summarized in Table 1. Permeabilities in the range of 1.2 to 6.5 x 10^{-7} m/s were determined with a dry density of 2.06 t/m^3 for TS1 and 2.17 t/m^3 for TS2. Particle size analyses of samples from both test strips were measured and are shown on Figure 1. In both cases, 8% was finer than 75 microns.

Samples from TS1 and TS2 were also subject to large diameter (225 mm) triaxial tests at Works Consultancy Central Labs. These tests were to determine permeability and static and dynamic strength properties. The permeability results are summarized in Table 2. Permeability values measured during these tests ranged between 0.6 and 1.6 x 10^{-8} m/s. The triaxial tests indicated a non-linear shear strength envelope which could be described by the following equation,

$$\tau = 2.43\, \sigma^{0.83}$$

where τ = shear strength (kPa)
σ = effective confining pressure (kPa)

In addition to measuring the density achieved in the trial strips, compaction tests on samples of schist and schist/loess blends were conducted using both New Zealand Heavy and Vibrating Hammer test methods (NZS 4402:1986 Tests 4.1.2 and 4.1.3). Representative results are shown on Figure 2 together with a number of *in situ density* values measured in compaction trials and subsequently during construction.

DESIGN ASSUMPTIONS AND EARTHWORKS SPECIFICATION

Based on the results of the field compaction trials and triaxial tests, a permeability value of 1 x 10^{-7} m/s was adopted for design purposes. Review of the test results and results of permeability tests on other cohesionless materials indicated that the key to obtaining low permeabilities was a well graded material with a fines content (75 micron) of about 8% combined with heavy compaction to achieve a high density. These factors were reflected in the Specification that was adopted. The Specification also recognized that permeability cannot be readily measured in the field. Permeability is controlled indirectly by specifying a well graded material with the minus 75 micron size lying between 9% and 25%, a minimum dry density of 2.15 t/m^3 and a minimum water content of 6.5%. *In situ* permeability tests are specified to confirm that the indirect controls are appropriate. In addition, minimum lift heights (200 mm) and six passes of compaction plant are required. The use of a large self-propelled sheepsfoot compactor (30 tonne static weight) to provide the maximum amount of particle breakdown of the schist rock and to provide some compaction is also specified.

In 1990 additional compaction trials were undertaken during earthworks for the ore stockpile area (ROM Pad). This was prior to construction of the dams. The results of these trials are presented in Table 1. They provided confirmation of the specified requirements. They also allowed development of fill testing techniques with a nuclear densometer including initial correlations with direct testing methods (water replacement for bulk density and oven measurements of water content).

EMBANKMENT CONSTRUCTION

The practical difficulties of manufacturing a low permeability material from mine waste rock was not simply resolved. In the initial stages of construction in 1990, all site staff (Mining Company as well as Contractor), went through a major learning phase. To successfully achieve a low permeability fill required,

* selection of sufficiently weathered or lower strength waste rock that would, when worked with standard construction equipment, break down to achieve the design objectives, (i.e. grading, density and permeability)

* the correct material handling and compaction techniques (i.e. the appropriate type of plant, number of passes, techniques for placement of fill to avoid segregation)

* development of appropriate fill control methods.

In spite of the difficulties involved the Contractor soon established a suitable modus operandi which usually results in an acceptable standard of construction. The rock is brought to the embankments in large dump trucks and tipped in heaps. It is then spread by a bulldozer into an approximately 200 mm thick layer. A Cat 825 Compactor is used to roll and crush the material to form the required grading before water is added to bring it to the optimum water content. Finally, the conditioned material is rolled with a loaded dump truck (135-210 tonnes gross weight and 500-600 kPa tyre pressure).

Vibrating drum rollers have been used at various stages as an alternative to the use of loaded dump trucks. However in the ongoing construction of the tailings dams, they are not now used because:

* full compaction is not achieved up to the surface and to measure representative densities it is necessary to excavate below the current working surface

* it is easy to utilize the loaded dump trucks for compaction because of the large open working area and the fact that they can traffic across the low permeability zones on their way to other areas.

