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N.Z. GEOMECHANICS NEWS

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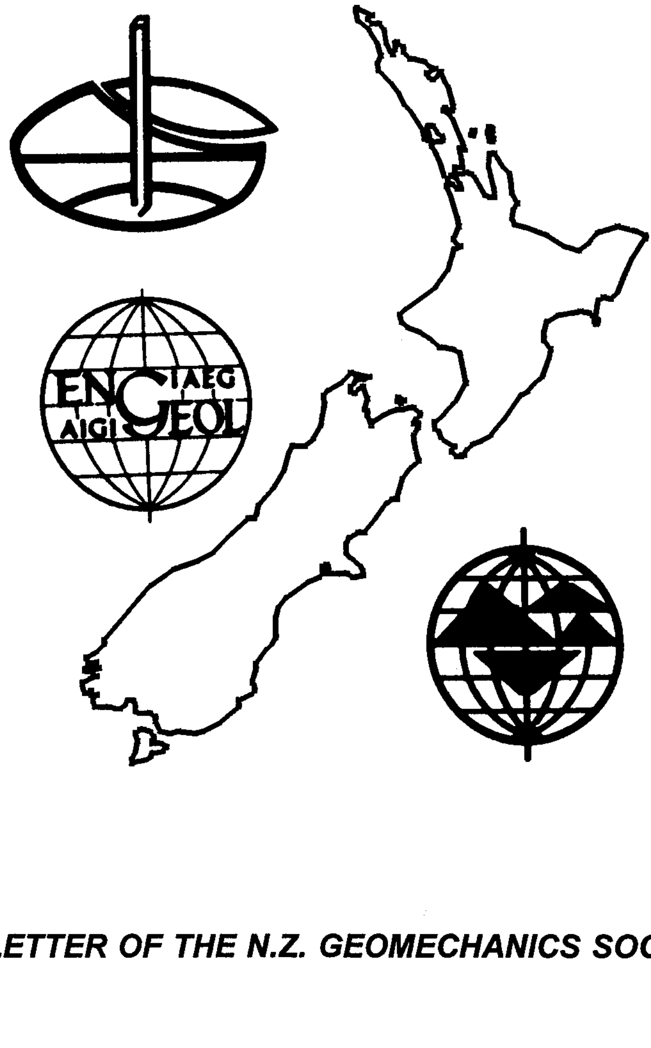

A NEWSLETTER OF THE N.Z. GEOMECHANICS SOCIETY

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CONTENTS

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.
  - news of current projects

Submission of text material in camera-ready format is not necessary. However, typed copy is encouraged particularly if on floppy disk. 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

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 rates are given on the information pages at the rear of this issue and are supplemented according to which of the international societies, (namely Soil Mechanics, Rock Mechanics or Engineering Geology) the member wishes to be affiliated. Members of the Society are required to affiliate to at least one International Society.

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TECHNICAL PAPERS IN THIS ISSUE

This issue contains papers written by members and presented at IPENZ conferences and other forums. The standard of these papers and others published in recent issues is high. Some of the papers have been published elsewhere but are reprinted in Geomechanics News for the benefit of members who would not otherwise see them. Please remember that Geomechanics News is a magazine for Society members and papers are not necessarily refereed.

It pleasing to see the quality of geotechnical research and application to engineering problems that these papers reflect and members are encouraged to maintain this level of publication. We are a small group and can not advance geotechnical and geological practice in NZ unless we share our experience with the profession. This is particularly important as the Society seeks to develop "Residential Development Guidelines" (refer to the 'Chairman's Corner'). Any form of paper or technical note is welcome, especially brief case histories that present a particular geotechnical facet.

STABILITY ASSESSMENTS

Thank you for those who commented on the results of the stability assessments survey questionnaire as presented in our last issue. We would welcome further comment from members and others who are affected by the assessment and development on marginal sites. A suggestion is for NZGS local branches to hold a local "slope stability forum" amongst consultants, developers, territorial land authority officers/engineers/planners and other concerned parties to raise issues or discussion items prior to the next issue of NZ Geomechanics News (in October/November 1995). Feedback from these forums will be most useful in compiling a set of guidelines which will be presented at the 1996 NZGS Symposium (for details of the symposium refer to the information presented later in this issue).

CURRENT GEOTECHNICAL PROFESSIONALISM

The Australian Geomechanics Society is currently preparing their next issue of AGS newsletter journal. The theme is current developments within the geotechnical professions. It is timely then that a similar review be carried out in NZ and a combined collation of the results be prepared. A brief questionnaire will be distributed to NZGS members in the next two months to gather information on topics such as:

- current geotechnical professional standards
- respective roles of designers, peer reviewers and council officers
- frequency and effects of litigation on consulting practitioners
- the changing structure of consulting companies
- changes to clients' expectations of consultants
- operational effects of recent health and safety legislation
- recent trends in consultants working in overseas areas

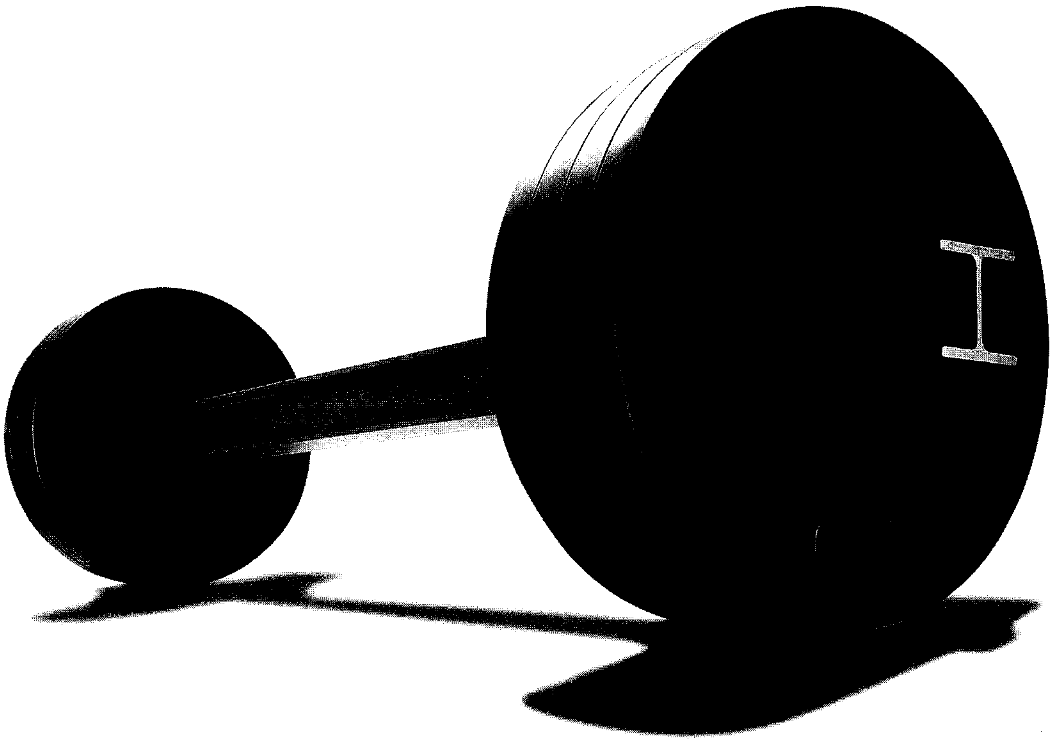
All responses will be treated in confidence as required by those who reply to the questionnaire.

Geoffrey Farquhar & Stephen Crawford
EDITORS

THANK YOU TO RETIRING NZGS MANAGEMENT COMMITTEE MEMBERS

On behalf of the Society members, Geomechanics News wishes to thank Trevor Matuschka for his service to the Society over the last 6 years (including the last 4 years as Secretary). Trevor stepped down as Secretary at the last AGM. The job involves many hours of work and is vital to the smooth running of the Society.

Our thanks also to Fred Smits and Ian MacPherson (who also stepped down in 1994) for their efforts on the NZGS management committee and symposium activities.



THE NEW ZEALAND SOCIETY ON LARGE DAMS

AUSTRALIAN NATIONAL COMMITTEE ON LARGE DAMS

Combined NZSOLD-ANCOLD Two-day Symposium

Christchurch Town Hall
86 Kilmore St, Christchurch, New Zealand

Tuesday and Wednesday 14 - 15 November 1995
plus Optional Study Tours

Dams: The Implications of Ownership

Many countries are now providing legislative measures in the area of dam safety by establishing certain minimum requirements for the design, operation, and maintenance of dams, and dam owners are responding to these changing requirements.

NZSOLD is taking the initiative to use this symposium as a forum for the introduction of their New Zealand Dam Safety Guidelines at a time when legislation for this country is close to being implemented.

We are fortunate to welcome as a keynote speaker Gary Salmon, Director of Dam Safety for BC Hydro, who will also present a paper dealing with the practical experience of dam safety in British Columbia. This, together with input from ANCOLD members on the current status of Australian guidelines and related experience should guarantee interesting and lively debate on a topical issue.

Other papers will focus on technical matters which currently have significant influence on dam engineering. These include consideration of environmental aspects through to recent developments in design and construction, the trends in monitoring and warning systems, and how all of these will ultimately impinge on the assessment of risk from an insurance viewpoint.

Enquiries for further information can be made to:

The Technical Secretary
New Zealand Society on Large Dams
PO Box 12-241
Wellington
New Zealand

Fax: 64 4 473 1296 or Phone: 64 4 471 7132

STABILITY ASSESSMENTS

The Editor's questionnaire on land stability issues in urban development clearly struck a chord with members, and generated a very positive response in the last issue of Geomechanics News. This whole question of factors of safety and their applications to stability analysis of urban slopes is one that should be aired further, and I hope that at the forthcoming Hamilton Symposium on "Geotechnical Issues in Land Development" there can be a structured discussion on at least some of the matters raised. My own experience with residential development of land in the past 20 or so years has been that quantitative stability analysis is often difficult to justify, and that the costs of providing the necessary input parameters for both rock and soil slopes would far exceed any realistic budget in many (if not most) situations.

We presently have legislation which permits residential housing in "hazardous" locations, albeit often with approval in terms of s36(2) of the Building Act, and I am aware of hundreds of houses on landslides where the long-term safety of the occupants is <u>not</u> at risk because the factor of safety is close to 1.00. Whilst as geotechnical professionals we have a responsibility to ensure the safe location and construction of dwellings, I believe that we also have an obligation to resist "recipe-driven" analytical solutions that may not be appropriate to the site or situation. The proposed "Residential Development Guidelines" document that will be presented for discussion in Hamilton will, I hope, allow us to go forward with a New Zealand-wide geotechnical approach to urban planning and development that takes clear account of professional judgement.

Another highly relevant issue in this context is the move by some territorial authorities to implement a listing of specialist geotechnical consultants from whom reports would be accepted as of right. Such individuals would be expected to have sound local knowledge, and demonstrated expertise in either engineering geology or geotechnical engineering. This is seen by many as running counter to accepted practice. It has the added complication that if an individual was to practice widely geographically, many such accreditations would be required and would involve significant time and expense. A further consideration is whether the firm or the individual should be accredited. Again I hope that this issue can be aired constructively at the Hamilton Symposium, and I must add that having served on one such interview panel recently I have a degree of sympathy with the fact that the Council involved was genuinely trying to ensure that they receive competent advice upon which they can rely.

NZGS PROFESSIONAL LISTING

This brings me to another point which I intend to pursue further in the coming year, and that is the question of whether the Society should implement its own professional listing along the lines of the British Geotechnical Society. Allied to this is the possibility of introducing a class of Society membership termed "Fellow" which would recognise demonstrated professional expertise and relevant experience, possibly in conjunction with IPENZ via the Member/Technical Member categories. It may be that an "FGS" would be an acceptable level of professional status in conjunction with a competency listing criterion, and I would certainly be interested in any feedback from members of this matter. I should hasten to add that I do not see such a category of membership replacing or conflicting with the present "Life Member", which should continue to recognise both longevity and service to the Society.

NZGS NAME CHANGE

The final issue I want to raise is that of the possible Society name change. From the polling to date, it would seem that a majority of those responding favour the "New Zealand Geotechnical Society", and I would urge members to make their views known on this matter. As I indicated in my Annual Report the time for actioning such a change would be at the AGM next February in Hamilton, and some lead time is needed to draft the Rule changes. I, personally, am relaxed about the issue, as I can see the merit in the suggested name change but I also know that the Society is still highly regarded with its present name. I have asked the Australian Geomechanics Society to comment on this aspect also, as we have close ties with that organisation and tend to move in tandem with them on most issues.

David H. Bell
CHAIRMAN

REPORT FROM THE SECRETARY

1. MEMBERSHIP

The following new members are welcomed to the society:

A Betts	J S W Fong	I Hendy	T McFarland
P J Millar	L Price	B R White	R K Wilson
A R Wilton	Z Viljevac		

2. MANAGEMENT COMMITTEE

A new management committee was confirmed at the February 13th meeting. The new committee is:

Dave Bell	Chairperson
Colin Newton	Secretary
Dick Beetham	Treasurer
	Vice-Chairperson ISRM
Guy Grocott	Vice-Chairperson IAEG
Brahba Brabhaharan	Vice-Chairperson ISSMFE
Stuart Palmer	National Activities Officer
	Publications Officer
Geoffrey Farquhar	Editor Geomechanics News
Stephen Crawford	Assistant Editor
Mick Pender	Australasian Vice President ISRM
Warwick Prebble	Australasian Vice President IAEG

We welcome the new members to the committee and thank the following who have stood down; Trevor Matuschka, Fred Smits and Ian MacPherson.

3. NAME OF THE SOCIETY

The response to the article in the Geomechanics news showed a majority of those who replied favoured changing the name to New Zealand Geotechnical Society. However, the committee considers further discussion on the issue is required before any action is undertaken. *More information on the results of the questionnaire is contained later in this newsletter.*

4. NZGS 1996 SYMPOSIUM - "GEOTECHNICAL ISSUES IN LAND DEVELOPMENT"

The committee is continuing the preparatory work for this symposium. February 1996 is the preferred date for the symposium. Dave Jennings and Tim Browne have agreed to be the local co-ordinator and is in the process of forming a committee to organise the event. *More information and a call for papers is contained later in this newsletter.*

5. 1996 GEOMECHANICS LECTURE

It is proposed that the next Geomechanics Lecture will be presented at the next ANZ Geomechanics Conference to be held in 1996. The committee is in the process of selecting a suitable person to present the lecture.

6. CONFERENCES

6.1 Second ANZ Young Geotechnical Professionals Conference

Organisation of the Conference is underway. It is planned to hold the conference in Auckland, from 29 November to 2 December 1995 and will be open to young geotechnical professionals up to the age of 35 years of age.

The first young geotechnical professionals conference was very successful with conference attendees finding it extremely stimulating and beneficial. All those who are eligible are encouraged to attend. *More details on this conference are contained later in this newsletter.*

6.2 Seventh ANZ Conference on Geomechanics, Adelaide

Similarly, the organisation of this conference is well underway with a date of the first week in July 1996. As in the past, the conference will be very stimulating and well worth attending. *Further information is presented later in this newsletter.*

Colin Newton
MANAGEMENT SECRETARY

XIVth ICSMFE, HAMBURG 1997

The ISSMFE (through the German Society for Geotechnics) is planning the XIVth International Conference on Soil Mechanics and Foundation Engineering to be held in Hamburg, Germany, (from 6 to 12 September 1997.

The Plenary Sessions will comprise the following subjects:

- Soil Testing & General Property Characterisation
- Recent Developments in Foundation Techniques
- Retaining Structures and Excavated Slopes
- Underground Works in Urban Environment
- Soil Improvement & Reinforcement
- Waste Disposal and Contaminated Sites

The Parallel Sessions will comprise:

- Recent Developments in Laboratory Stress-Strain Testing in Geomaterials
- Ground Property Characterisation by Means of Insitu Tests
- Interplay between Physical and Numerical Models as Applied in Engineering Practice
- Soil Structure Interaction for Shallow Foundations under Static Dynamic Loadings
- Design and Performance of Piled Rafts
- Limit States Concept in Design of Shallow and Deep Foundations
- Design Construction and Performance of Anchored Walls and Strutted Excavations
- Large Excavations with Dewatering in Urban Environment
- Subsidence as Related to Various Tunnelling Techniques
- Performance and Monitoring of Underground Works
- Soil Improvements for Tunnel Works
- Deep in Place Mixing Methods including Jet-Grouting
- Use of Geosynthetics and Geotextiles in Geotechnical Engineering
- Pollutants Containment via Passive Barriers
- Active Pollutants Control and Remediation of Contaminated Sites
- Dredging Sludge and Tailings Impoundments
- Teaching and Education in Geotechnical Engineering

ISSMFE is calling for papers. The following Milestones apply:

- 1 Nov 1995 — Closing Date for submitting Abstracts to the NZ Geomechanics Society and Pre-Registration
- 31 Dec 1995 — Closing Date for Contributions
- 30 June 1997 — Closing Date for Registration

We encourage our members to submit papers and participate in the Conference, where possible. It's time again to think of all the innovative projects you have worked on, and put up some papers to share your experience with colleagues around the world! Please contact me for further information.

ISSMFE is also setting up technical committees to co-ordinate exchange of information and co-operation in various areas on geotechnical engineering. The terms of reference for each of the 14 technical committees are briefly outlined in the February 1994 issue of ISSMFE newsletter. Please contact me for further information.

P. Brabhaharan
VICE CHAIRMAN ISSMFE

AUCKLAND BRANCH

The Great Hanshin Earthquake (Kobe Earthquake)

The Auckland Branch of the Geomechanics Society combined with its counterpart branch of the Structural Engineering Society on 14 March for a presentation by the NZSEE team which visited the earthquake damaged city of Kobe. Professor Bob Park, David Jennings, Charles Clifton, Ian Billings and John Sinclair presented a précis of their geotechnical, structural and architectural observations supported by slides. Details of the observations have been reported in the March 1995 issue of *NZ Engineering*. Of particular interest to Society members was the concentration of damage along a distinct zone running parallel to the shoreline through the city. The damage in this zone was a significant obstacle to rescue efforts and in delaying restoration of lifelines after the quake. The geotechnical/ geological reasons for this were not clear and may emerge following further study by the Japanese.

(A brief outline of the effects of the Kobe earthquake is presented by Standards NZ in their article later in this newsletter).

The Use of Cone Penetration Techniques for Site Contamination Studies

On 19 April, Environmental Geologist, Simon Hunt of Tonkin & Taylor Ltd, Auckland presented a lecture to Auckland Branch members on his experience using the cone penetration testing techniques to investigate contaminated sites in Britain. Simon described different types of cones and probes developed in Europe, particularly by his former employer Delft Geotechnics, to detect, sample and measure different parameters such as pore pressures, conductivity, soil gas, groundwater contamination, soft sediment sampling, etc. Key advantages of the CPT in this type of work are rapid data acquisition and minimal soil/groundwater disturbance or contamination during sampling.

Two British case histories were presented where site contamination was extensive and which CPT techniques had been used successfully to investigate the sites.

Proposed Meetings

Thurs. 25 May	Hayes Creek Dam Upgrade (V. Jairaj, Watercare)
Thurs. 6 July	Stability Assessments Forum
Thurs. 17 August	Engineering Roles - Designer, Reviewer, Council
Thurs. 28 September	Geomechanics Society Student Prize - Presentations
Thurs. 2 November	SH20 Mangere Motorway Extension & Ormiston Road Projects

Clive Anderson
AUCKLAND BRANCH CO-ORDINATOR

WELLINGTON BRANCH

March 1995 - The NZSEE team gave their presentation on the Kobe earthquake.

(A brief outline of the effects of the Kobe earthquake is presented later in this newsletter by Standards NZ).

P. Brabhaharan
WELLINGTON BRANCH CO-ORDINATOR

LOCAL GROUP ACTIVITIES

GEONEWS

CHRISTCHURCH BRANCH

March 1995 - The NZSEE team gave a presentation on the Kobe Earthquake.

(A brief outline of the Kobe earthquake is presented later in this newsletter).

Guy Grocott
CHRISTCHURCH BRANCH CO-ORDINATOR

OTAGO BRANCH

Dr Mauri McSaveney, (Institute of Geological & Nuclear Sciences, Lower Hutt) gave a Lunchtime Seminar on "Recent Rock Avalanches of Mt Cook National Park" at the start of April 1995.

We are planning to have a seminar on the Clyde Power Project with respect to the current state of landslides and landslide monitoring. At this stage, this seminar is proposed for June and will be held at the Crown Research Building, 764 Cumberband St, Dunedin. Further details will be advertised locally closer to the meeting time.

Philp Glassey
OTAGO BRANCH CO-ORDINATOR

NZGS members recently came up winners at the 1994 IPENZ Awards, taking four awards (including the prestigious Fulton Downer award) and a highly commended. The winners are listed below along with a description of their prize.

FULTON-DOWNER AWARD (GOLD MEDAL) - This award was presented to **Graham Ramsay** for his technical paper "Ewen Bridge Replacement Foundation Construction". (*A copy of this paper is included in this issue of NZ Geomechanics News*).

The Fulton-Downer Awards were established in 1929 by a bequest from the late J.E. Fulton to which were added, in 1973 and 1988, donations from A.F. Downer. The Fulton-Downer Award is the premier award of the Institution and consists of two medals, a gold and silver and a sum of money.

The Fulton-Downer Gold Medal is for presentation to a person in any class of IPENZ membership presenting the best paper on a technical subject at an annual IPENZ conference. No award is made unless the papers presented are up to the standard set by the assessors.

ENVIRONMENTAL AWARD - This award was presented to **Tonkin & Taylor Ltd and Waste Management (NZ) Ltd** for their entry on the Redvale Landfill Project.

The winning project was innovative at the time for New Zealand. The firms demonstrated a policy of environmental management beyond statutory requirements and they achieved excellent public consultation which led in large measure to community acceptance of the project. In carrying out the project, it was apparent the firms had met the requirements of the IPENZ Environmental Code.

The Environmental Award made approximately every two years, is for the predominantly engineering work, which in the opinion of the judges best exemplifies care for and consideration of environmental values. Account is taken of the identification of environmental values in the design, the manner in which the resulting problems were resolved and the overall contribution of the end result to environmental values and public enjoyment.

Projects to qualify for the award do not have to be of national importance but may be of significance only in their immediate locality.

The recipient of the Award may be an individual or public or private body, and in the case of an individual, the person need not necessarily be an engineer or a member of the Institution.

FURKET AWARD - This award was presented to **Timothy Sinclair and Christopher Freer** for their paper "Aspects of Tailings Dam Design for the Golden Cross Mine" published in the Proceedings of the New Zealand Geomechanics Society Symposium, "Geotechnical Aspects of Waste Management" Volume 20 Issue 1 (G) May 1994 and described by the Committee as providing good background information on the subject proceeding to specific design details.

The Furkert Award was endowed by the late F.W. Furkert, a former Engineer-in-Chief of the Ministry of Works and Past-President of the Institution of Engineers.

The Award is made for the best paper by an IPENZ member(s) 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, published by the Institution within the three year period ending 31 July preceding the Conference at which the award is given.

RABONE AWARD - This award was presented to **Anthony Kortegast** for his paper "Redvale Landfill - General Design and Construction Considerations" published in the Proceedings of the New Zealand Geomechanics Society Symposium on "Geotechnical Aspects of Waste Management", May 1994 as described by the Committee as an extensive review of the design elements associated with the construction of municipal Solid Waste landfill facilities providing both an assessment of the suitability of overseas design codes to New Zealand conditions and a summary of the work undertaken.

The Rabone Award was endowed in 1969 to recognise papers of special merit in categories other than those covered by the awards existing at the time.

The Award is for the best paper by any class of IPENZ member(s) preferably under 40 years, on a subject of a general nature which does not qualify for another of the Institution awards, published by the Institution during the five year period ending 31 July preceding the Conference at which the Award is given.

CARTER HOLT HARVEY INNOVATIVE TECHNOLOGY AWARD

A "highly commended" award was presented to **Murray Gillon** and **Greg Saul** for their paper "Cairnmuir Landslide Infiltration Protection Stabilisation Works" and their work in remediating the Cairnmuir landslide above Lake Dunstan at the Clyde Dam. (*A copy of their paper is included in this issue of NZ Geomechanics News*).

Carter Holt Harvey Packaging sponsored two awards in Innovative Technology - one for the paper in the technical sessions which described the most innovative technology.

Eight technical groups selected the most innovative paper in their sessions and then a committee selected four to be presented at a main session at the Conference, when the final winner was selected.

These Carter Holt Harvey Packaging awards were a new development for the IPENZ Conference and it is hoped that they may be continued so that the engineers can be encouraged to be innovative and their's is recognised not only by IPENZ but by the wider New Zealand public.

STANDING ON SHAKY GROUND

Professor Robert Park, Deputy Vice Chancellor of the University of Canterbury, was leader of the reconnaissance team sent to Kobe six days after the earthquake. In the following edited account, Professor Park stresses the need for New Zealand to enforce good building practice if it is to endure a major earthquake.

At 5.46 am on Tuesday, 17 January 1995, in south Kobe, a 7.2 Richter magnitude earthquake occurred with epicentre 20 km out to sea, off Awaji Island, with its source 16 km deep.

The strong ground shaking in Kobe lasted 20 seconds and the maximum horizontal ground accelerations recorded were 0.85 g.

The result: 5,400 people dead, and more than 33,000 injured; around 80,000 buildings badly damaged including many collapses, hundreds of thousands of people homeless; major roadways and railways cut due to collapses of bridges; fallen debris and ground surface movements which distorted railway lines; port facilities made unusable; electricity, water and gas supplies cut; telephones out of order; and sewerage disposal uncertain.

Some damage was particularly devastating. A hospital building in which the columns of the fifth storey collapsed caused the floor above to drop, crushing to death 49 people.

More than 75,000 residential houses were badly damaged, including many collapses, followed by terrible fires in many areas. On the fourth day after the earthquake about 310,000 residents spent the night at 1,077 refugee centres.

An elevated expressway, which was a main artery for traffic movement between Osaka and Kobe, collapsed over part of its length, and many railway bridges collapsed. Japan's top container port was put out of action due to ground movements (settlement and spreading) and crane collapses in the berth areas.

The New Zealand National Society for Earthquake Engineering and the Earthquake Commission of New Zealand sponsored the 13-strong team to visit the devastated area. The team's objectives were:

- to compare damaged and undamaged areas and structures, so as to bring back lessons for New Zealand

- to assess the preparedness and emergency response.

Damage

While traditional Japanese houses performed badly, modern housing, with lighter roofs and better bracing against lateral shaking, stood the earthquake well.

Seismic codes in Japan, New Zealand and other countries do not recommend design seismic forces which will ensure that during a major earthquake the structure will remain in the elastic range. That is, the strength of the structure may not be as great as the inertia forces that could be imposed by the ground shaking. To remain in the elastic range would require huge seismic design forces.

Modern buildings in Kobe, which were designed and constructed in accordance with the most recent Japanese seismic code (1981), performed well on the whole. Care is needed when comparing this finding to New Zealand structures. The design seismic forces used in Japan are higher than those used in New Zealand for ductile design, but nevertheless the use of capacity design in New Zealand and the ductile detailing used in New Zealand (for a greater expected ductility demand than in Japan), give confidence that the current design Standards in New Zealand are adequate. This emphasizes the need in New Zealand to enforce our current building design and construction Standards strictly, and again justifies the considerable upgrading of our seismic Standards which has occurred since the mid-1970s, commencing with the general design and loadings code NZS 4203:1976.

This upgrading involved the introduction of capacity design, in which it is ensured, as far as possible, that the balance of strength of structural elements is such that, in the event of a major earthquake, yielding (post-elastic deformation) only occurs in regions of the structure which are chosen by the designer, where it can be tolerated by the structure. The chosen regions where yielding is expected are carefully detailed by the designer to ensure that adequate ductility is available there.

Many older buildings in Kobe did not fare so well – typically buildings of the 1950s and 1960s, which were designed to old (now sub-Standard) Standards. Most deaths in buildings were due to the collapse of the columns of a storey, typically the bottom but also at times an upper storey, crushing people as the floor slab above those columns pancaked. Engineers refer to this as a "soft storey failure". Modern design requires stronger columns to prevent this column failure and more ductile behaviour in the case of yielding. Those buildings had been designed according to the Standards of the day.

The Kobe earthquake emphasizes again that many of New Zealand's older buildings need to be assessed for seismic resistance in the light of current seismic design Standards. Some older structures are inherently strong and have satisfactory earthquake resistance. Others are neither strong nor ductile and need to be retrofitted.

In addition, the seismic forces used in Japanese design correspond to slightly greater than those associated with limited ductility design in New Zealand. Hence, more ductility is required of New Zealand ductile structures to survive a similar earthquake.

The lesson is that buildings which did not have good seismic detailing ensuring ductile behaviour in the inelastic range often performed poorly, with severe damage and collapse common in areas of high intensity shaking. Almost all the failures of medium rise buildings could be attributed to a lack of capacity design to prevent brittle failure mechanism and a lack of ductile detailing, plus some failures which could be attributed to inadequate lateral strength.

The 20 seconds of strong ground shaking of the Great Hanshin Earthquake has provided many important lessons for New Zealand.

- Buildings designed to modern seismic codes survived the earthquake well. This justifies the design and construction provisions of current codes (which are very much more severe than older codes) and emphasizes the need to enforce current codes strictly.

- Many pre-1970s structures in New Zealand may need retrofitting.

- Lifelines of cities need to have adequate seismic resistance. The use of buildings after an earthquake will be severely hampered if, for example, the water is cut for weeks.

- Disaster preparedness and emergency response services are of critical importance. How well prepared are we for the effects of a major earthquake in an urban area of New Zealand?

SLOPE STABILITY ASSESSMENTS FOR LAND DEVELOPMENT

Dear Sir,

My congratulations to the Society for airing this subject. It is 20 years since the Nelson Symposium, 17 years since the Palmerston North Symposium and certainly time to try to resolve differences which complicate approvals, especially for housing and other relatively low cost developments.

The results of a questionnaire on this topic published in NZ Geomechanics News, No.48, December 1994, show a wide range of opinion amongst those who responded and significant differences amongst professional people with some degree of specialist knowledge of the topic, as to the appropriate means of assessing stability and as to who shall make the assessment.

If it were simply a technical or scientific matter, then a range of opinion might be a sign of healthy scepticism. Unfortunately, property, assets and sometimes lives are at stake and that means that opinions will be subject to legal scrutiny seeking "black and white" answers.

Courts of law in New Zealand reply upon expert witness to advise them on technical matters, and, more specifically what standards of care are the norm for the profession in question.

That there is such a range of opinion as to what constitutes an acceptable investigation of land stability, bodes ill for the professional justifying him/herself against opposing witnesses in a Local Body public hearing or in a court action. Without seeking to stifle all innovation, it is important that the NZ Geomechanics Society reach a consensus on, at least, the broader issues of slope stability assessment.

Perhaps we can proceed by establishing some Points of Agreement and then examining questions which follow.

Possible Points of Agreement:

Can we agree that:

(1) It is the responsibility of the Territorial Authorities to consider, and reach decisions about land stability in the processes of planning and approving land development, although they may rely upon the opinion of professional experts to guide them.

(2) Geomorphological and geological assessment and evaluation of the performance history of the area containing a particular site for development, is an essential first part of a slope stability investigation - whatever further investigation may be considered necessary.

(3) The extent of these further investigations depends upon a judgement of the degree of risk (of instability) and the hazard to property, assets and life in the particular case.

(4) A quantitative analysis of slope stability is useless, or even misleading, unless it is based upon accurate definitions of the geology, soil properties and groundwater pressures at the site. Such definitions are expensive to achieve.

(5) The design of engineering works involved in land development, including measures to improve stability, requires professional training and experience in civil engineering.

If we can accept these points, then we can proceed to ask some leading questions:

(6) Who is competent to make the geological assessment (2 above)?

(7) Should the geomorphological/geological assessment be accepted as a judgement by the Territorial Authority in the first instance?

(8) Who should decide whether further investigations are necessary?

(9) Who should decide whether the necessary investigation is too expensive, or uneconomic?

(10) Should there be established Codes of Practice setting the essential features and minimum content of stability investigations as an amplification of Clause 2 of Regulation B1/VM4 of the Building Act?

(11) Should the Territorial Authorities rely upon the geomorphological/geological investigation, without quantitative analysis, in deciding "likely" or "unlikely" for the purposes of Section 36 of the Building Act?

My own opinion on these questions is as follows:

1 to 5 I regard as axiomatic.

(6) A geologist qualified by a University Degree and experienced in assessment of land stability, preferably with experience in advising on civil engineering development, should make the assessment for the benefit of the engineers responsible for the development. Regional geological assessments already may be available.

(7) The Territorial Authority could reasonably rely upon a competent and comprehensive report and leave the developer to decide whether he/she will bear the cost of more detailed investigations to challenge the decision.

(8) The developer or his advisers may decide that further work is necessary to justify their proceeding, or the Territorial Authority may direct that further investigation must be done if the Developer is not prepared to accept the Territorial Authority's decision based upon the geological examination.

(9) The professional adviser might advise that he considers the cost of further investigations to be excessive in relation to the potential benefits but the decision **must** be the Developer's. It will be no defence in law if the adviser stops short of appropriate (in the eyes of the profession) investigations because of their costs.

(10) The Building Regulations do not define the "appropriate" investigations and the quality of some of the investigation reports being submitted to Territorials Authorities is quite unacceptable by reasonable professional standards. This may be the result of ignorance or of pressure from developers to cut investigation costs. The Territorial Authorities must take a stand to prevent either source of inadequacy.

On the matter of Registration of Technical Competence:

If the specialists in the profession (i.e. Geotechnicians) can't agree on the fundamental points discussed above then there is no way that IPENZ or any other professional body can do it for them.

The Territorial Authorities have an obligation to satisfy themselves about the competence of the advice they receive (directly or via developers) and cannot rely simply upon the advisers judgement of his own capability - no matter what the IPENZ Code of Ethics says, for the remedy under that heading will always be far too late.

Don Taylor

"GEOLOGICAL ENGINEERING OF EMBANKMENT DAMS"
by Robin Fell, Patrick MacGregor
& David Stapledon; Balkema, 1992

Dear Sir,

This title was included in "Book Reviews" in the December 1994 issue of Geomechanics News, but without any analysis of the content. It has many good features, notably the treatment of selecting the most appropriate investigation methods and recognising the key questions which need to be answered in relation to various geological environments, in order to identify appropriate engineering solutions. Much of this is based on well-illustrated actual examples.

Every geotechnical professional should at least borrow, if not own the book. Don't be put off by the dam title - the investigations section will be relevant to any project.

Bruce Riddolls

The response rate to the questionnaire on possible new names for the NZ Geomechanics Society, as enclosed in the last issue of Geomechanics News, was poor. Fifty three responses were received from approximately 350 members, i.e. 15%. The results were clearly in favour of a change to "NZ Geotechnical Engineering Society."

NZ Geotechnical Society	70%
NZ Geomechanics Society	15%
NZ Geotechnical & Geological Engineering Society	9%
NZ Geosciences Society	4%
NZ Ground Engineering Society	2%

Other Suggestions:

NZ Geotechnical Engineering Society	6%
NZ Earth Sciences Society	2%
Australasian Geotechnical Society	2%

If you did not respond and still wish to, please send your preference to the Editor. The Management Committee are still considering the issue. Any proposal to change the name will be brought to an AGM.

The following letters to the Editor from John Galloway and Stuart Read give some background to the origins of the Society's name. (Also of interest is the recent correspondence regarding the proposed name change for the ISSMFE - see later in this issue of the newsletter).

"Dear Sir,

At the start of the seventies when Mick Pender, Peter Imrie and I were drafting new rules for the NZ National Society for Soil Mechanics and Foundation Engineering, the choice of a suitable new name for the Society was one of the matters we had to consider. Clearly, we needed to find something much shorter and in the end, we decided to follow the Australians who had recently adopted "Geomechanics Society". The obvious logic for this choice was that it gave consistency throughout the Australasian Region. The logic still prevails and from the point of view of information retrieval, has enduring merit. To change the name of a serial such as "Geomechanics News", or of a continuing series of conferences, complicates the tracing of references. In my view, it should never be done without specific and cogent reason. I do not consider such reason currently exists.