CONTROL TESTING

The density of each layer is measured using a nuclear densometer. Such measurements are indirect and so densities are also measured regularly using a water replacement method. This enables correlations to be developed between the direct (water replacement) and indirect (nuclear) tests. These correlations are constantly reviewed and modified if changes are detected. The latest correlations indicate that nuclear determined bulk densities need to be increased by 76 kg/m^3 and that the adjustment for water content is dependent on actual water content. Lower values of nuclear water content need to be increased and high values decreased. To obtain the *in situ* dry density it is necessary to use the corrected bulk density and water content values. Particle size analyses are also undertaken regularly. Layers that do not meet the specified requirements are removed or reworked before being retested.

In situ permeability tests are conducted whenever operations permit using a 300 mm standpipe. Their frequency was quite low in the initial months, because it was necessary to continue the embankment building operation whenever weather permitted. The results of the tests that have been carried out are normally below the specified value and are summarized in Table 3.

From time to time, variations in the nature of the schist have resulted in the compacted densities falling below the specified level. Generally, this has been associated with material containing larger quantities of fines. On occasion this has resulted in a suggestion that the Specification be relaxed. However, the Designers were concerned that this could lead to the acceptance of material with lesser fines which would not meet the permeability criterion ($< 10^{-7}$ m/s). The recommendation in this case has been to mix the more weathered material with less weathered material to achieve a grading with which it was possible to meet the specified density.

TAILINGS EMBANKMENTS

The tailings embankments are zoned structures with a low permeability zone on the upstream shoulder. The construction of the embankments is continuous and is maintained at a rate sufficient to provide the required capacity to store the tailings and the design storm. Water ponding on the surface of the tailings is returned to the Process Plant for re-use.

In the initial years there was ample weathered waste rock available from the main pit to construct the low permeability zones, but by 1991 this was almost exhausted and a second pit was opened up to augment the supply. Material from this pit was more highly weathered and broke down readily to form the required grading, but when overworked resulted in greater fines and lower dry densities than specified. Continued compaction did not improve the density and as a consequence the Contractor had to reconstruct sections of fill. Because the permeability and strength are sensitive to changes in the density and grading, a reduction in the specified density was resisted. Instead a review of the construction procedures was carried out to determine whether changes to either the procedures or the Specification were warranted.

A series of trials were carried out on one section of fill using fine graded weathered schist over one half and fresher (grey) rock in the remainder. Two types of compactive effort were evaluated. The material was conditioned spread and initially compacted in the normal way with the 825 Compactor. Subsequent compaction was with either a loaded dump truck (Cat 777) or vibrating drum roller (V24T). *In situ* density measurements were made with the nuclear densometer with two measurements made at each position.

For the first trial, the normal method of compaction involving four passes with a loaded dump truck was used. Five density measurements were made. The section was then rolled with four passes of a Vibrating Roller and the density measurements repeated. The section was then rolled with a further four passes of the vibrating roller and the density measured. The results of this trial are presented in Table 4. The results indicated that following compaction with the loaded dump truck, the vibrating drum roller had negligible effect on the density of the finer moderately weathered (brown) schist with the density apparently decreasing after four passes of the V24T. Some shearing of the surficial layer was also noted, which is not uncommon for vibrating rollers. A small increase in density was noted for the coarser slightly weathered grey schist following four passes of the V24T but then the density apparently decreased following eight passes. In all cases, the density of the finer, moderately weathered schist was less than for the coarser, slightly weathered schist.

The second trial was carried out on a strip of fill adjacent to the first. In this case the vibrating roller was applied immediately after compaction and conditioning with a self-propelled sheepsfoot roller had been completed. In the first instance, two passes of the V24T were applied and the density measured. Then a further two passes were applied and the density remeasured. Finally, the area was rolled with four passes of a laden dump truck. The results presented in Table 5, were also rather inconclusive. An increase in average density was noted following four passes of the V24T but this conclusion may be biased by the two low readings (Tests 1 and 2) following the first two passes. One of these tests (Test 1) was in an area of finer weathered (brown) schist. Additional compaction with the loaded dump truck had no apparent influence on the measured density. In fact a small drop in average density was noted.