As regards the name of the Society, my personal justification for the name "Geomechanics" was that it neatly summarised the interests of the Society. The Society was, and still is, I believe, a civil engineering organisation and as much, is concerned with the development of the surface of the earth for the benefit of mankind. The main mental disciplines used by engineers in development were those of the "mechanical sciences" so the name "Geomechanics" was a sort of shorthand for the phrase "The application of the mechanical sciences to the development of the surface of the earth", which I consider a fairly concise description of the interests of the Society at the time it was formed.

But as time passes, perceptions change! I consider civil engineering remains the core of the Society's concern, but the prudent civil engineer is now expected to be aware of how the works he undertakes will react with the whole environment during construction and when in use. Much more than the "mechanical sciences" is now involved. So a good case can be made for a new name which, in a single word, summarises the scope of the Society's interests. These, I consider, are:

(a) Civil Engineering is the core interest, (i.e. the Society is concerned with development and works, and not just with knowledge for its own sake).

(b) The importance of our scientific colleagues in all branches of knowledge relevant to the wise design and execution of developments.

(c) The importance of the many sub-professionals whose knowledge and skills are vital to the successful completion of works.

(d) The links to the International Societies (IAEG, ISRM, ISSMFE).

I cannot think of a word which conveys all these aspects and wonder if such a word exists! In terms of my original justification, I consider that "Geotechnical" is a better approximation to the current interests of NZGS than "Geomechanics" and for this reason, is to be preferred. But I still hope that someone has the inspiration to coin a new word which more completely summarises the interests of NZGS.

Yours faithfully,

J.H.H. Galloway
LIFE MEMBER

Dear Sir,

This is not the first time that name changes have been mooted, by us or even the ISRM, IAEG and ISSMFE to group together under the one umbrella as geomechanics.

Our grouping of soil mechanics, rock mechanics and engineering geology is a natural one and endorsed by the Aussies in their name (with them we stick together on this one). All other societies cover one or two of these 3 disciplines, and in particular geotechnical engineering, be it ASCE, UK or Canada, do not outwardly cater for engineering geology directly and there are separate societies for these activities e.g., AEG, Geological Society.

I would debate that geotechnical engineering is more popular today and wider understood - it remains one branch within geomechanics. We have had, and still have, something useful and worthwhile for educating people in New Zealand and overseas - be proud of it and don't let changes be made for the sake of popularity, thus letting the tail wag the dog.

Yours faithfully,

Stuart Read

PROPOSED NAME CHANGE OF THE SOCIETY - PRELIMINARY CONSIDERATIONS

Dear Members,

In the last two decades, Geotechnical Engineering has experienced a wide, impressive expansion that has lead to the development following:

- Several new areas of activity, such as Environmental Geotechnics, Earthquake Engineering, Offshore Engineering, etc.
- An increased importance, both from a theoretical and a practical technological standpoint, of activities in a number of fields including, for example: Soil Improvement, Underground Construction in Soils and Soft Rocks, Mining Waste Disposal, etc.
- An increasingly decisive role in Regional Planning and Natural Hazard Assessment as well as in the rationalisation and modernisation of transport infrastructures, such as high speed railway systems, continental highway networks and offshore airports.

With such scenario in mind and looking ahead, beyond the year 2000, many geotechnical engineers have been questioning if the present name of our Society is fully representative of the role and of the range of activities and responsibilities in modern Geotechnical Engineering.

In the light of the above considerations, the ISSMFE Board in its last meeting, held in Edmonton on July 10th, started a preliminary discussion on the possibility of sounding the Member Societies' views as to a possible change of the name of our Society.

The change is aimed at making its name more appropriate to the role that Geotechnical Engineering plays in the world of Modern Engineering Disciplines, as well as at better reflecting the current terminology.

In facing this problem, the Board Members were fully aware that opening an inquiry on such a topic is quite a delicate and many-sided matter. It cannot be ignored that the present name, of International Society for Soil Mechanics and Foundation Engineering was coined by K. Terzaghi, the founder of the modern geotechnics, and that, for half a century, it has represented the benchmark for all the devotees of our area, all over the world. Moreover, any decision regarding a change in the name of our Society should take into account the existence and the areas of interest of our Sister Societies International Association of Engineering Geology (IAEG) and International Society for Rock Mechanics (ISRM).

In these circumstances and in the attempt to cope with the different requirements, the Board has decided to explore with all Member Societies their opinion for the following possible name:

International Society for Soil Mechanics and Geotechnical Engineering

Retaining the present term "Soil Mechanics" within the name will ensure the continuity of our heritage and our tied links with the theoretical fundamentals of continuum and particulate mechanics as applied to geomaterials. On the other hand, the replacement of the term "Foundation Engineering" with "Geotechnical Engineering" would meet the requirement of better characterising the fields of application of our discipline nowadays, proceeding well beyond engineered construction design.

Aiming at getting an option of the International Geotechnical Community's with respect to a change from ISSMFE to ISSMGE, I would appreciate if you could consider the proposal and fill in the enclosed form, sending it before March 1, 1995, to the ISSMFE Secretary General, Dr R.H.G. Parry.

Thank you for the special attention you will put to the matter.

Sincerely yours

Prof. M. Jamiolkowski
PRESIDENT, ISSMFE

PROPOSED NAME CHANGE FOR ISSMFE

Dear Sir,

With reference to Prof. Jamiolkowski's letter of 13 October 1994, which was discussed at our Management Committee meeting on 13 February last and also at the Society's AGM on the same day, I have been instructed to respond to you as follows on the matter:

1. We fully appreciate that ISSMFE members are today engaged in considerably more than just "Soil Mechanics" and "Foundation Engineering". We also accept that the term "Geotechnical" or "Geotechnical Engineering" is increasingly being used to describe these professional activities. We note as well that there is an increasing involvement in what can be called "Environmental Engineering", and that this is closely related to but may be quite distinct from the practice of "Geotechnical Engineering".

2. Our principal concern in this matter is, however, that the term "Geotechnical" involves and implies the three disciplines of soil mechanics, rock mechanics and engineering geology. As you are well aware, it is for this reason that the Australian and New Zealand Geomechanics Societies exist as single organisations representing the national activities of ISSMFE, ISRM and IAEG in each country. Based on our experience in New Zealand, our preference would be for an "International Geotechnical Society" which combined the functions of the present three international societies, but we realise that for historical and other reasons such a development is extremely unlikely in the short to medium term (i.e. 5-10 years).

3. We therefore cannot support your President's proposal because we consider that any move to establish an international "Geotechnical" Society must involve the three disciplines of soil mechanics, rock mechanics and engineering geology. Given the format of your questionnaire we have therefore indicated our preference for the status quo, and we note that there are sound historical reasons why ISSMFE should continue with its present name. The "International Society for Soil Mechanics and Foundation Engineering" has considerable status as a professional organisation representing the broader interests of those involved in "Soil Engineering", and it would certainly not be detrimental for this name to continue being used.

4. If there is strong support for ISSMFE to change its name, then we would suggest as an alternative the "International Society for Soil (or Soils) Engineering". This name accurately identifies the primary function of the Society as its involvement with all aspects of "Soil" (or "Soils") engineering, and has the shorter abbreviation "ISSE". The use of such a term avoids any argument about whether you are dealing with soil mechanics, foundation engineering, environmental or geotechnical aspects of the subject, and yet retains the ISSMFE emphasis on "soils".

Given the short time still available for a response, I will fax this letter to you. We look forward to future developments on this question, and I note in passing that our Society members are presently debating whether or not to change its name to the "New Zealand Geotechnical Society".

With my best wishes.

Yours sincerely,

David H. Bell
CHAIRMAN - NEW ZEALAND GEOMECHANICS SOCIETY

"GEOTECHNICAL ISSUES IN LAND DEVELOPMENT"

Hamilton, Friday 16 - Sunday 18 February 1996

INFORMATION

The aims of the New Zealand Geomechanics Society are:

(a) To advance the study and application of soil mechanics, rock mechanics and engineering geology among engineers and scientists.

(b) To advance the practice and application of these disciplines in engineering.

(c) To implement the statutes of the respective international societies in so far as they are applicable in New Zealand.

A topical area currently facing the engineering profession is the issue of land development. The symposium hopes to address this issue, and provide a forum for discussion.

The last Geomechanics Society symposium dealing with land development was in 14 years ago (Geomechanics in Urban Development, 1981). Significant changes to the legal system in New Zealand, particularly the introduction of the Resource Management Act, have been made since 1981; what impact has the Act had on geotechnical practice? Technology has advanced considerably, particularly the use of personal computers, and many local authorities now use a geographic information system (GIS) to monitor and track resources and developments in their areas; how is the geotechnical profession applying this technology? Are hazards effectively identified? There is also increasing pressure to develop marginal land - should we recommend construction on dormant landslides for example?, and what are implications of developing sites on contaminated land? How are hazards mitigated? Some organisations are generating approved lists of consultants - are these appropriate?

Discussion sessions will cover topics such as Planning & Development Guidelines and appropriateness of "approved consultants" lists.

The Society is delighted that Don Taylor will present the keynote address. Mr Taylor is a Fellow of IPENZ and a former Managing Director of the engineering consulting company Tonkin & Taylor. Mr Taylor has extensive experience with investigating sites in difficult ground".

If you wish to obtain further information, please contact the:

NZGS Symposium Organiser Tel: (07) 838 9344
Mr Tim Browne Fax: (07) 838 9324

PRELIMINARY REGISTRATION

Delegates are requested to register their interest in attending the symposium by returning the attached form. If sufficient numbers of accompanying persons register, a day trip programme will be arranged which would include visiting the Waitomo Caves and the Otorohonga Kiwi House.

CALL FOR PAPERS

Papers are requested for the following themes:

- Legal and Planning Framework - The Resource Management Act
- Hazard Identification and Risk Analysis
- Investigation Analysis and Assessment of Sites
- Development on Marginal Land - contaminated sites, dormant landslides, landfills, steep slopes
- Mitigation Case Histories
- GIS in urban development
- Planning and Development Guidelines

Delegates wishing to prepare a paper for the symposium are requested to supply an abstract by 1 July 1995, with final manuscripts supplied by 30 September 1995. Any suggestions related to the symposium would be welcomed.

PRE-REGISTRATION FORM

Name: ______________________________

Organisation: ______________________________

Address: ______________________________

____

____

Phone: ____________________ Fax: ____________________

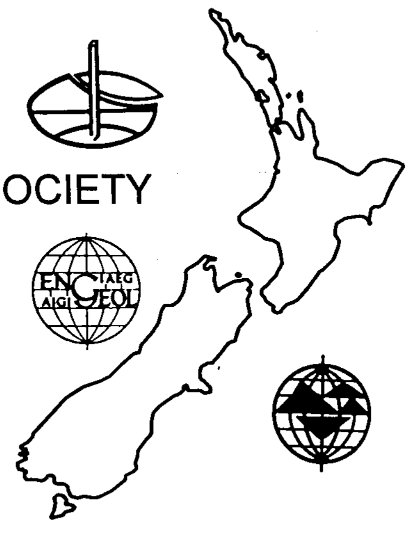
Please tick appropriate box:

I plan to attend the symposium and present a paper	YES ☐	NO ☐
I will require accommodation	YES ☐	NO ☐
I plan to attend conference dinner	YES ☐	NO ☐
I will be accompanied and wish my partner to attend:		
accompanied persons programme	YES ☐	NO ☐
conference dinner	YES ☐	NO ☐

Please complete the pre-registration form, and mail or fax it to the address below by 1 July 1995.

Tim Browne
Geomechanics Symposium Organiser
Private Bag 3057
Hamilton
NEW ZEALAND

Fax: (07) 838 9324



NEW ZEALAND GEOMECHANICS SOCIETY

A Technical Group of IPENZ
The Institution of Professional
Engineers New Zealand

Suggestions:

THE SECOND AUSTRALIA-NEW ZEALAND YOUNG GEOTECHNICAL PROFESSIONALS CONFERENCE

NOTICE/CALL FOR PAPERS

29 NOVEMBER TO 2 DECEMBER 1995
AUCKLAND, NZ

AIMS

- To bring together young geotechnical engineers and engineering geologists so they may become more aware of the work of others (with similar experience) and to benefit from the insights of those with considerably more experience.
- To provide young geotechnical professionals with a more active role in Australasian geomechanics societies (i.e. NZGS & AGS) and prepare them for a future leading role in Society affairs.
- To continue the success of the first conference of this type in the Australia-NZ region.

LOCATION & TIME

From 29 November (Wednesday evening) to 2 December 1995 (Saturday afternoon) at O'Rourke Hall, the University Auckland, Auckland, New Zealand. It is hoped that organisations from NZ, Australia and the Pacific will sponsor registrants to this conference.

WHO SHOULD ATTEND?

Applications are encouraged from any young geotechnical engineer or engineering geologist. We consider that young in this context is 35 years old or younger. The organising committee hopes that major Engineering organisations, Universities, Consultants, Regional Councils, Crown Research Institutes or Government departments will sponsor one or two registrants. It is expected that each organisation will cover the full conference cost for their registrants which will include travel to and from the conference and the conference registration fee of $400.00 (which includes accommodation, meals and site visits).

The number of registrants will be limited to about 30 to ensure a relatively relaxed atmosphere and to enable all registrants to present papers. If too many applicants are received, then applicants will be selected on the basis of maintaining an equitable distribution between organisations and on a review of the synopses submitted by intending registrants. If more stringent selection is required, then the organising committee will select applicants between five and ten years from initial graduation. Organisations should select potential registrants with a view to encouraging those they see as having a future leading role in the geotechnical community. It should be seen as an honour to be selected by your organisation to attend the conference.

FORMAT

- The conference will be residential even for those registrants who reside in the Auckland area. Registrants will arrive on Wednesday after 6 p.m. and depart on Saturday after 4 p.m.
- The daily programme will consist of: presentations by the registrants; a guest lecture; a site visit; an informal lunch; and an evening function.

THE SECOND AUSTRALIA-NEW ZEALAND YOUNG GEOTECHNICAL PROFESSIONALS CONFERENCE

NOTICE/CALL FOR PAPERS

- The guest lecturers are being invited from experienced and well respected professionals from a variety of local organisations. It is expected that there will be lectures on topics from engineering geology, environmental geomechanics and soil and rock engineering. The speakers, along with the organising committee and other experienced local professionals will act as mentors for the day.

- Each registrant will have to submit a 3 to 5 page "paper" on a project (research or work-related) in which they have been involved. These papers will be assembled into a limited distribution proceedings which will be distributed prior to the conference. The registrants will then present their paper to the conference. The presentation is expected to be of 10 minutes duration with a question/discussion period at the end of each group. These papers are intended to present some of the more interesting work each registrant has been involved with. They will provide valuable experience in preparation and presentation of papers and enable the ready transfer of experience between registrants. This activity is a requirements of all registrants. The audience for these presentations will therefore be, say, four or five senior professionals and the young registrants themselves. Slide projectors, overhead projectors and video facilities will be available.

- The site visits will be to local engineering projects and areas of geological interest and led by experienced geotechnical professionals. Auckland offers some 64 dormant volcanoes within the greater urban area as well as sedimentary (Jurassic and Miocene and volcaniclastic formations).

- Informal social gatherings are planned for the evening. The programme is yet to be finalised but will probably include: first evening, registration and drinks; second, meal at a local restaurant; third, site visits and either a barbecue at a local vineyard or a harbour cruise.

COST

Registration is $400.00. This cost has been kept to an absolute minimum and includes:

- College accommodation, breakfasts and lunches
- Transport to site visits
- Three evening functions
- Proceedings

ARE YOU INTERESTED?

If you or your organisation are interested in participating/sponsoring, please send the attached **Application Form** by 31st May 1995. Note that numbers will be limited so please respond as soon as possible.

We look forward to your participation and to meeting with you in Auckland at the end of November.

EDITOR'S NOTE: *A brief report on two Young Geotechnical Engineers Conferences (YGEC's) is presented in the ISSMFE February 1995 newsletter:*

- *2nd Asian YGEC held at the Asian Institute of Technology (AIT), Bangkok, Thailand, in late June 1994. The 3rd Asian YGEC will be held in 1996.*

- *8th European YGEC held at the High Tatras Mountains in Czechoslovakia in early September 1994. The 9th European YGEC will be held in Belgium in 1995.*

NOTICE

THE SECOND AUSTRALIA-NEW ZEALAND YOUNG GEOTECHNICAL PROFESSIONALS CONFERENCE

Application Form

FAMILY NAME:
Mr/Ms/Mrs/Miss/Dr: ____________________

GIVEN NAMES: ____________________

Organisation: ____________________

Geotechnical Engineer or Engineering Geologist

Other: ____________________

Address for correspondence: ____________________

____

Phone: ____________________

Fax: ____________________

Qualifications & dates awarded ____________________

Date of Birth (optional): ______/______/______

Brief experience:

Proposed title of paper (please attach a one to two paragraph summary)

Declaration:

If accepted to attend the conference I will submit a 3 to 5 page paper on the topic indicated above (or approved alternative) and pay registration of $400 by 30th September 1995.