In the third trial, all compaction was carried out using the loaded dump truck. Initially, four passes were applied and the density at four marked positions were measured. Then a further four passes were applied and the density at the same four positions was measured. Finally, a further four passes were applied to give a total of twelve passes. The results of this trial is presented in Table 6. The results of this trial showed that no improvement in the density occurred after the first four passes of the loaded dump truck. The results were consistent with that observed in the first trial on the finer moderately weathered schist following compaction with four passes of a loaded dump truck. The results were also comparable with that achieved by four passes of the vibrating drum roller (V24T) on similar material (refer to Table 5).

In conclusion, the trials demonstrated that:

* similar levels of compaction could be achieved by four passes of a vibrating drum roller (V24T) or a loaded dump truck (Cat 777)

* provided the material was conditioned and compacted by the 825C then little improvement in density was achieved following four passes of either the V24T or Cat 777

* higher densities could be achieved for material containing a lower quantity of fines (i.e. higher densities were achieved with the slightly weathered schist compared to the moderately weathered schist)

As a result of the trials it was decided that modification to the Specification was not required. Rather an improvement in the density could be achieved if more attention was paid to blending the more weathered (brown) schist with the less weathered schist.

LONE PINE WATER SUPPLY DAM

In 1991, the construction of a new 35 m high water supply dam (called the Lone Pine Dam) commenced. The design of this dam was similar to other water storage dams on the site with a low permeability central core. Due to the distance of this dam from the mine, and to ensure that a low permeability was achieved, it was decided to use a schist/loess mixture, borrowed from an adjacent area, to form the low permeability zone.

A series of trials were carried out to establish the optimum quantity of loess to mix with the schist and to determine the type of construction plant to be used. A mixture of 60% schist and 40% loess was adopted based partly on experience and supported by laboratory compaction tests. A test section was constructed and tested to determine dry density, water content, particle size distribution and permeability.

The mean dry density, permeability and fines values measured on the test section were 2.0 t/m^3, 1.5×10^{-8} m/s and 18% passing 75 microns respectively. Conditioning and compaction in this case involved 10 passes with the 825 Compactor to break the rock down, 8 passes with the water cart and 4 passes of a V24T Vibrating Roller. For this dam the fill material was conditioned on the borrow area then transported by a motor scraper and spread on the embankment.

The above procedure was continued until the source of partially weathered schist was exhausted and it was decided to complete the embankment using weathered waste rock from the mine.

CONCLUSIONS

1. Low permeability fill ($<10^{-7}$ m/s) can be manufactured, at low cost, from slightly to moderately weathered schist from the mine.

2. Careful selection, blending and conditioning is necessary.

3. Maximum density can be achieved, following conditioning with four passes of a heavy vibrating roller or loaded dump truck where the material is spread in loose layers of 200 mm.

ACKNOWLEDGEMENTS

The authors would like to acknowledge Macraes Mining Co. Ltd for permission to publish information pertaining to the Macraes Flat Gold Mine Project.

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TABLE 1: Summary of Compaction Trial Results

Date	Location	Water Content (%)	Dry Density (t/m^3)	% Finer than 75 μm (%)	Permeability (m/s)
1989	TS1	9.4	2.06	8	$2.4 - 6.5 \times 10^{-7}$
1989	TS2	8.3	2.17	8	$1.2 - 5.2 \times 10^{-7}$
1990	ROM Pad	6-7.9	2.09-2.13	17 approx.	7.1×10^{-8}
1990	ROM Pad	12.5	2.007	17 approx.	1.2×10^{-8}

Notes: 1. The range in permeability results for TS1 and TS2 are a result of different interpretations of the test data.