Signed: ____________________

Please return before 31st May 1995, preferably by post to:

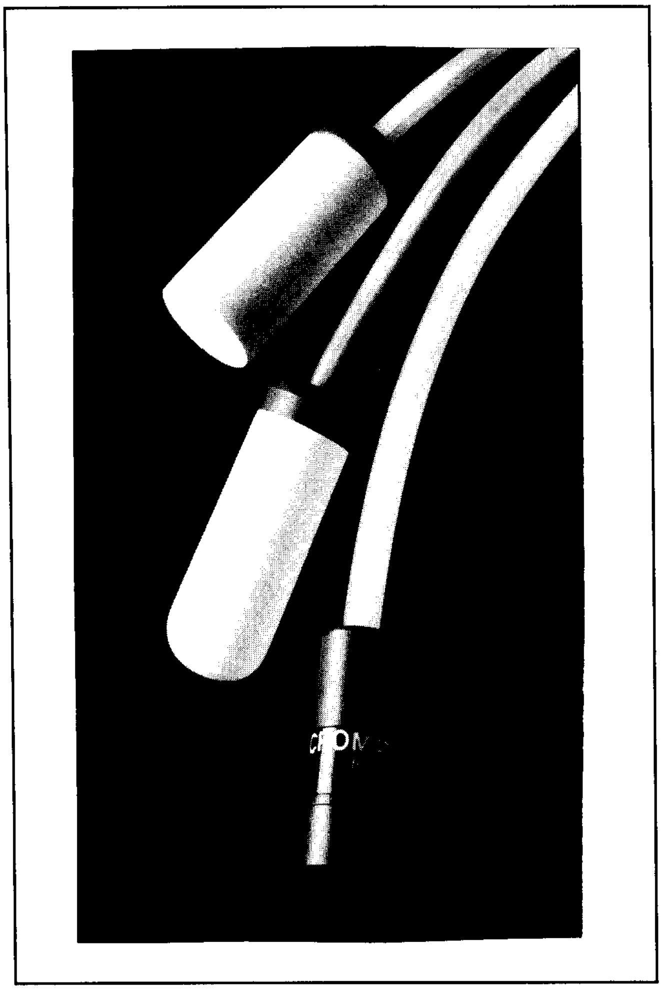
Mr Sergei Terzaghi
Organising Committee
Second ANZ Young Geotechnical Professionals Conference
C/- Auckland IPENZ Branch Office
P O Box 6748
Auckland
NEW ZEALAND

Tel: (64-9) 379-3515
Fax: (64-9) 379-7550

GEOTECHNICAL INSTRUMENTATION

- Pneumatic Piezometers
- Hydraulic Piezometers
- Inclinometer Tubing
- Data Loggers
- Vibrating Wire Piezometers
- Load Cells
- Hydrostatic Profile Gauges

Agents for: ***Soil Instruments (UK)***
Geosystems (Aust.)
QED - Groundwater Specialists (USA)
BAT - Groundwater Systems
Enviroequip (Aust.)
Solinst (Canada)
RST Instruments (Canada)



Pneumatic Piezometers

Water Level Meter

MONITORING/SAMPLING EQUIPMENT

- Borehole Screen (slotted PVC & access)
- Water Level Meters
- Filter Sock
- Borehole Upstands - lockable
- Disposable Bailers
- Waterra Pumps and Access
- Walton Park sand and gravel
- pH, Temp and Conductivity Meters
- Toby Boxes
- Soil Probes
- NOMIS Seismograph (standard or mini version)

GEOTECHNICS LTD — SALES DIVISION
UNIT 3C 76 CARR ROAD MT ROSKILL AUCKLAND
PO BOX 27 053 MT ROSKILL AUCKLAND
PH 09 620 0280 TOLL FREE 0508 223 444 FAX 09 620 0281

RULES OF AWARD

1. The New Zealand Geomechanics Society wishes to recognise and encourage student participation in the fields of soil mechanics, rock mechanics, and engineering geology. It has therefore agreed to present annually two merit awards, each of 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 Synopses shall not 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 clarity, 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 through the National Activities Officer regarding the timing, format and venue for the annual Student Prize Meeting in each centre. they are also to ensure that the awards are made each year under generally similar conditions, and that invitations to participate are extended to students at institutions outside Auckland and Christchurch.

D.H. Bell

NOTE: Students wishing to submit a paper for the 1995 NZGS Student Prize should contact:

Stuart Palmer
NZGS National Activities Officer
C/o Beca Carter Hollings & Ferner
P O Box 3942
Wellington
Tel: (04) 473 7551
Fax: (04) 473 7911

THE DEADLINE FOR SYNOPSES IS 30 JUNE 1995

1996 IPENZ CONFERENCE

"ENGINEERING : PROVIDING THE FOUNDATIONS FOR SOCIETY"

Dunedin 9-13 February 1996

This is a call for papers for presentation at the 1996 IPENZ Conference by members of technical groups and members of IPENZ.

The theme of the conference is "Engineering - providing the foundations for society". Engineers and allied professionals have played an integral part in the establishment of society as we know it today be it in infrastructure, services, structures and civil works, industry, manufacturing, agriculture, catchment and flood control, provision of energy or material development. The theme of the 1996 IPENZ Conference will provide the opportunity to review the activities of the past, study the actions of the present and to explore the exciting opportunities and challenges of the future.

The Institution is made up of members from a wide range of groups involved in every aspect of engineering and allied disciplines. The Annual IPENZ Conference provides an ideal opportunity for your group to conduct sessions which enable those in other fields of activity to see and hear what are the key issues, developments, challenges an exciting advancements in the future in your field. Participation in the conference will give each group an opportunity to provide a "window" on the activities of its members which will be of interest and benefit to others.

Papers must be submitted in the format and to the timetable required by the conference committee and the papers must be presented personally on Sunday 12 February, Monday, 12 February or Tuesday, 13 February. All papers presented will be published in the Conference Proceedings prior to the conference and these will be supplied to all delegates as part of their registration fee. Paper format details will be provided to all authors accepted for the Conference.

The papers should be submitted to your technical group secretary.

DEADLINES: Abstracts (50 words) - 22 August 1995
(Hard copy and floppy disk required)
Final Submissions - 4 December 1995

All enquiries to: Colin Newton
NZGS Secretary
C/o Works Consultancy Services Ltd
Tel: (04) 471 7088
Fax: (04) 473 1296

"GEOMECHANICS IN A CHANGING WORLD"

Adelaide, Australia, July 1-6, 1996

The 7th ANZ Geomechanics Conference is organized by the Australian Geomechanics Society in association with the New Zealand Geomechanics Society and endorsed by the International Society for Soil Mechanics and Foundation Engineering (ISSMFE), the International Association of Engineering Geology (IAEG) and the International Society of Rock Mechanics (ISRM).

INFORMATION

Object

This four-yearly meeting of the Australian and New Zealand Geomechanics Societies is aimed at providing a forum for geotechnical engineers and engineering geologists, active in the field through research and practice, to present and discuss their work.

Venue

The Conference will be held at the Adelaide Convention Centre, in the heart of Adelaide, the capital city of South Australia.

Technical Program

Papers are invited in the disciplines of soil and rock mechanics, engineering geology and mining. Papers from related sciences and technologies are also encouraged. The theme of the Conference is "Geomechanics in a Changing World" and authors are asked to address this theme where possible.

Sessions of the Conference shall be structured according to papers received and are expected to include:

- unsaturated soil mechanics;
- foundations and pavements;
- dams and embankments;
- earthquake engineering;
- field and laboratory testing;
- probabilistic analysis;
- urban re-development;
- mine rehabilitation; and
- professional concerns.

A wide ranging and lively specialty session in keeping with the Conference theme is planned. A panel of eminent speakers will lead the session to address issues of current concern to the profession. Speakers will be drawn from experienced practitioners having a wide range of backgrounds in Geomechanics.

PAPER PRESENTATION

To maximize the number of papers which can be accepted, theme session reporters will summarize papers for general discussion. A small selection of papers will be presented in each session. Poster sessions will be available for the authors of the remaining papers. Postgraduate research students are encouraged to attend the Conference and present papers, where appropriate.

CONFERENCE SECRETARIAT

Ms Angela Schaeffer	Mr Mark Jaksa
Conference Manager	Chairperson of the Organizing Committee
ICMS Pty. Ltd.	7th ANZ Conference on Geomechanics
Adelaide Convention Centre	Dept. of Civil and Environmental Engineering
North Terrace	The University of Adelaide
Adelaide, South Australia, 5000	Adelaide, South Australia, 5005
Telephone: 08 210 6776	Telephone: 08 303 4317
International: +618 210 6776	International: +618 303 4317
Fax: 08 212 5101	Fax: 08 303 4359
International: +618 212 5101	International: +618 303 4359
	Email:mjaksa@aelmg.adelaide.edu.au

CALL FOR PAPERS

Abstracts of less than 300 words and set in 12 pt characters, should be sent to the Conference Manager before 31 July, 1995. The names of the author(s) attending the Conference shall be underlined. Abstracts may be sent by electronic mail or facsimile, but a hard copy should follow by post.

Papers of sufficient quality will be published in the Conference Proceedings. Notification of acceptance will be given in September 1995 and will be accompanied by detailed instructions to authors for the preparation of camera-ready copies of their papers. Papers are to be submitted for review by 12 January, 1996. Contributions are to be no more than six A4 pages in length.

The Conference Proceedings will be available to participants upon arrival at the Conference. Early registrants will receive, by mail prior to the Conference, a copy of abstracts of the papers in the Proceedings.

DEADLINES

Submission of abstracts	1 July 1995
Notification of acceptance	September 1995
Submission of papers	12 January 1996
Early registration	31 May 1996

TECHNICAL EXHIBITS

A technical exhibition will be run at the conference and will allow either single day exhibiting or continuous exhibits throughout the week. More information is available upon request.

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- **1994 GEOMECHANICS LECTURE**

  SEISMIC LIQUEFACTION OF COHESIONLESS SOILS
  - John B. Berrill

  Every few years the Society honours one of its members, who has made a significant contribution to geotechnical engineering and engineering geology, by asking them to present the Geomechanics Lecture. John Berrill, a Reader in Civil Engineering at the University of Canterbury, was invited to give the 1994 Geomechanics Lecture. Geomechanics News is pleased to print John's paper from which his lecture was taken.

  John graduated from the University of Canterbury in 1963 and worked as a structural engineer in NZ and Canada for 7 years before turning to geotechnical engineering, with graduate studies at the University of Colorado and Caltech. He returned to NZ in 1977 and has been carrying out research and teaching in engineering seismology and geomechanics at Canterbury University since then.

- CAIRNMUIR LANDSLIDE INFILTRATION PROTECTION STABILISATION WORKS
  - M.D. Gillon & G.J. Saul

  This paper was recently nominated for the **Innovative Technology Award** (as sponsored by Carter Holt Harvey Packaging) for the February 1995 IPENZ Conference. One of four finalists for the award, Murray Gillon and Greg Saul received a "highly commended" for their work on restraining the Cairnmuir landslide above Lake Dunstan at the Clyde Dam.

- **FULTON-DOWNER AWARD WINNER**

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  The Fulton Downer Award is the premier award of the Institution of Professional Engineers NZ. The award was established in 1929 by a bequest from the late J.E. Fulton to which we added in 1973 and 1988, donations from A.F. Downer. The gold medal is for the person (in any class of IPENZ membership) presenting the best paper on a technical subject at an annual IPENZ conference. It is the 15th time since 1931 that a paper in geomechanics has received the annual award from amongst the whole range of engineering fields covered by IPENZ.

- EWEN BRIDGE REPLACEMENT PILE GROUTING
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- EXPERIENCE OF GEOTECHNICAL LIMIT STATE DESIGN IN RUSSIA
  - A.K. Murashev

SEISMIC LIQUEFACTION OF COHESIONLESS SOILS

John B Berrill
Department of Civil Engineering
University of Canterbury
Christchurch
NEW ZEALAND

ABSTRACT

An overview of the liquefaction problem is presented in the context of the 1964 Niigata and Alaska failures. The distinction is drawn between liquefaction flow failures and deformation failures, corresponding, in the laboratory, to the difference in behaviour of loose and dense sands in undrained tests. Methods for predicting level-ground liquefaction potential are reviewed. The CPT test is favoured over the SPT, and a pattern-recognition procedure exploiting all three piezocone sensor measurements, is presented. The problem of flow failure versus cyclic mobility is reviewed and an approach to estimating the stability of dam slopes against flow failures is sketched in outline. The success of the Newmark sliding-block method in modelling deformation failures is noted. References are given to studies of a number of other aspects of liquefaction not covered in the review.

INTRODUCTION

Liquefaction of fine-grained, cohesionless soils has been a major cause of damage in earthquakes. Liquefaction damage has occurred either directly, through failure of foundations, slopes and embankments, for example, or indirectly through damage to lifelines. Loss of water for fire fighting, leading to uncontrolled spread of fire is an unfortunately common example of indirect damage. Although evidence of liquefaction can be found in most major historical earthquakes, and research into both its causes and effects had been pursued well before 1964, it was the Niigata, Japan and Anchorage, Alaska earthquakes of that year which compelled widespread recognition of the importance of liquefaction and gave impetus to the large amount of research that followed and continues to the present day.

The city of Niigata is located on the west coast of Japan and if founded on 30 m or so of alluvial sand deposited by the Shinamo River. The M7.5 earthquake of 16 June, with epicentre about 50 km offshore to the north, caused extensive liquefaction in loose sands, especially in low-lying fill and old river channel material along the lower reaches of the Shinano River. Damage included the settlement and tilting of buildings, with some structures settling by a metre or more and tilting several degrees off vertical. In one case an apartment building tilted as much as 80° as soil beneath its foundation liquefied. Lateral spreading on shallow slopes of just a few degrees caused widespread damage to buried services and to bridge and building foundations.

Lateral displacement of the piers of the Showa Bridge, for example, caused five simply supported spans to fall. Other bridges suffered less dramatic but nevertheless important damage. Lateral spreading also caused severe damage to embankments and to railway yards. Light-weight buried structures floated upwards in liquefied sands. Settlement resulted in inundation of already low-lying areas, and the ejected sand itself proved to be a great nuisance and hindered recovery operations.

The M8.4 Alaska Earthquake of 27 March, 1964 caused several landslides in and around the City of Anchorage. While there is some doubt about the role of liquefaction in the large Turnagain Heights slide which occurred principally in clay soil, there were several lateral spreading failures that have been attributed to liquefaction of sands as well as flow failures due to liquefaction, most notably, the Potter Hill slide (Long, 1973). Numerous bridges were damaged in the Alaska earthquake. The class of bridges suffering the greatest damage were those founded on piles driven into loose to medium dense fine sands and coarse silts. Here, lateral spreading of foundation soils towards stream channels caused displacements of abutments and pier structures, failure of piles, and settlements, often damaging the bridge superstructure as well. Spreading of approach fills was also common. Much of this damage was attributed to liquefaction of fine sands (Ross et al., 1973). Bridges on piles driven into medium or coarse sands suffered considerably less damage, while those founded directly on rock were hardly damaged at all.

The damage seen in 1964 at Niigata and in Alaska illustrates the principal classes of liquefaction problem and brings out the importance of soil density and particle grading.

Let us review some fundamental concepts. Firstly, under cyclic loading in shear, cohesionless material tends to decrease in volume [provided a small shear strain threshold of about 0.01 percent is exceeded (Dobry et al., 1980)]. This tendency to decrease in volume is much greater in loose than in dense soils but, nevertheless, it is present also in dense soils. When the soil is saturated and drainage of pore water is prevented, the tendency to volume decrease under cyclic loading results in an increase in pore pressure. In the laboratory, this effect can be obtained by carrying out undrained tests on saturated samples of cohesionless soil; in the field, drainage is impeded naturally in fine-grained soils. Thus cohesionless soils with low permeability, such as fine sands and silts, exhibit pore pressure increase under seismic loading, and are the most susceptible to liquefaction. Ranges of most critical gradings, obtained from field studies by Tsuchida and Hayashi (1971), are shown in Figure 1.

Laboratory tests on loose sands show that pore pressure increase occurs rapidly, and that pore pressure can approach effective confining pressure in a few cycles of cyclic loading. In a dense specimen of the same sand, many more cycles of a generally greater amplitude are required to produce a condition of *initial liquefaction*, where pore pressure u equals initial effective confining pressure σ_o'. Furthermore, with the loose sand, u remains near σ_o' during subsequent cycles of loading, and consequently, effective confining stress, and hence strength, becomes and remains small. Whereas in dense sand, u approaches σ_o' only momentarily (twice) during a loading cycle. During the remainder of the cycle effective confining stresses are significant, and hence considerable

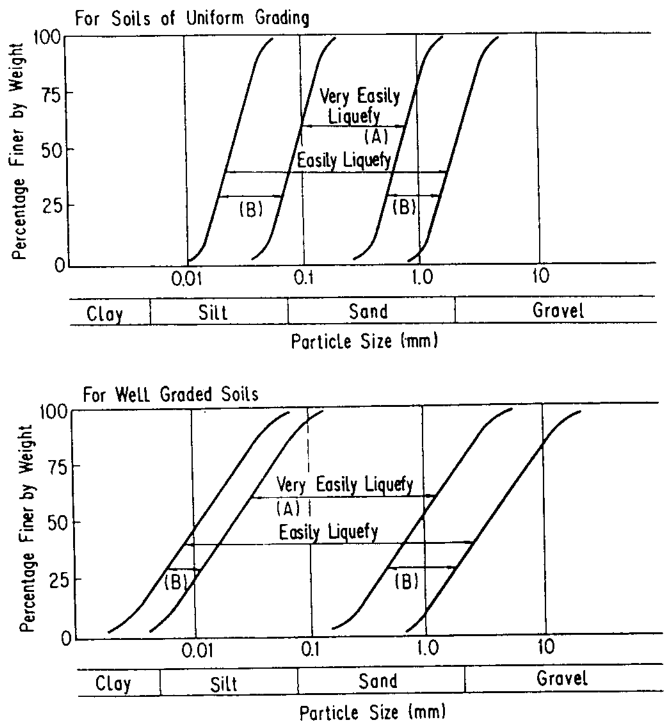


Figure 1. Tsuchida's curves for grading ranges of liquefiable soils (from Iwasaki, 1986)

strength remains. These two different behaviours are illustrated in Figure 2, taken from Ishihara (1985), showing the results of undrained cyclic torsional shear tests on a uniform, medium sand.

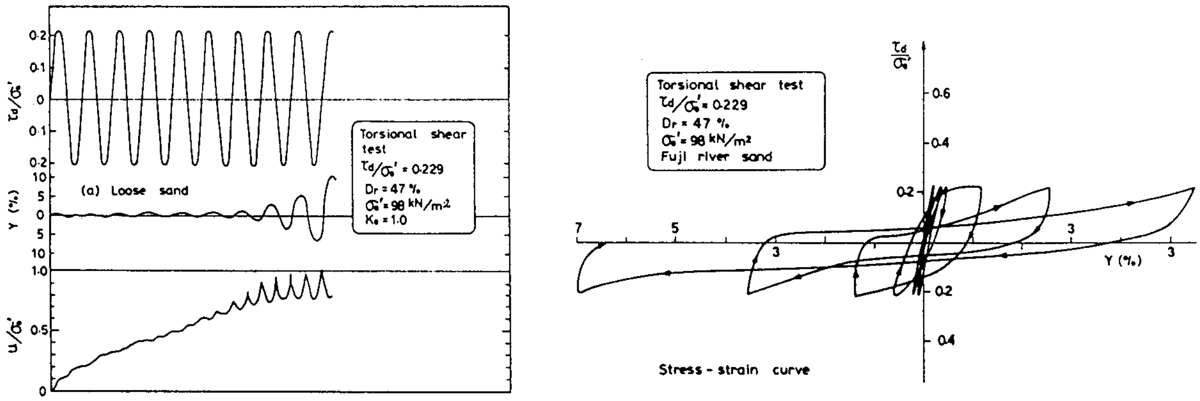
A very rapid build-up of excess pore pressure and subsequent loss of strength together with the development of large strains are characteristics of seismic liquefaction of loose sands. The behaviour of the dense sand in which strain amplitudes build-up in small increments, is termed *cyclic mobility*. In the field, liquefaction of loose sands can lead to flow failures of slopes and large displacements of foundations. Cyclic mobility, on the other hand, results in limited soil displacement, seen for example in the limited settlement and deformation of some earth dams during earthquakes.

Thus liquefaction problems (using the term "liquefaction" in the broad sense) fall into two classes, depending on whether the soil is loose or dense. The seismic behaviour of a soil mass also depends on whether or not shear strength is required for static equilibrium. For example, a sand deposit with a level ground surface may loose its shear strength entirely yet still remain in static equilibrium. On the other hand, some shear strength is always required to maintain a slope or a loaded foundation in static equilibrium.

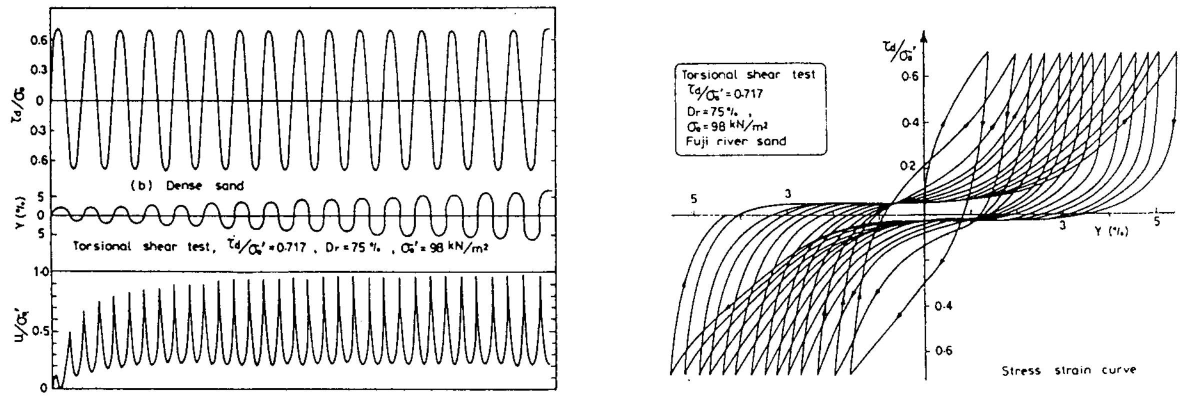
The case of liquefaction of loose deposits in level ground has been widely studied, and there are many procedures for estimating whether or not a particular deposit is likely to liquefy in a given earthquake. Some involve laboratory testing, others in situ testing.

The widely-used technique of Seed and Idriss (Seed and Idriss, 1971; Seed et al. 1985) is an example of the latter. Procedures for solution of other cases are not as well established, and are still largely in the province of research. For the case of flow failures in loose sands, the work of Castro et al. (1985) and Dobry et al. (1984) offer procedures for estimating whether or not a slope is stable. Bartlett and Youd (1992) present a method for estimating lateral spreading distances, and the work of O'Rourke and Pease (1992) allows likely damage to buried pipes to be estimated.

Tsuchida's (1970) grading curves point to the general importance of geological considerations. Loose, fine-grained sediments are deposited only under certain geologic conditions, and resistance to liquefaction increases with age, as weathering, cementation and, certainly, other processes cause the *fabric* of a soil to develop. Tinsley and Dupré (1992) find very clear correlations between geology and liquefaction effects in the Monterey Bay area during the 1989 Loma Prieta earthquake. They note that laterally accreted structures such as point-bar formations are especially susceptible to liquefaction, and the effect of age was very evident, with lateral spreading, for example, being restricted to late Holocene deposits.



(a) Loose sand (D_r = 47%)



(b) Dense sand (D_r = 75%)

Figure 2. Results of cyclic torsional shear tests on (a) loose and (b) dense specimens of Fuji River sand (From Ishihara, 1985)

In the remainder of this paper, I intend to discuss these basic aspects of the liquefaction problem in more detail and outline some of the methods of solution proposed.

LEVEL-GROUND LIQUEFACTION

Although liquefaction of level ground does not cause as great a threat to life and limb as flow and bearing capacity failures, it is responsible for very costly material losses, chiefly through damage to buried pipelines, but also through damage to pavements and other surface works. The main surface manifestation of the liquefaction of an underlying layer is the formation of sand boils, in which sand and water are ejected through fissures or circular vents, leaving shallow cones of sand on the ground surface. Scott and Zuckerman (1973) have studied the formation of sand boils in the laboratory. Their experiments indicate that a layer of finer grained soil overlying the liquefiable soil is necessary for the formation of sand boils. The vent is formed by the upwards enlargement of a cavity which begins at the base of the upper layer as it unravels in an unstable, localised fashion into the underlying liquefied sand-water mixture. Once the vent has broken through to the surface, sand and water are ejected, driven by the hydraulic gradient generated as the liquefied layer carries the full weight of overburden by fluid pressure.

Florin and Ivanov (1961) noted that, given uniform density, liquefaction begins at the top of a layer and propagates downwards. They also observed that the tendency to liquefaction decreases with increasing overburden or confining pressure. This observation has been made independently by Seed and Lee (1966) and by many other researchers.

Both Scott and Zuckerman and Florin and Ivanov also observed that subsequent solidification (the opposite of liquefaction) begins at the bottom of the liquefied layer and proceeds upwards, as particles settle out of suspension. Scott and Zuckerman also observed that a denser granular layer overlying a loose layer may be induced to liquefy as support for its solid skeleton is lost as the underlying layer liquefies, even though this layer would not liquefy on its own. They term this *secondary liquefaction*. Here, a liquefaction front propagates upwards. Some of the Niigata foundation failures have been attributed to secondary liquefaction, provoked by the initial liquefaction of a deep layer.

In general, an intact surface layer overlying a liquefied stratum is floating on a fluid of quite similar density to its own. It therefore has a tendency to sink to establish equilibrium. Any departure from uniform density, thickness or surface loading will tend to induce bending stress in the surface layer which, if brittle, may crack. Fissures and cracks are commonly associated with level-ground liquefaction, and are the cause of great damage to buried pipes. Youd (1984) terms this disruption *ground oscillation* (as blocks of cracked surface material move relative to one another) and states that ground oscillation together with lateral spreading have caused more property damage in earthquakes this century than flow and bearing capacity failures.

Prediction Procedures

Procedures for predicting liquefaction potential of level ground sites fall into two classes. Those based on laboratory testing of field specimens, and those based on in situ testing. Because of the difficulty in obtaining undisturbed samples of cohesionless soils, methods based on in situ test results have become more common, and we will focus on them. These methods are all, to some degree, empirical.

Whether or not a site will liquefy in an earthquake depends both on the strength of shaking (seismic loading) at the site and on the state of the soil. The seismic loading can be characterised in a *local* fashion for example, by peak acceleration. a_{max} or modified Mercalli intensity I at the site, or by a *source* description using, for example, magnitude M and epicentral distance, r_e. Liao, Veneziano and Whitman (1988) have made a rigorous evaluation of both types of model and conclude that in the study of a specific site, local characterisation fits case history data better. But the advantage is lost when the value of the local ground motion parameter must be estimated separately by an empirical attenuation expression. Thus, for regional hazard mapping, say, it is better to go straight to a source-type liquefaction model.

By tradition in the US, Japan, and many other seismic countries, the state of cohesionless soils has been characterised in situ by the Standard Penetration Test (SPT). This is somewhat fortuitous, since the SPT N-value is related to the relative density which, together with effective confining pressure, provides a measure of the tendency of the soil to dilate or contract under shearing. However, the SPT has some serious disadvantages, discussed later, especially in loose sands and silts. Furthermore, a lack of adequate standardisation has led to a wide range of hammer efficiencies and the need for corrections to a reference efficiency, usually 60% (Seed et al., 1985). Correction for overburden pressure is also required if the N-value is to correspond to soil density. Liao and Whitman (1986) review the overburden correction problem and recommend correcting the measured value of N to a value, N_1, normalised to 1 ton/sq.ft (100 kPa) by the formula

$$N_1 = C_N N \tag{1}$$

where

$$C_N = \sqrt{100/\sigma_o'} \tag{2}$$

and where σ_o' is effective overburden stress in kPa.

Seed's Procedure

The first and still most widely-used procedure for evaluating whether a site is likely to liquefy is that of Seed and Idriss (1971), which has been modified successively over the years. A local characterisation of the earthquake loading is employed.

Seismic loading on the soil layer is characterised by an average cyclic stress ratio τ_{av}/σ_o' given by the expression:

$$\frac{\tau_{av}}{\sigma_o'} = 0.65 \frac{a_{max}}{g} \frac{\sigma_o}{\sigma_o'} \frac{r_d}{C_M} \tag{3}$$

where τ_{av} is equivalent average shear stress, σ_o is total overburden stress, a_{max}/g is peak ground acceleration as a fraction of g, r_d is a factor to account for soil flexibility, and C_M is a magnitude correction factor.

This expression is derived by considering the equilibrium of a rigid soil column under a horizontal acceleration a_{max}. The factor of 0.65 allows for an average shear stress somewhat less than the peak stress corresponding to peak acceleration. Allowance for flexible rather than rigid response of the overlying soil mass is made by the term r_d, which is conveniently obtained in Japanese practice (Iwasaki, 1986) from the expression:

$$r_d = (1 - 0.015z) \tag{4}$$

where z is depth in metres. The factor C_M takes the value unity for M = 7.5 (reflecting the procedure's origins in Niigata data); its value for other magnitudes may be found from Table 1 below. Originally, Seed and Idriss applied the C_M factor at a later stage; we follow Liao et al. (1988) in incorporating it in the loading expression, since it does indeed represent a loading effect.

Table 1 Magnitude Correction Factor C_M

Earthquake Magnitude	Number of Representative Cycles at 0.65 τ_{max}	Correction Factor C_M
8.5	26	0.89
7.5	15	1.0
6.75	10	1.13
6.0	5-6	1.32
5.25	2-3	1.5

The soil is characterised by its SPT N-value, corrected for overburden pressure using equation (1) and to a hammer efficiency of 60%, using the information in Table 2. The resulting corrected SPT blow count is denoted by the symbol $(N_1)_{60}$.

The final step in the procedure is to check whether or not the seismic loading given by equation (3) exceeds a threshold value obtained for that soil state $(N_1)_{60}$. The threshold

Table 2 Summary of Energy Ratios for SPT Procedures (from Seed et al., 1985)

Country	Hammer Type	Hammer Release	Estimated Rod Energy (Percent)	Correction Factor for 60 Percent Rod Energy
Japan[a]	Donut	Free-fall	78	78/60 = 1.30
	Donut	Rope and pulley with special throw release	67	67/60 = 1.12
United States	Safety	Rope and pulley	60	60/60 = 1.00
	Donut[b]	Rope and pulley	45	45/60 = 0.75
Argentina	Donut	Rope and pulley	45	45/60 = 0.75
China	Donut	Free-fall[c]	60	60/60 = 1.00
	Donut	Rope and pulley	50	50/60 = 0.83

[a] Japanese SPT results have additional corrections for borehole diameter and frequency effects.
[b] Prevalent method in the United States today.
[c] Pilcon-type hammers develop an energy ratio of about 60 percent.

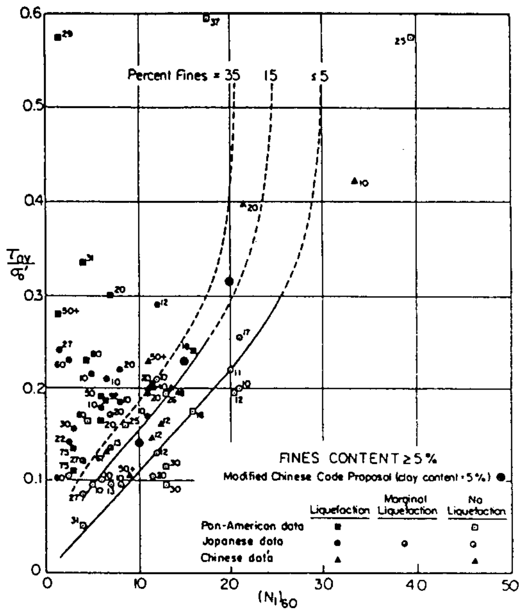


Figure 3. Relationship between Stress Ratios Causing Liquefaction and $(N_1)_{60}$ Values, from Seed et al. (1985)

value of $\tau_{av}/\sigma_o^{'}$ is found in the chart reproduced in Figure 3, derived empirically from liquefaction case histories (Seed et al., 1985).

Figure 3 contains three curves, for different fines contents. In their appraisal of liquefaction prediction methods, Liao et al. (1988) investigated how well their comprehensive set of case history data supports such a marked influence of fines content. They conclude that while the presence of more than a moderate percentage of fines does have an effect on probability of liquefaction, the effect is not nearly as marked as Figure 3 indicates. The data does not suggest a progressive increase in resistance with increasing fines content. It does, however, support a division into two classes: *clean sand* and *silty sand*, with a fines content of 12% as the dividing line. Liao et al. point out that the curves for soils with fines in Figure 3 are based on laboratory tests on specimens at constant relative density, D_r, not at constant $(N_1)_{60}$. The field data in terms of $(N_1)_{60}$ does not show such a strong effect. Liao et al. note further than when seismic loading is represented by magnitude and distance rather than peak acceleration at the site, the uncertainties associated with attenuation overwhelm the distinction between clean and silty soil. Thus when a source characterization of the earthquake is employed, it is not worth the trouble of distinguishing between silty and clean sand deposits.

Finn (1992) raises a further point with regard to the effect of fines. The greater liquefaction resistance for silty sands implied in Figure 3 and observed by Liao et al., is seen when comparisons are made *at similar values of* $(N_1)_{60}$. However, Troncoso (1990) found that cyclic strength decreased with increasing silt content when he compared samples at constant void ratio. Furthermore, Kuerbis and Vaid (1989) tested a particular sand at constant sand-skeleton void ratio. This sand-skeleton could accommodate up to 20% fines. They found that for fines contents of less than 20%, the specimens had the same cyclic strength. Finn observes that from another point of view, these results imply that the penetration resistance of a silty sand is somewhat less than that of a clean sand with the same cyclic strength.

Energy Dissipation Approach

When good estimates of ground motion intensity are not available for the site, it is more appropriate to use a procedure based on magnitude and distance to the earthquake source. One such model is that of Davis & Berrill (1982), which performed well amongst a number of procedures of that type tested by Liao et al. (1988). The model is based on the suggestion of Nemat-Nasser and Shokooh (1979) that pore pressure increase is proportional to the density of seismic energy dissipated.

Its derivation, which uses well-established results from seismology and soil mechanics, and seeks to keep empirical steps to a minimum, proceeds as follows. Combining the expression of Gutenberg and Richter (1954) for total radiated energy with a simple geometric spreading rule, yields the density of seismic energy arriving at the site. Hardin (1965) found that energy dissipation is proportional to $(\sigma_o^{'})^{-1/2}$. Using this result and the assumption that Δu is proportional to the density of dissipated energy yields the following expression for seismic pore pressure increase:

$$\Delta u = \frac{\lambda(N_1)\ 10^{1.5M}}{r^2\ (\sigma_o')^{1/2}} \tag{5}$$

where λ is an unknown function of the corrected SPT value N_1, characterising the state of the soil. The function $\lambda(N_1)$ is then found from case history data using linear discriminant theory to give the final result:

$$\Delta u = \frac{450\ 10^{1.5M}}{r^2\ (N_1)_{60}^2\ (\sigma_o')^{1/2}} \tag{6}$$

Here, r is expressed in metres, σ_o' in kPa, and $(N_1)_{60}$ is substituted for N_1 which was used in the original derivation. Its simple functional form makes equation (6) particularly suited to probabilistic hazard analysis, and an example is worked in the original paper. Liao et al. (1988) observe that this model does not perform well with respect to their data set for dense sands, and caution against using it for $N_1 > 20$.

Liao et al. themselves devise an expression for probability of liquefaction P_L which employs the same seismic loading term

$$\Lambda_e = \frac{10^{1.5M}}{r_e^2 \left(\sigma_o'\right)^{3/2}} \tag{7}$$

but which fits their larger and more complete data set better. This expression is

$$P_L = 1/\{1 + \exp[12.922 - 0.87213\,\ell n(\Lambda_e) + 0.21056\,(N_1)_{60}]\} \tag{8}$$

where r_e is epicentral distance. They present a corresponding expression in terms of hypocentral distance, which fits the data a little better.

Remarks on the SPT

The shortcomings of the SPT for use in liquefaction analyses have been discussed at length in the literature, especially in connection with energy standardisation (Seed et al., 1985; Liao and Whitman, 1985, for example). Apart from the problem of standardising energy input, the test has two major difficulties when employed in loose sands and silts. The first arises from the discrete nature of the blow count. For a perfectly executed test yielding a blow count of 5, for example, the resolution of the discrete scale is no better than ± 10%. With N then raised to the power 2, as in equation (6) for example, an even greater uncertainty is introduced. The second objection comes from the difficulty in obtaining a clean drill hole, without disturbing the material in the test region at the bottom of the boring. It has been the writer's experience that even with careful rotary boring with mud, it is often difficult to avoid disturbance at the bottom of the hole.

Even experienced drillers operating under research conditions have had difficulty in very loose sands. The writer is quite sceptical of N-values of much less than 6.

These considerations, together with the problem of energy standardisation, have led us to adopt the cone penetration test (CPT) in our studies of liquefaction sites in New Zealand. Not only does the CPT offer more precision, but also it is repeatable and gives a continuous measurement of soil resistance rather than the discrete measurements of the SPT, spaced at 1 m or greater centres.

Since there is a well established relationship between them, CPT cone resistance values, q_c, can be converted to SPT N-values for use in SPT-based procedures such as the two described above. The q_c-N correlation of Robertson & Campanella (1985), shown in Figure 4, is widely used for this purpose.