2. The range in water content and dry density values for the first ROM Pad result represent the range in values measured from small water replacement tests.

3. Permeability tests were conducted using a 300 mm diameter apparatus. Tests were by falling head except for the first ROM Pad result which was a constant head test.

TABLE 2: Summary of Large Diameter (225 mm) Triaxial Test Results

Date	Sample Description	Water Content %	Dry Density (t/m^3)	% Finer than 75 μm (%)	Effective Confining Pressure (kPa)	Permeability (m/s)
1989	Zone A	6.9	2.22	14	150 300 500	1.60×10^{-8} 0.92×10^{-8} 0.61×10^{-8}
	Zone A	7.9	2.18	14	150	0.80×10^{-8}
	Zone C	2.3	1.96	11	150 300 500	6.10×10^{-6} 3.90×10^{-6} 2.40×10^{-6}
1990	Zone A	6.2	2.11	21	150 300	4.90×10^{-8} 1.50×10^{-8}

Notes: 1. Permeability values were measured following consolidaiton at the effective confining pressure stated. The ratio of the applied head to effective confining pressure was 0.2.

2. Zone A samples were compacted using NZ heavy compaction with the sample close to optimum moisture content. The Zone C sample, representative of non-structural fill, was compacted at natural moisture content to a target dry density considered likely to be representative of field conditions.

3. All samples were screened to remove material greater than 37.5 mm prior to compaction, but the grading less than 4.75 mm was adjusted to be similar to that in compacted 'unscalped' samples.

TABLE 3: Summary of *In situ* Permeability Tests

Year	Water Content (%)	Dry Density (t/m^3)	Permeability (m/s)
1990	9.7	2.13	5.3×10^{-8}
1990	8.6	2.17	4.7×10^{-8}
1991	8.5	2.13	7.5×10^{-9}
1991	10.6	2.02	1.8×10^{-8}
1993	6.3	2.170	3.97×10^{-8}
1993	9.2	2.297	5.96×10^{-8}
1993	7.4	2.290	4.50×10^{-8}
1994	7.1	2.309	4.20×10^{-8}
1994	7.9	2.162	8.64×10^{-8}
1994	9.0	2.164	3.75×10^{-8}

Notes: 1. These tests were conducted using a 300 mm diameter falling head test method.

2. Dry densities are based on bulk densities measured using a water replacement method and water content determined by oven drying.

3. Tests in 1990 and 1991 were conducted using a concrete standpipe, but this changed to a steel standpipe in 1993 because it was possible to achieve greater penetration into the fill with less problems of leakage.

TABLE 4: Truck Rolling Followed by Compaction with Vibrating Roller

Compaction Method	Test	Dry Density kg/m^3	Water Content %	Material Type
Four passes of loaded 777 Dump Truck	a	2093	6.3	Fine graded, pale brown weathered schist
	b	2043	6.3	
	c	2073		
	d	2102	4.1	Transition (i.e. mixed)
	e	2116	5.3	Coarse graded grey schist
Four Passes of loaded 777 Dump Truck followed by four passes V24T Vibrating Roller	a	2039	6.5	As for a-c
	b (approx)	2134	6.3	Similar d
	-	2174	4.1	Coarse graded grey schist
	-	2191	6.4	Coarse graded grey schist
Four passes of loaded 777 Dump Truck followed by eight passes of V24T	-	2135	5.3	Coarse graded grey schist
	-	2131	5.0	Coarse graded grey schist
	-	2104	6.1	As for a-c

Notes: Dry density and water content values are values recorded by nuclear densometer and have not been corrected.