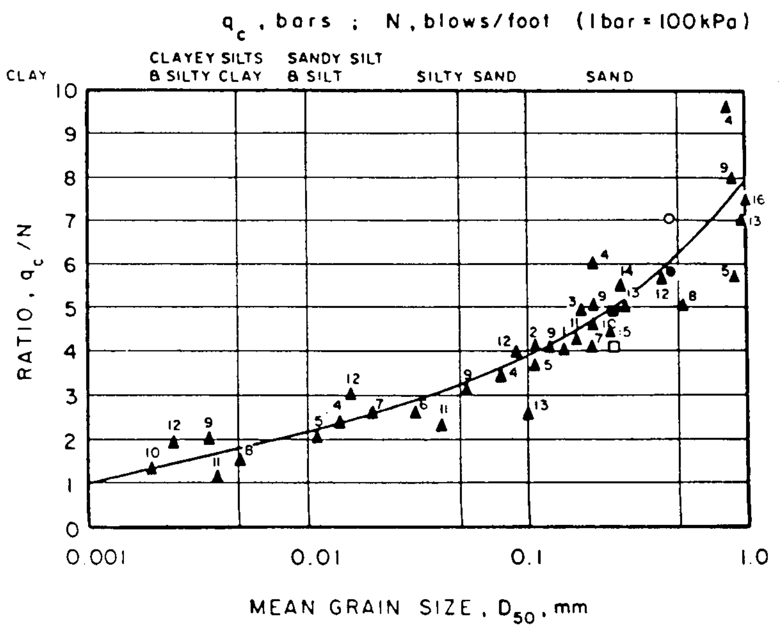


Figure 4. Variation of q_c/N with mean grain size (from Robertson and Campanella, 1985)

Cone Penetration Methods

Some procedures originally based on SPT have been converted to CPT using the q_c-N relationship. Other procedures have been formulated directly in terms of q_c, using CPT field data. These include the method of Shibata and Teparaska (1988) and the well-known Chinese method (Zhou, 1980). These procedures benefit from the greater precision and sensitivity of the CPT, but they do not fully exploit its potential to estimate grain size and, in the case of the piezocone test (CPTU), drainage conditions.

In an attempt to better exploit the diagnostic capability of the piezocone, Dou and Berrill

(1991, 1993) have employed the pattern recognition technique from information theory, together with case history data, to develop a procedure for estimating the probability of liquefaction, using all three CPTU measurements.

In the most recent implementation of the system, the state of a soil layer under seismic load is represented by a point in a 7-dimensional measurement space. The coordinates of this space comprise the three CPTU measurements (q_c; friction ratio, R_f; pore pressure, u), together with excess pore pressure, overburden pressure and penetration rate, as well as cyclic stress ratio characterising the seismic load. Penetration rate is a significant parameter since it contributes to the magnitude of excess pore pressure (Canou, 1989; Berrill et al. 1992). The pattern recognition procedure then classifies the layer (for the given loading) into one of the three classes: liquefiable soil, non-liquefiable cohesive soil and non-liquefiable cohesionless soil. Figure 5 shows the results for a site which has liquefied in two recent earthquakes.

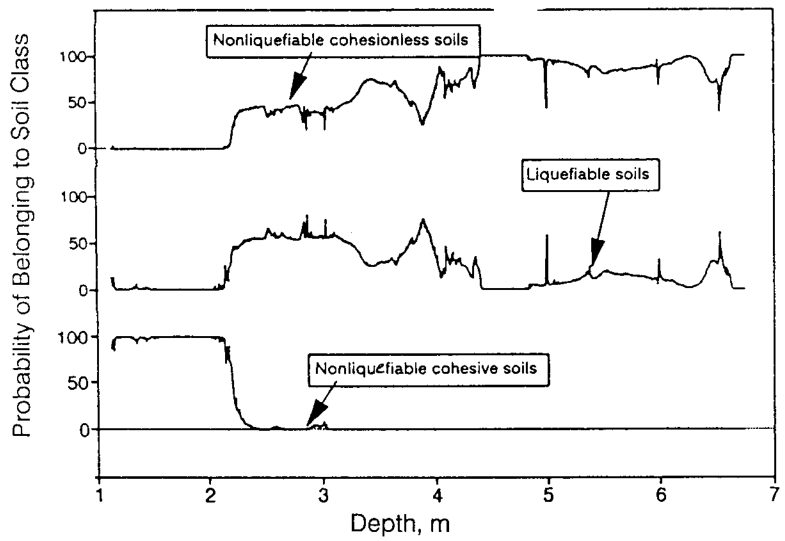


Figure 5. Result of pattern recognition computation for a site that liquefied in the M7.1 1968 and M.0 1991 earthquakes near Westport, NZ.

The system is completely empirical, and functions as follows. First, the vector of 7 measurements, representing the state of a layer whose liquefaction potential we wish to estimate, is transformed into *feature space*. Feature space comprises orthogonal co-ordinates which are derived from a *training* (calibration) data set in such a way that they enhance useful information in the measurements and maximise the separation between the three data classes. The second step is to compute the probability that the transformed data point belongs to each of the three possible classes (liquefiable, non-liquefiable cohesive, non-liquefiable cohesionless). This is done by comparing its position in feature space with probability distributions for the three classes, determined from the training data.

Using a training set of CPTU data from sites that have liquefied or not in two recent earthquakes in the Buller region of New Zealand, a recognition error rate of less than 8 percent has been obtained for independent data. While the procedure needs to be calibrated against a broader training set before being used routinely, the success of this preliminary work is encouraging. It is noted that pattern recognition could be a powerful technique for the interpretation of data in other geotechnical contexts besides liquefaction.

SUSTAINED SHEAR STRESS

In the case of level ground liquefaction discussed above, shear strength is not required for static equilibrium. But in most other cases (retaining-wall backfill is a possible exception), the soil mass must resist sustained shear stresses to remain in equilibrium. Slopes and foundations are two common examples.

The behaviour of the soil mass depends, in the large, on whether it is loose and contractive or dense and dilative. If the soil is dense, then any perturbation by the earthquake will cause it to dilate, thereby increasing its undrained, or short term, strength. A typical static undrained stress strain curve for a dense sand is shown in Figure 6. At large strains, a steady state of deformation is reached at which shear strain continues at a constant shear stress. This undrained steady state shear strength has been denoted by S_{US}. A characteristic of a dense granular soil is a large value of S_{US}. On the other hand, loose soils have static, undrained stress-strain curves typically like that illustrated in Figure 7. In this case the curve drops off with increasing strain to a relatively small value of S_{US}.

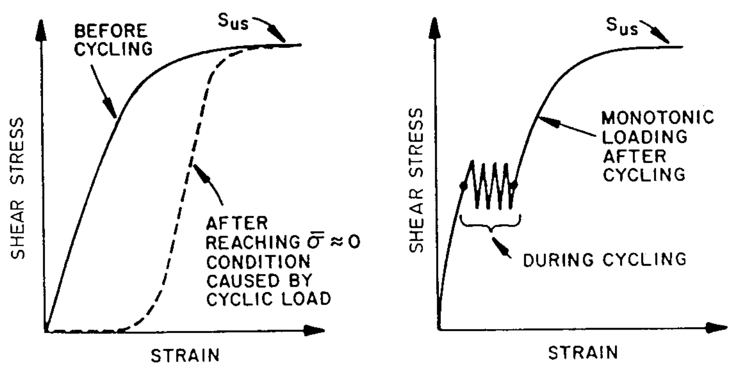


Figure 6. Stress-strain behaviour for undrained loading of dense sand. From Whitman (1987).

Clearly, a slope or foundation soil composed of dense material should remain stable under seismic loading (provided the soil remains undrained). On the other hand, the stability of a loose soil mass depends, to a first analysis, on whether or not the steady state strength exceeds the static driving stress, τ_s. We will now examine these two cases in more detail.

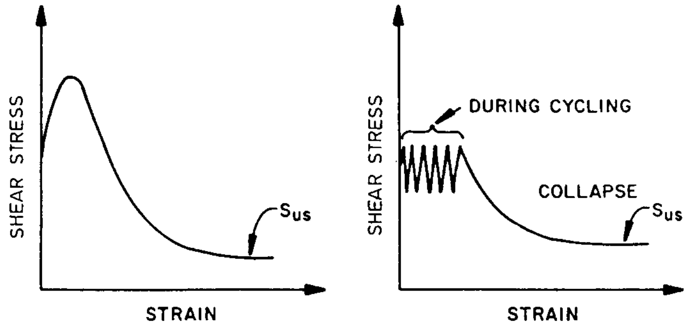


Figure 7. Stress-strain behaviour for undrained loading of loose sand. From Whitman (1987)

Liquefaction Flow Failures

Dobry et al. (1984) note that in liquefaction failures involving flow of material and large displacements, there are two consecutive stages. The first comprises the build up of pore pressure, depending mainly on the amplitude and duration of cyclic shear strain γ_c, induced by the earthquake. The second stage comprises flow driven by the static shear stresses τ_s, and proceeds only if $\tau_s > S_{US}$. Thus to analyze the stability of a slope or foundation against flow failure, we need first to check the static stability, using appropriate values of S_{US} in regions of contractive soil. If the structure is stable under these conditions, the analysis can stop there. However, if it is not, then it is necessary to check whether the build up of pore pressure which triggers the strength reduction, will indeed occur under the design earthquake. Dobry et al. present this approach for earth dams, and give details of how it might be applied to the dam problem. Their procedure includes a novel laboratory test in which a torsional cyclic shear stress is applied to an undrained triaxial specimen which has been consolidated and is maintained under an anisotropic stress system representing the static in situ stress state, simulating the two aspects of the problem. However, this general approach could equally well be applied to the seismic stability of shallow foundations and to static liquefaction flow failures such as the Nice Airport failure (Schlosser et al., 1985).

While the approach is simple in concept, the determination of in situ values of S_{US} is far from trivial. Laboratory determination of S_{US} depends on the very difficult task of obtaining "undisturbed" samples. Poulos et al. (1985) describe a procedure they have developed over the years. It involves undrained tests on both field and reconstituted samples, with an allowance for sample disturbance. Their method is based on the premise that for a given soil S_{US} depends uniquely on void ratio, e. But Vaid et al. (1989), for example, find that preshearing has a marked effect on the dilatancy behaviour of sand and thus on S_{US} and Konrad et al. (1991) question the uniqueness of the steady-state line.

Seed (1987) and Seed et al. (1988) have presented correlations between corrected SPT values, $(N_1)_{60}$, and values of S_{US} obtained by back analyses of flow failures. However, Finn (1992) points out that there are difficulties with this approach, and it suffers from the general shortcomings of the SPT, mentioned above. Because flow failures usually involve quite loose and often silty materials, the CPT should be the more appropriate in situ test. Ishihara et al. (1990) present correlations between q_c and S_{us} obtained by back analyses of flow failures in Japan.

Deformation Failures

In a dilating rather than contractive soil, permanent displacements may occur during momentary strength reductions, as discussed in the introduction, but these are intermittent, of limited magnitude, and cease when the shaking stops. While permanent displacements in dilating soils are limited, they may still be large enough to impair the function of the structure. Whitman (1987) terms such occurrences *deformation failures*, and states that their analysis presents "one of the present-day frontiers of soil dynamics". Whitman discusses some computational procedures that have not yet become part of every-day engineering practice. He emphasises the need to test computation against experiment, and suggests the use of centrifuge model tests because of the infrequency of earthquakes for full scale tests. Use of the Newmark (1965) sliding block analogy has been suggested for the calculation of limited, permanent displacements. This suggestion has been taken up by, among others, Baziar, Dobry and Alamo (1992) who study lateral spreading at the Wildlife, California site in the 1987 earthquake, and by Byrne, Jitno and Salgado (1992) who apply it to the upper San Fernando dam in the 1971 earthquake. The success of both modellings suggest that this is a fruitful approach.

Intermediate Cases

We have considered the two extreme cases where the soils were either clearly contractive or clearly dilative, and completely undrained. Partial drainage could lead to a reduction in S_{US} in a dilating soil. For a soil mass that was in equilibrium before the earthquake, partial drainage should only pose a problem during shaking and then only if S_{US} drops below the sum of the seismic and static shear stresses. Here, because the seismic component of shear stress is cyclic, displacements should remain limited.

Whitman (1985, 1987) points out two other subtle variations to the simple cases. The first occurs when a cohesionless soil remains globally undrained, but undergoes local changes in void ratio which cause a loss in strength and thus, possibly, a flow failure. The second concerns high excess pore pressures generated in a non-critical region, which lead to a critical loss of strength when they diffuse into a more sensitive region. This could explain several delayed flow failures that have been observed.

CONCLUSION

In this brief paper it has been possible to cover only a few of the significant results about liquefaction, and there have been a number of omissions. I should have liked to discuss results from centrifuge tests, such as those of Lambe and Whitman (1981), Hushmand, Scott and Crouse (1988) and Lin and Dobry (1992). Centrifuge tests on shallow foundations show that smaller excess pore pressures tend to develop beneath foundations than in the free field. This implies that if a reliable level-ground analysis shows there should be no significant pore pressure increase in the free field, then any foundations should be safe, too.

The empirical study of lateral spreading distances by Bartlett and Youd (1992) should be described, as should the work of Tokida and his colleagues on drag loads imposed on piles (Tokida et al., 1993). Another topic of great practical importance is the susceptibility of lifelines to liquefaction damage. The case histories assembled by Hameda and O'Rourke (1992) and O'Rourke and Hamada (1992) should serve as an introduction to current work in this field.

The main points made in this review may be summarised as follows:

1 Damage due to liquefaction effects has been extensive and costly in past earthquakes.

2 Fine sands and coarse silts are the most susceptible soils to liquefaction. From a geological viewpoint, laterally-accreted late Holocene deposits are particularly liable to liquefy.

3 Liquefaction problems can be separated into two classes: Those where static shear stresses must be sustained, as in slopes or foundations, and those involving level ground, where static equilibrium does not require any shear strength.

4 Procedures for predicting the liquefaction potential of level-ground sites are well-established and a number of methods were presented.

5 In cases where shear stresses must be resisted for static equilibrium, behaviour after *initial liquefaction* depends on whether the residual strength is sufficient to resist the driving stress. This in turn depends principally on whether the soil is in a dense or a loose state. If the soil is loose, then a flow failure, with large displacements, is likely. If it is dense, then deformation should be limited, but may still be damaging.

6 Determination of the residual or *steady state* shear strength is a very difficult problem because of sample disturbance. Simple methods for measuring S_{US} have not yet been found. For rough, preliminary assessments, in situ test methods may be used.

7 The Newmark sliding-block approach appears promising for estimating displacements in limited-deformation problems.

8 Between the two extreme cases of liquefaction flow failures and deformation failures, there are many intermediate cases influenced by secondary effects such as partial drainage, diffusion of excess pore pressure and redistribution of void ratio.

Great progress has been made in our understanding of liquefaction phenomena, in understanding the mechanics of the various aspects of the general problem and in formulating analysis procedures. However, further work is required, for example, in finding more robust methods for determining steady-state strength. Finally, let us note the importance of case histories, which have played a central part in the past. They will continue to be important, with emphasis directed more to flow and deformation failures than to level ground cases.

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CAIRNMUIR LANDSLIDE

INFILTRATION PROTECTION STABILISATION WORKS

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Abstract

Cairnmuir Landslide is one of a number of landslide areas which have required stabilisation as part of the Clyde Power Project. It is an active schist rock slide, with a volume of 8 million m^3, located above Lake Dunstan, the Clyde Dam reservoir. Whole slide velocities of up to 180 mm/year have been measured.

The slide was treated with underground and limited surface drainage works prior to the filling of Lake Dunstan. Continued movement of the slide necessitated more extensive work, including surface infiltration protection over a steep broken area of 3.8 ha in the toe region of the slide. Development of a practical engineering solution required an innovative approach which integrated a variety of technical features. The main infiltration protection was provided by polymer modified bitumen sealing of terraces formed between steel mesh faced reinforced earth walls. A 0.8 ha area of steep slope below the terraces and two drainage gullies were lined with geomembrane held in place by draped rock filled mattresses supported by cables.

The innovative solutions adopted allowed a wide range of materials to be manufactured on site, minimised the import of materials and enabled construction to be completed in 7 months. Reinforced earth technology provided ease of construction, flexibility to follow ground contours, deformation tolerance and a natural appearance. The combination of technologies used to form the sealed surfaces is believed to be unique in this application.

1 Introduction

Cairnmuir Landslide is one of a number of landslide areas which required stabilisation measures as part of the Clyde Power Project development (Brown, Gillon & Deere 1993; Gillon and Riley et al 1992). The landslide is located on the right bank of the Lake Dunstan reservoir, 15 km upstream of the Clyde Dam (Figure 1).

A portion of the landslide is active and slide movement in response to rainfall was observed. No improvement in stability is expected with slide deformation because the failure surface exits on the steep toe slopes, 50m above lake level. The volume of the active portion is sufficient to block the reservoir and rapid failure of the slide could form a reservoir wave higher than the normal free board at the Clyde Dam.

An initial stage of remedial works was implemented to isolate the slide from the potential effects of lake filling. Drainage tunnels and subsurface drainage drilling were installed prior to lake filling which effectively drained and controlled groundwater systems below the slide but

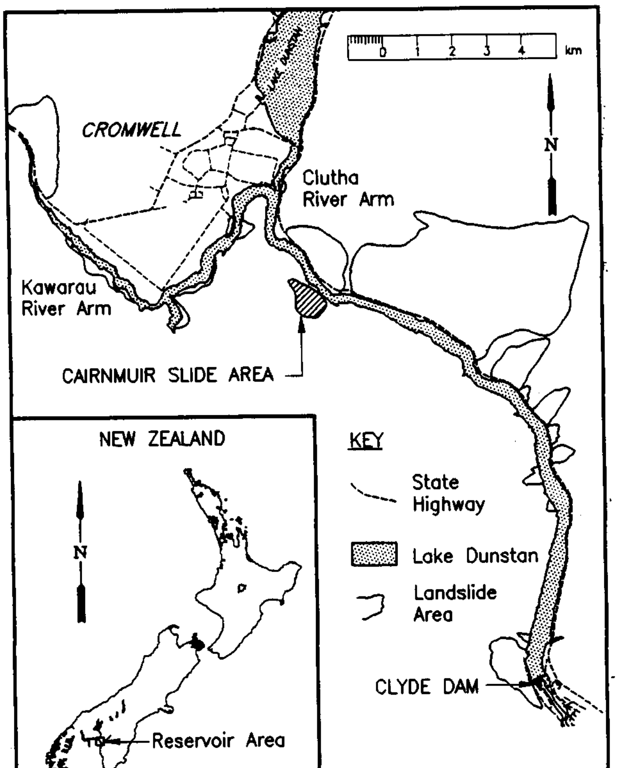


Figure 1 LOCALITY PLAN

had less effect on water perched on the failure surface of the slide. Limited surface drainage works were carried out including the formation of some surface run off interception channels and filling of sinkholes. Slide deformation response to rainfall events continued after the initial stabilisation works and necessitated a second stage of remedial works involving surface works to limit infiltration and more intensive subsurface drainage to minimise perched groundwater seepage into the sensitive toe region of the slide. The surface stabilisation works are the subject of this paper.

The design and construction of drainage and infiltration protection measures on the slide required an innovative approach, in order to develop a safe practical and economic engineering solution, which was tolerant of slope movement and had a low visual impact.

2. Slide Description

The slide is a relatively planar rock slide with a 500m wide and 650m long, active portion, covering an area of 28 hectares and comprising 8.3 million m^3 of chaotic debris (Figures 2 and 3). The inclination of the slide surface varies from 20 degrees mid slope to more than 35 degrees in the head and toe.

The slide mass is up to 60m deep and comprises chaotic mica schist landslide debris (Figure 3). The main active failure surface is a discrete, slickensided sandy silty clay gouge, 100 to 300mm thick, and is located at the top of a Basal Failure Zone. Multiple failure surfaces within the landslide debris correlate with surface scarps in the Frontal Lobe Area (Figures 2 and 3).

Prior to drainage, groundwater was both perched on and confined beneath the Basal Failure Zone. Low permeability crushed zones and the Frontal Lobe failure surfaces acted to compartmentalise groundwater in the slide debris while tension zones and sinkholes on the surface provided permeable paths for infiltration.

3. Slide Movement

Geological features indicate that the slide has moved at least 600 m since initiation. Movement since the most recent glacial outwash terrace was deposited below the landslide toe, approximately 16,000 years ago, is inferred from accumulated debris to be some 30 m in the toe. Aerial survey data interpretation indicates total movement between 1949 and 1991 of 2m in the head of the slide and 4m mid slope.

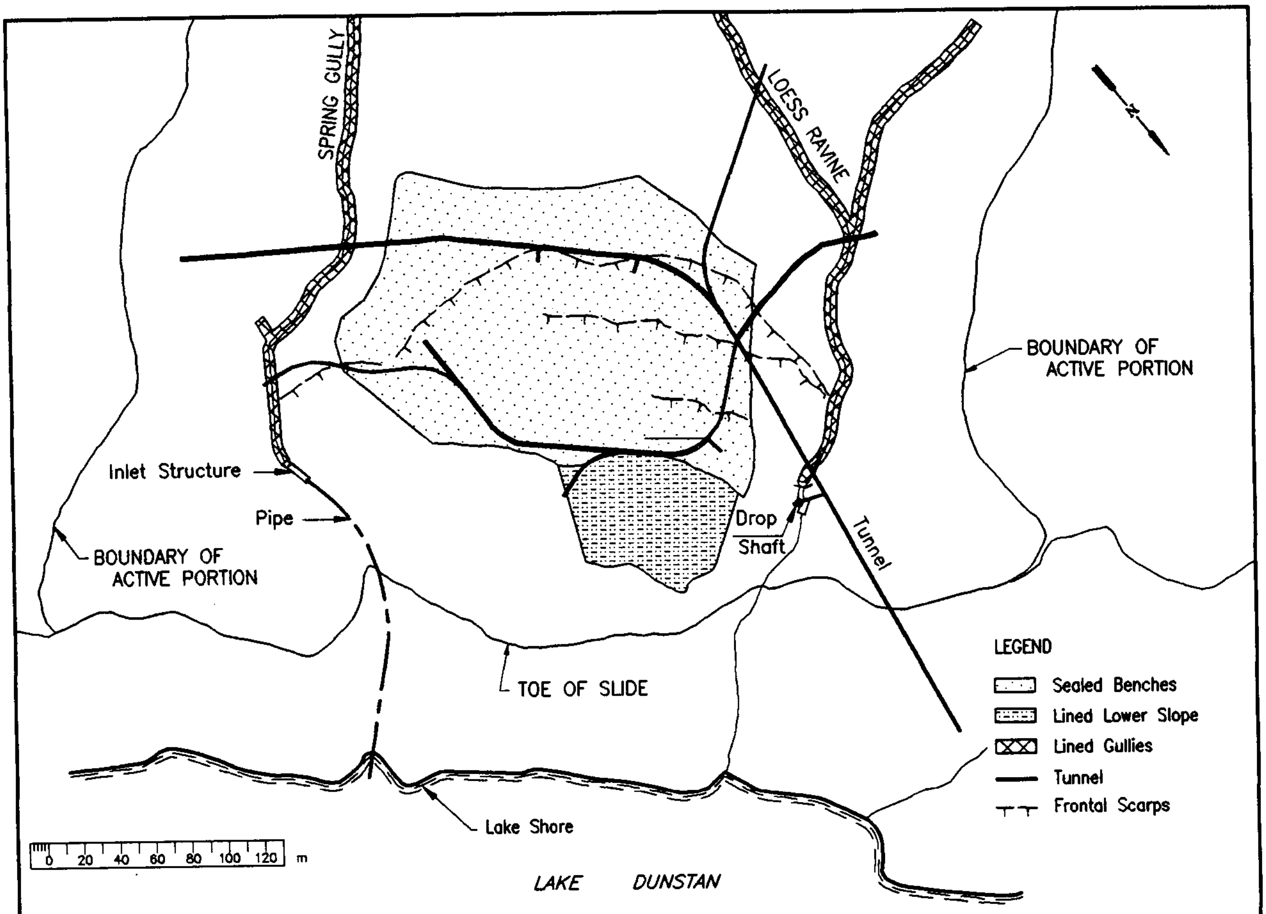


Figure 2 PLAN OF LANDSLIDE TOE AND STABILISATION MEASURES

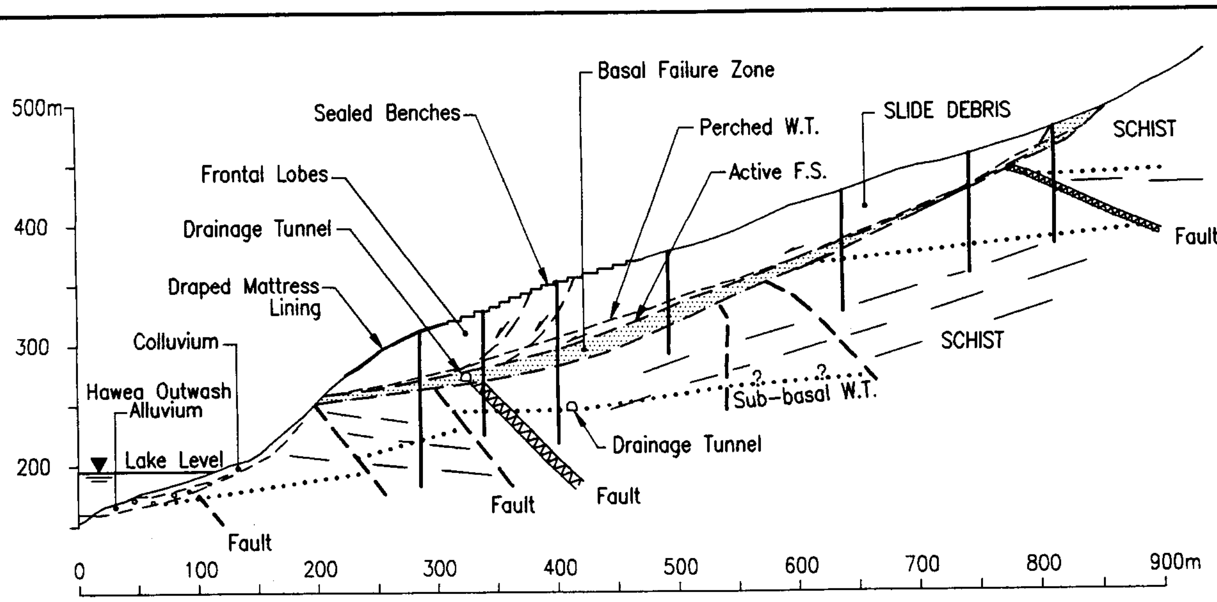


Figure 3 TYPICAL CROSS SECTION

Photograph 1 Cairnmuir Landslide Prior to Construction and Reservoir Filling. (ECNZ)

Rainfall initiated movement episodes have been observed since detailed monitoring was installed in 1989. Total observed movements during individual episodes have been up to 150 mm and 30 mm in the toe and head of the slide with rates of up to 3.3 and 0.5 mm/day respectively. The annual rainfall in the area is 430 mm/year and deformation has occurred in response to rainfall events of 20 to 50mm in 1 to 3 days, which have average recurrence intervals of 2 to 5 years. Piezometric and underground drainage flow responses to rainfall have been observed during wetter periods.

4 Surface Infiltration Protection

4.1 Design

Surface works were designed to limit infiltration from the surface into the more active Frontal Lobe Area near the toe of the slide and from the two main drainage gullies near the lateral boundaries of the slide (Figure 2 and Photograph 1). The aim of the stabilisation measures was to limit deformation to creep rates of less than 5mm/year. Hence the adopted measures had to be deformation tolerant.

The requirements for the infiltration protection were that the overall landslide mass distribution was unaltered, that the works were tolerant of slide movement and able to be easily monitored and maintained. Construction methods and sequences had to ensure safe working on the steep and marginally stable slopes of the slide. The visual impact from the nearby State Highway along the left bank of the reservoir was an important consideration.

The difficulty of construction access favoured simple methods with a minimum of imported materials.

A variety of innovative options were evaluated for the infiltration protection including tent and roof structures, buried and surface membranes and draped membranes. Buttressing was not considered to be a viable stabilisation alternative because of the height of the failure surface above the lake.

Conceptual designs were evaluated in a workshop involving representatives of the project management team, designers, construction specialists and external review consultants. A stabilisation plan was formulated. Design and planning for construction were carried out between August and October 1993, and construction commenced in November 1993.

Photograph 2 Cairnmuir Landslide Surface Infiltration Protection. (ECNZ)

5 Surface Infiltration Protection Measures

The development of practical measures to control infiltration required an innovative approach in order to satisfy the project objectives. Construction and safety considerations were major issues addressed in the detailed design.

5.1 Sealed Benches

Sealed benches were adopted in the 2.6 ha region of the Frontal Lobes (Figure 2) where the slope was typically 20° and up to 30° locally. Reinforced earth walls, typically 3.5 or 4.2m high, were formed (Figure 4, Photograph 3). Plain steel mesh facing panel and galvanised steel strips were used which should provide a design life in excess of 50 years in the low corrosion environment of the area.

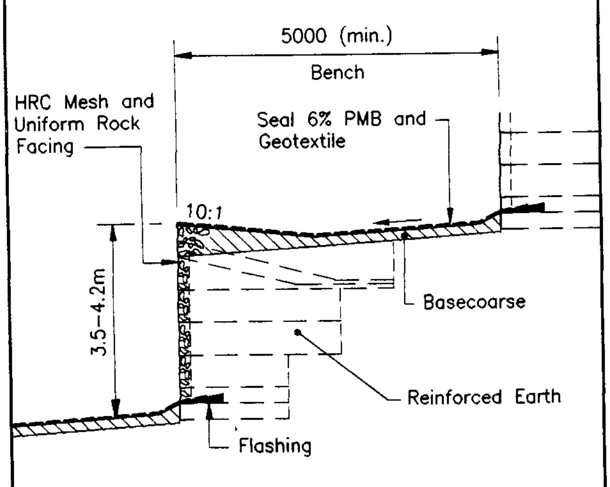


Figure 4 TYPICAL SEALED BENCH DETAIL

Tracked excavator cutting of the initial benches was adopted which proved to be safe and practical. The benches were constructed from the bottom up for safety reasons and for stability. The benches created a safe working environment and facilitated access.

A layer of coarse 100 to 200 mm uniform rock was placed against the steel mesh to provide a natural looking finish and a steel flashing plate was located near the base of each wall to intercept rainfall penetrating into the wall face.

The 5 to 15 m wide benches between walls were shaped to contain and direct water for discharge to the main gullies via 300 to 600mm diameter HDPE pipes draped over the ground. The surface of each bench was sealed with a specially developed geomembrane. The geomembrane was required to be flexible, durable, trafficable, and easily inspected and maintained. After trials and tests, a geotextile reinforced 6% polymer modified bitumen was chosen. A chip coating provided protection from ultra violet rays and traffic damage.

Photograph 3 Excavation and Construction of Walls for Sealed Benches. (ECNZ)

Drainage details included leak detection features to intercept any water seeping through the seal and discharge it to the outside of the wall below. Buried pipes were sleeved in key locations to provide flexibility, and leakage detection and control.

To ensure that the slide was not destabilised and a mass earthworks balance would be achieved the location of the walls was adjusted during construction for the actual ground contours and a mass balance recalculated on a wall by wall basis using Moss computer software. Walls were constructed using slide debris from excavation for the wall above. A number of different fill material gradings were produced on site from the slide debris using nearby crushing plant and mobile, skid mounted screens, towed behind the excavator (Photograph 3). The on site processing of materials resulted in cost effective and efficient construction. A net mass balance was achieved to within 2% of the total earthworks volume.

The innovative sealed bench treatment at Cairnmuir Landslide is the first known application of Reinforced Earth and geomembrane technology for landslide stabilisation by infiltration protection.

5.2 Draped Mattress Weighted Lining of Lower Slope

Near the toe of the slide a 0.6 hectare area of the surface with slopes of 30 to 35° was protected with a 2.5 mm thick HDPE geomembrane. The geomembrane was held in place with a gravel filled welded wire mattress (100 mm thick) supported by cables and anchors (Figure 5, Photograph 4). Wire cables were used to prevent the mattresses from sliding off the slope and were linked to grouted anchors, using steel spreader beams.

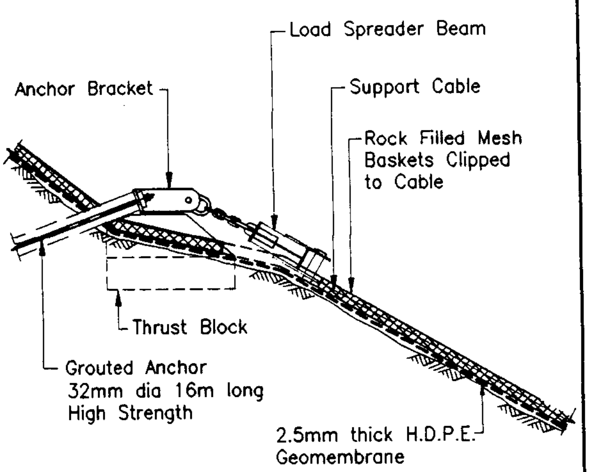
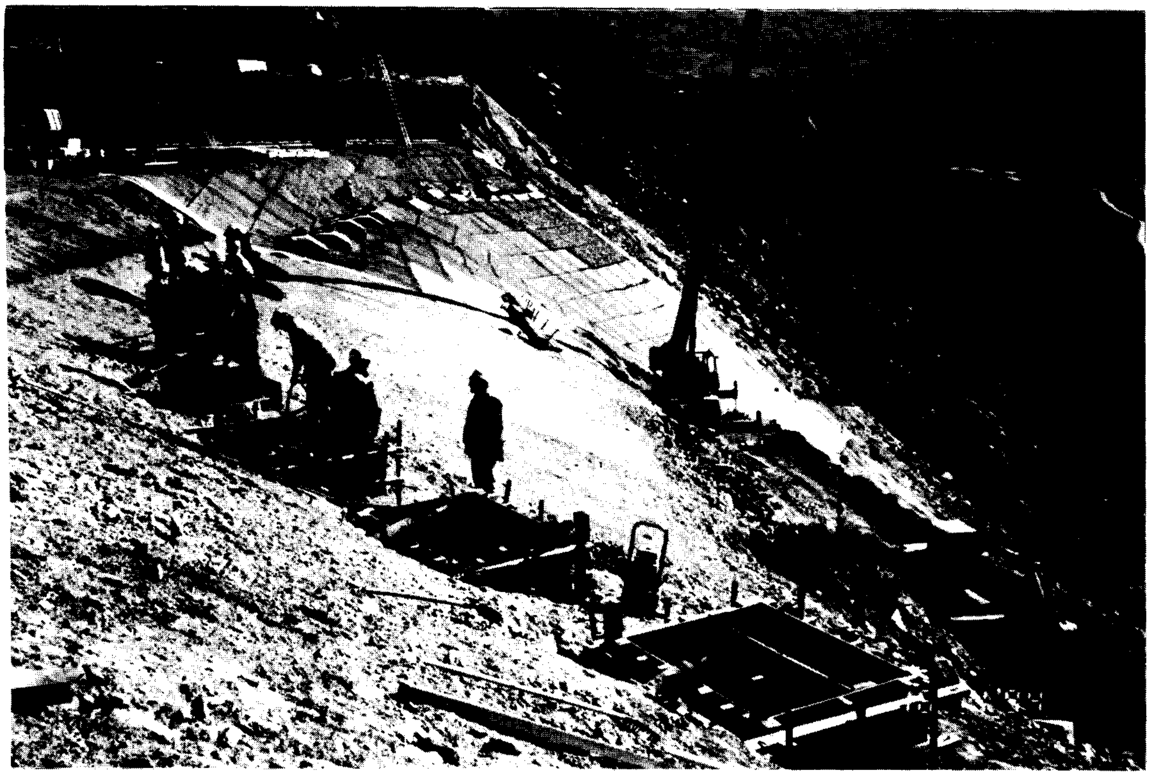


Figure 5 TYPICAL LINED SLOPE DETAIL

The gravel mattress filling was designed to absorb and drain intense "Thunder Pump" rainfall without damaging overspill onto the lower unprotected slopes and to counter uplift on the geomembrane from strong winds in the gorge. The mattress system was draped over the ground contours and the exposed gravel filling provided a natural appearance.

The ground surface was unloaded by 300 mm prior to construction to ensure a mass balance was maintained in the long term. Two cross slope run off interception drains were incorporated which discharged to the Northern drainage gully via HDPE pipes.