TABLE 5: Compaction with a Vibrating Roller

Compaction Method	Test	Dry Density kg/m^3	Water Content %	Material Type
Conditioning with 825C followed by two passes V24T	1	1953	6.3	Fine graded pale brown weathered schist
	2	1948	5.9	
	3	2070	7.7	-
	4	2054	6.1	
	5	2072	6.7	
	Mean	2019	6.5	
Conditioning with 825 but with additional two passes V24T (four total)	6	2049	8.8	-
	7	2081	6.6	
	8	2165	6.2	Coarse graded grey schist
	9	2092	4.9	Coarse graded grey schist
	Mean	2097	6.6	
Area of above tests rerolled with four passes of loaded Dump Truck	a	2024	6.8	
	b	2027	6.1	
	c	2021	8.6	
	d	2064	6.0	
	e	2053	7.9	
	f	2061	9.3	
	g	2024	8.2	
	h	2181	6.0	
	i	2157	5.2	
	Mean	2068	7.1	

Notes: Dry density and water content values are values recorded by nuclear densometer and have not been corrected.

TABLE 6: Repeated Rolling with Loaded Dump Truck

Compaction Method	Test	Dry Density kg/m^3	Water Content %	Material Type
Conditioning with 825C followed by four passes of loaded 777 Dump Truck	a	2067 2028	6.1 6.1	Fine graded pale brown moderately weathered schist
	b	2097 2058	5.0 5.7	
	c	2051 2086	6.3 5.6	
	d	2099	5.1	
	Mean	2069	5.7	
	S.D.	24.2	0.46	
Conditioning with 825C followed by four additional passes of loaded 777 Dump Truck (eight total)	a	2059 2100	6.6 5.5	
	b	2083 2039	4.7 5.3	
	c	2046 2058	5.8 5.5	
	d	2027	5.7	
	Mean	2058	5.6	
	S.D.	23.5	0.530	
Conditioning with 825C followed with four additional passes of loaded 777 Dump Truck (twelve total)	a	2070 2086	6.6 5.6	
	b	2098 2082	5.4 5.6	
	c	2072 2078	6.1 5.5	
	d	2118 2078	5.0 5.6	
	Mean	2078	5.7	
	S.D.	26.9	0.449	

Note: Dry density and water content values are values recorded by nuclear densometer and have not been corrected.

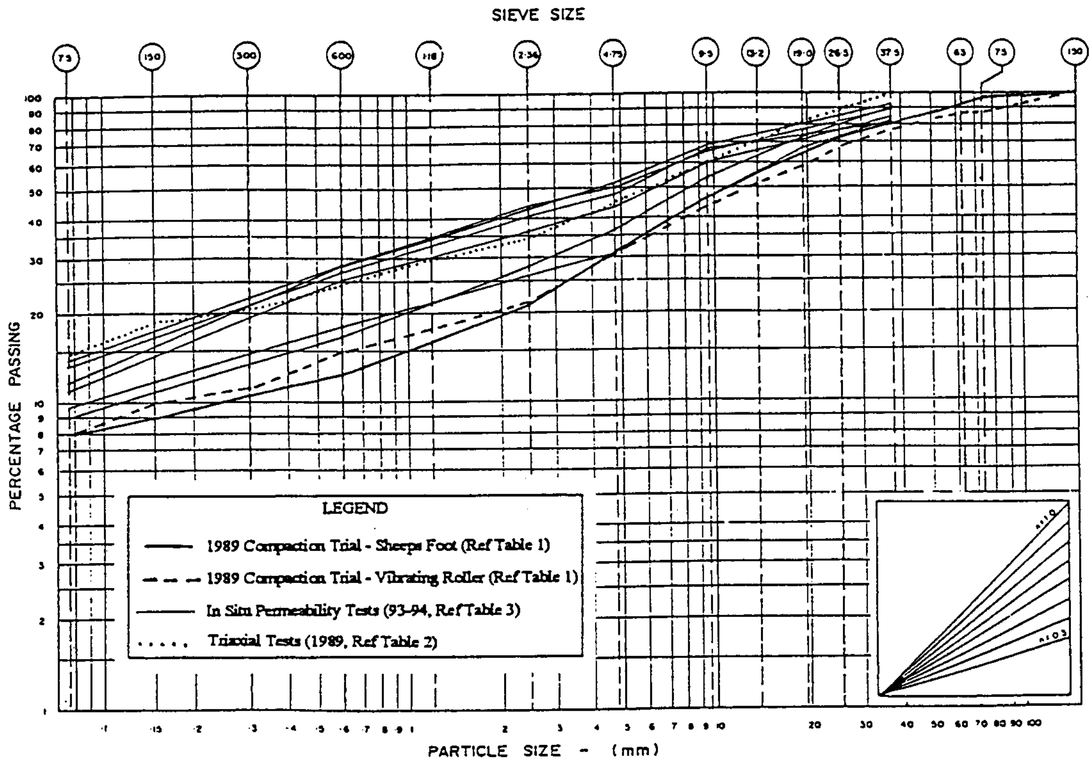

Figure 1 Particle Size Distributions

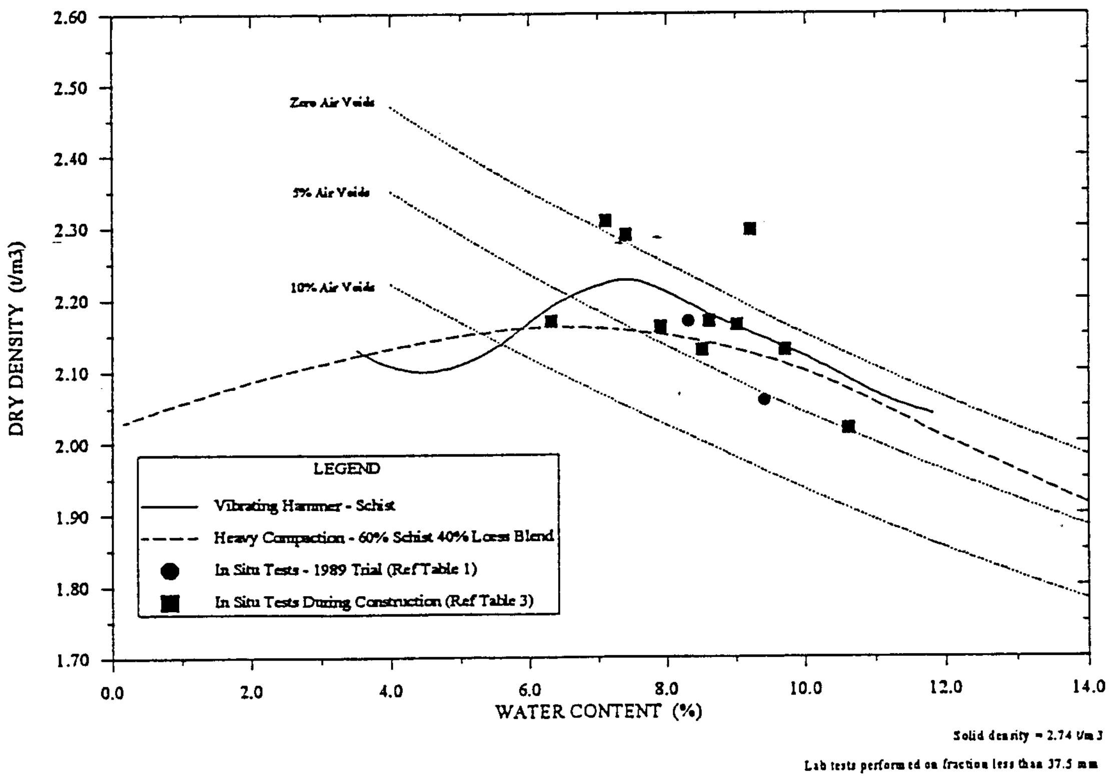

Figure 2 Compaction Test and In Situ Density Test Results

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Allan McKerchar	Palmerston North City Council	PALMERSTON NORTH

REGISTRATION LIST "GEOTECHNICAL ASPECTS OF WASTE MANAGEMENT" WELLINGTON 1994

NAME	COMPANY	CITY
Tim Mills	Ruapheu District Council	TAUMARUNUI
Philip Milne	Simpson Grierson Butler White	WELLINGTON
Chris Mitchell	Phillips Fox	WELLINGTON
Adrian Mitchell	Wellington City Council	WELLINGTON
Mark Mitchell	Geocon Testing & Equipment	HAMILTON
Vicki Moon	University of Waikato	HAMILTON
Murray Mulholland	Environment Waikato	HAMILTON
Nithyananthar Nadesananthan	Rodney District Council	AUCKLAND
Aidan Nelson	Earthtech Consulting Ltd	AUCKLAND
Colin Newton	Works Consultancy Services Ltd	WELLINGTON
Ross Nicholson	South Tamaki District Council	HAWERA
David Nobes	University of Canterbury	CHRISTCHURCH
John Norrie	Cheal Hindess Battersby & Norrie	TAUPO
Marianne O'Halloran	Works Civil Construction	HAMILTON
Jane Orsman	Wellington Regional Council	WELLINGTON
Shona Page	Works Consultancy Services Ltd	LOWER HUTT
Stuart Palmer	Beca Carter Hollings & Ferner Ltd	WELLINGTON
John Palmer	Hutt City Council	LOWER HUTT
Brian Paterson	Paterson & Coates Associates	CHRISTCHURCH
Mark Plested	Worley Consultants	AUCKLAND
Graeme Proffitt	Pattle Delamore Partners	WELLINGTON
Laurie Richards	Lincoln University	CANTERBURY
Bruce Riddolls	Riddolls Consultants Ltd	QUEENSTOWN
Grant Roberts	Groundsearch EES Ltd	AUCKLAND
Bob Roche	Russell McVeagh Mckenzie Bartleet & Co	WELLINGTON
Ian Rowden	Royds Consulting	PALMERSTON NORTH
Jeff Ruddock	Waihi Gold Mining Co Ltd	WAIHI
Wayne Russell	Woodward-Clyde (NZ) Ltd	AUCKLAND
Ray Ryan	N Z Rail Ltd	WELLINGTON
Angela Sheffield	Wellington Regional Council	MASTERTON
Patrick Shorten	Fraser Thomas Ltd	AUCKLAND
Timothy Sinclair	Tonkin & Taylor Ltd	AUCKLAND
Gary Smith	Tonkin & Taylor Ltd	WELLINGTON
David Smith	Civil & Environmental Engineer	AUCKLAND
Fred Smits	National Institute of Water & Atmospheric	WELLINGTON
Guy Sowry	Otago Regional Council	DUNEDIN
John St George	University of Auckland	AUCKLAND
Malcolm Stapleton	Babbage Consultants Ltd	AUCKLAND
David Stewart	Institute of Geological & Nuclear Sciences	DUNEDIN
Andrew Stiles	Works Consultancy Services Ltd	WELLINGTON
Ron Stroud	Beca Carter Hollings & Ferner Ltd	WELLINGTON
Joseph Thomas	Tasman District Council	RICHMOND
Carolyn Thomas-Lewis	Wellington Regional Council	WELLINGTON
Peter Thornton	Nelson City Council	NELSON
Don Turley	Bell Gully Buddle Weir	WELLINGTON
Scott Vaughan	Riley Consultants Ltd	AUCKLAND
Graham Wallace	Tonkin & Taylor Ltd	WELLINGTON
Andrew Watson	Bisleys Pump Industries	CHRISTCHURCH
Allan Watson	Christchurch City Council	CHRISTCHURCH
Jeanette Watson	Rudd Watts & Stone	WELLINGTON
Laurie Wesley	University of Auckland	AUCKLAND
Peter White	Truebridge Callender Beach	WELLINGTON
John White	Manukau City Council	AUCKLAND
Robert Wigmore	Caltex Oil (NZ) Ltd	WELLINGTON
Paul Williams	Kevin Kelly & Assoc	AUCKLAND
Philip Williams	Harrison Grierson Consultants Ltd	AUCKLAND