Photograph 4 Construction of Draped Mattress Weighted Lining. (ECNZ)

5.3 Lined Drainage Gullies

The two main drainage gullies for the slide area, Loess Ravine and Spring Gully (Figure 2) were conveying water into the more active toe region of the slide and lining was considered necessary to reduce infiltration over the full length of the gullies. The gullies were lined with a 2.5 mm thick HDPE geomembrane held in place by 300 mm thick Reno mattresses with under drainage. Reinforced concrete cut off walls were constructed across the gullies at approximately 60 m centres with a 1.4m drop enabling outlets for the under drains (Figure 6). Anchored cable support for the Reno mattresses was required where slopes were greater than 25° to ensure the tractive forces during a design flood flow of 5 m^3/s would not destabilise the Reno system. The mattress support cables were connected to grouted anchor bars at each cut off wall.

In the toe of the slide the gullies were steeper than 35° and prone to severe erosion in flood flow. A stable open channel solution could not be provided for full flood flows and alternative methods of discharging water had to be developed. A buried 1.2 m diameter corrugated HDPE pipe was installed in Spring Gully, to take all of the design flow and to protect the sensitive toe slopes of the slide from being destabilised. In Loess Ravine, the invert was partly comprised of in situ bedrock and it was considered acceptable to have limited intermittent flow in the lower slopes of this gully. A 650mm diameter, 80m deep, steel lined, drop shaft was constructed by raised bore air hammer drilling to divert up to 1.2m^3/s of the flood flow into the main drainage tunnel immediately below the gully. An inlet structure and bypass channel were designed to enable larger flows to continue to be discharged down the lower slopes of the gully bed. These solutions minimised the visual impact from the highway (Photograph 5).

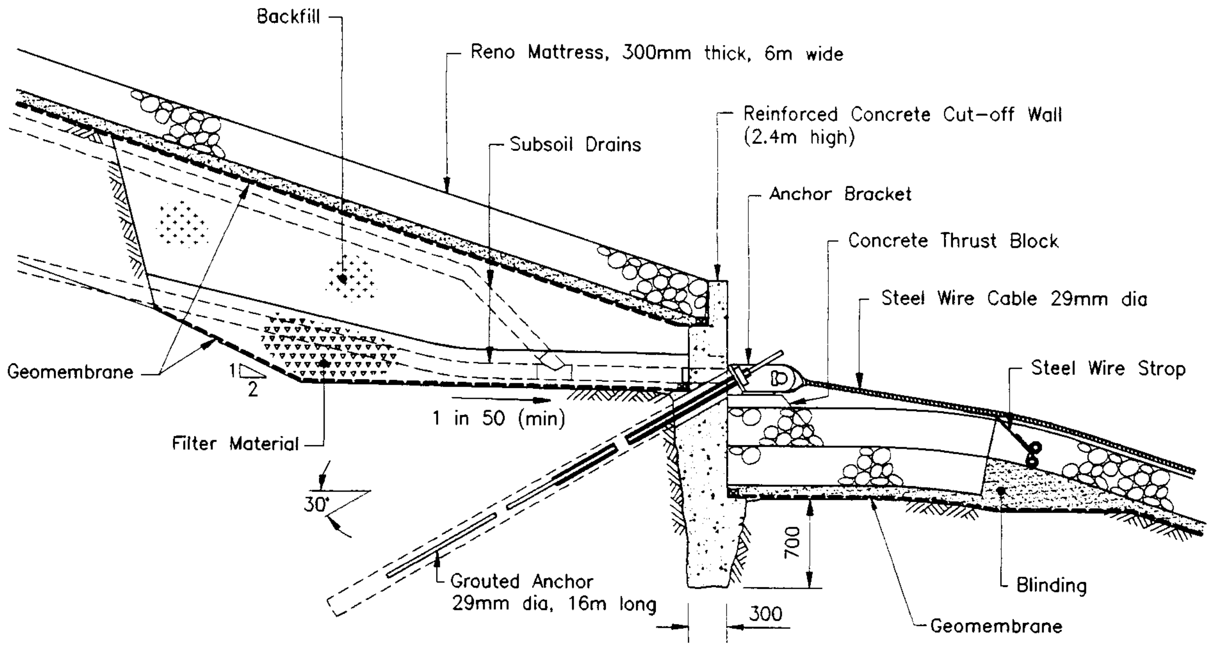
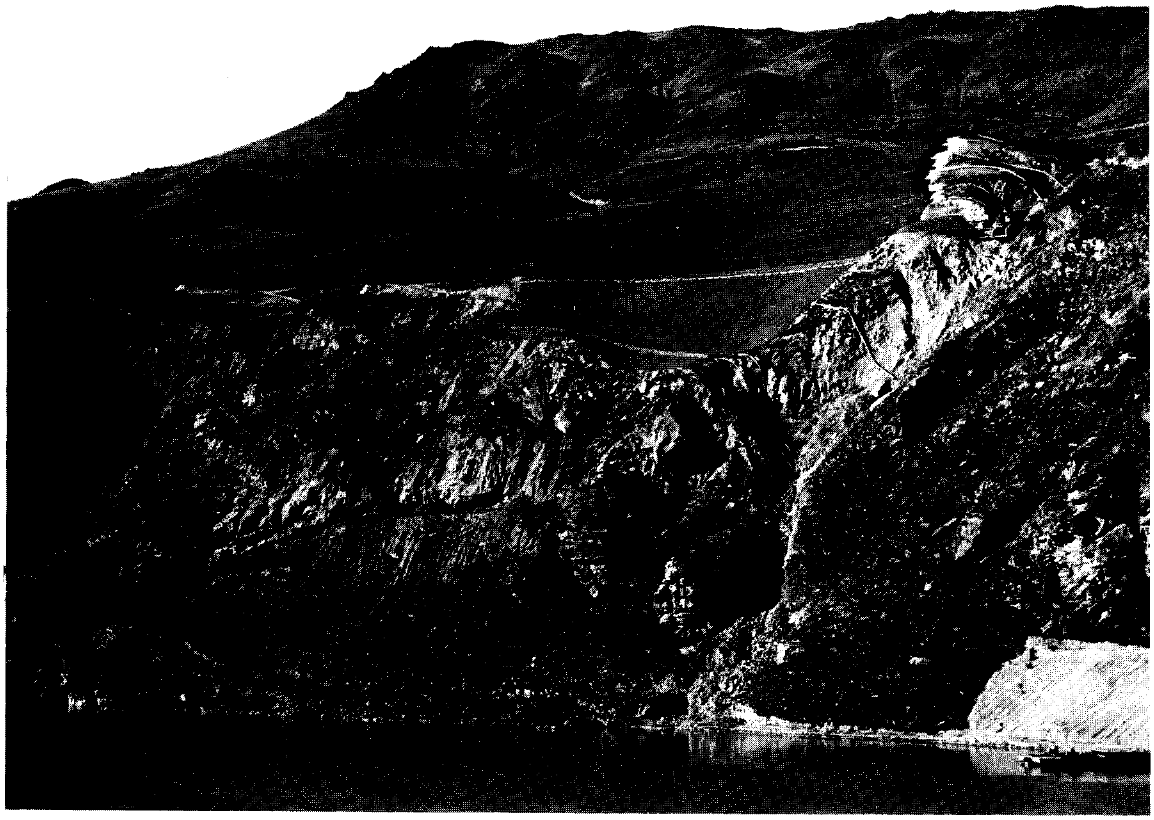


Figure 6 TYPICAL CUT-OFF WALL DETAIL FOR LINED GULLY



Photograph 5 Cairnmuir landslide infiltration Protection Viewed from The State Highway. (ECNZ)

5.4 Upper Slope Infiltration Treatment

Measures were implemented to limit infiltration and to reduce soil erosion and depletion in the upper slopes of the landslide above the Frontal Lobes. Run off interception drains were formed on the surface at 20 m vertical intervals on the slide and above the head scarp. The drains were sealed with a polymer modified bitumen and discharge to the lined gullies. Side drains were formed beside access tracks and were lined with a 0.3 mm HDPE geomembrane and a filter fabric, with a "geomat" fabric on the surface to encourage small plant growth.

Numerous sinkholes in loess and colluvium, and open features in schist debris above old tension cracks were sealed. The sparse vegetation on the slide surface had been depleted by rabbits and these are now controlled by fencing and culling. A programme of ground fertilizing and seeding has been carried out to improve vegetation coverage and limit infiltration.

6 Slide Performance

Since completion of the remedial works the slide has continued to slow and has not responded to 70mm of rainfall which fell between 5 and 8 November 1994. This amount of rainfall would previously have caused slide acceleration. Current rates of movement are less than 10 mm/year.

7 Conclusions

The design criteria and unstable nature of the slide surface required the development of various innovative stabilisation measures which extended the limits of existing technology. The solutions adopted and the manufacture of a wide range of materials on site minimised the import of materials from off the slide. The solutions also minimised construction time and enabled the works to be completed safely in 7 months before the onset of winter 1994.

Reinforced Earth technology offered a means of building benches on the relatively steep slopes which was tolerant of displacement, as well as variations in fill and foundation materials. The system was sufficiently flexible to enable the ground contours to be followed easily and alignments to be adjusted to maintain mass balance of the slope. The open mesh facing and uniform rock fill zone behind resulted in a natural coloured appearance of the structure (Photograph 5). Wall construction was simple and quick. The benches and bottom up construction sequence provided a safe working environment.

The benches enabled a flexible geomembrane to be constructed using local materials, labour and technology. The benches facilitate inspection and maintenance of the geomembrane.

Lining of the gullies allowed run off from the benches and other surface drainage works to be directed to the gullies until off the slide. The use of draped and cable supported mattresses enabled infiltration protection to be installed in the steep sensitive area of the slide and the upper portions of the gullies. The draped mattress provided a solution which was tolerant of deformation and had a natural appearance. In the lower and steeper areas of the two main gullies, a diversion alternative and a buried pipeline were used to maintain stability and preserve the natural slope appearance.

This combination of innovative applications to provide surface infiltration protection for the Cairnmuir landslide is believed to be unique.

8 Acknowledgements

The authors feel privileged to have been part of the large team that have designed and constructed remedial solutions for the Cairnmuir Landslide. The work has been carried out by staff from Electricity Corporation NZ Ltd, Works Consultancy Services Ltd, and specialist sub consultants.

The particular contributions of; C Newton, D Jennings and N Watson of Works Consultancy Services Ltd; R Fleming, J Rice, J Doolan and C Watts of ECNZ Clyde Power Project; and B Arthur who managed the site works for the main contractor Fulton Hogan Ltd are acknowledged.

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

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EWEN BRIDGE REPLACEMENT

FOUNDATION CONSTRUCTION

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Abstract

The construction of a 170 metre long four lane road bridge across the Hutt River necessitated the installation of 2.4 m diameter cylinder foundations penetrating the Hutt Aquifer. This aquifer, which provides approximately 25% of Wellington's water supply, has artesian heads of up to 5 m above the bed of the river in which some of the foundations were to be installed. Special innovative techniques were required to permit the foundation to be constructed while maintaining adequate safeguards against leakage from the aquifer. Comprehensive monitoring was undertaken and contingency monitoring and procedures developed in advance for identifying and responding to any leakages. Modifications were required to construction procedures during the project including the development of unusual and complex grouting techniques. Specific agreement to the modifications had to be obtained from the Resource Management Act Consent Authority.

1 Introduction

The existing three lane Ewen Bridge across the Hutt River at the southern end of the Lower Hutt CBD was constructed in 1928 as a two lane bridge. The Ewen Bridge Replacement is a part of the Ewen Floodway Project which was established by the Hutt City Council (HCC) and the Wellington Regional Council (WRC). That project arose from the need to reduce the flooding risk by removing the restriction provided by and improving flood protection works around the existing Ewen Bridge . WRC estimated a potential for millions of dollars worth of damage to commercial industrial and residential property and even for loss of life if the river breached the flood banks in the area of the Ewen Bridge.

The existing bridge does not meet current earthquake standards, cannot accomodate current or predicted traffic flows and is in need of substantial repair. After examination of a number of possible roading alternatives, HCC engaged Beca Carter Hollings & Ferner Ltd (BCHF) to carry out investigations, design and construction supervision of a four lane replacement bridge immediately downstream of the existing bridge.

In addition to the usual structural, traffic and hydraulic constraints encountered with any highway bridge across a river, the Ewen Bridge Replacement had to be designed and built so as not to jeopardise the security of the artesian Hutt Aquifer underlying the site. This aquifer provides up to 25% of the water supply for the Wellington region and the WRC has bylaws restricting construction activities that might breach the aquiclude layer that confines the aquifer and maintains the artesian head. Addressing the effect of the bridge foundations on the short and long term security of the aquiclude provided the major challenge in the development of the foundation design and construction procedures. There was also a requirement to limit further constriction of the existing floodway during the bridge construction period.

The objects of this paper are to document the experience gained in the construction of large bored foundations into an artesian aquifer and also to describe and comment on the application and effect of the Resource Management Act (RMA) on the project. Computer database literature searches did not identify any publications on the installation of bored foundations into artesian aquifers.

2 Bridge Arrangements

The bridge location, and vertical and horizontal profile (shown in Figure 1) were severely constrained by roading and floodway requirements. The east end of the new bridge connects (with minor adjustments) to an existing three leg roundabout while the west end connects to an existing grade separated interchange. The minimum length of the bridge was governed by the profile of the required floodway which fixed the abutment positions. The bridge vertical alignment was constrained by the lowest soffit profile acceptable to the floodway designers; the structural depth required by the bridge designers; and the accommodation of a highway vertical curve with steep approaches connecting into the existing (fixed level) highway junctions at the ends of the bridge. The bridge designed is 170 m long with 28 m precast U beams with hammerheads on a single 1.9 m diameter central piers as shown in Figure 2.

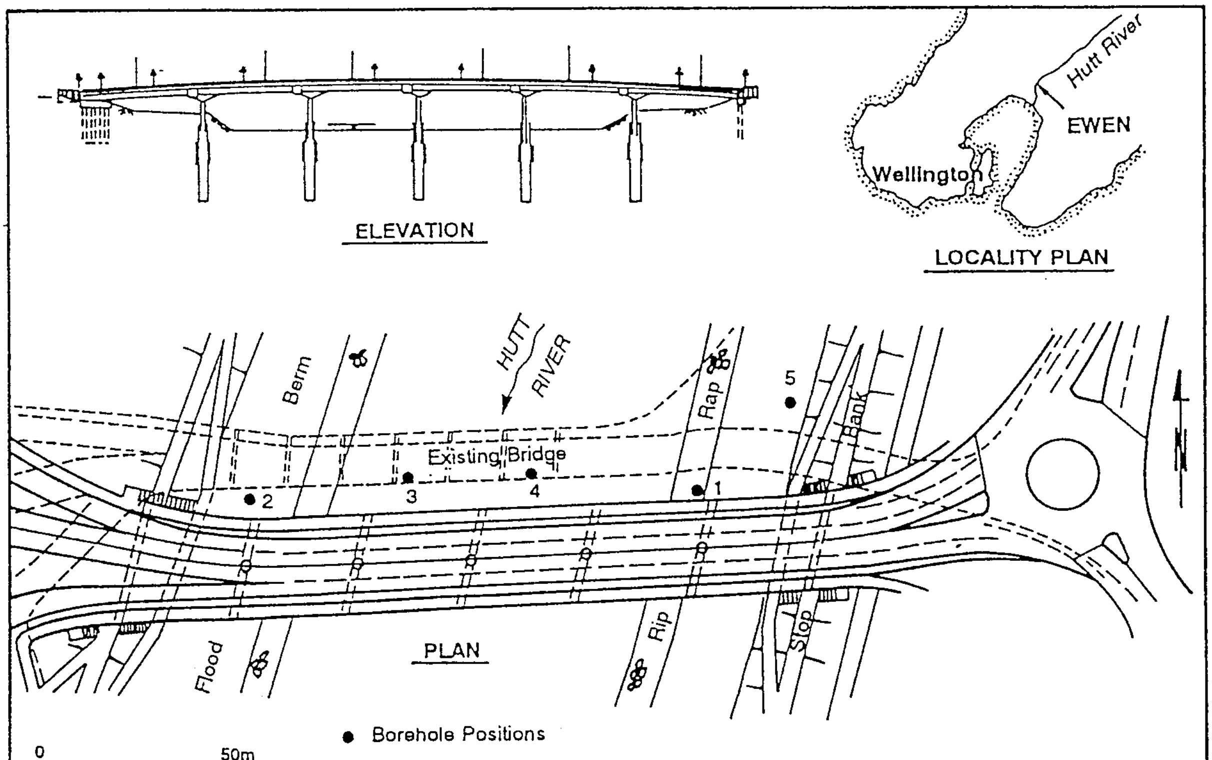


Figure 1: SITE & BRIDGE LAYOUT

The floodway designers placed a limit on the proportion of the riverway that could be obstructed by piers and would only accept single circular stem piers. They also nominated the general scour levels that were to be allowed for in designing the piers. Any pile caps or pier enlargements were to be located below the combined general plus local scour depth appropriate to the pier size.

3 Resource Management Act Consents

Both the bridge and the flood bank upgrading works required a number of consents under the Resource Management Act (RMA). The regulatory authorities involved (WRC and HCC) were also the owners and there was a complex interaction process. As the design and construction of the bridge and floodbank works were intimately interconnected, it was necessary in terms of the RMA that consent applications for the two structures be heard together. WRC and HCC appointed a Commission to hear the applications and consider submissions made by interested and affected parties. The Commission comprised a lawyer, civil engineer and town planner.

A major issue in the obtaining of consents for the foundations related to the prevention of leakage from the Hutt Aquifer and the monitoring and contingency procedures appropriate to enable detection and response to situations that might jeopardise the security of the aquifer. Prior to the hearings BCHF had developed a detailed Specification and a Monitoring and Contingency Plan. Extensive discussions with the WRC Consents staff and Works Consultancy Services (WCS) (who had been engaged by WRC as geotechnical advisors) were held over a six month period to obtain agreement on the construction methodology and monitoring and contingency procedures. The Consents were issued with a number of conditions, the major one being that all work proceed strictly in accordance with the tender Specification and the Monitoring and Contingency Procedures Plan. The Specification included certain method specification clauses to satisfy WRC requirements and was prescriptive in parts. It also contained a clause to the effect that Contractors could not assume that any variations to the specified methodology would be approved by the WRC Manager Consents, who was responsible for monitoring compliance with the Conditions of the RMA Consents. Any departure from the Specification required formal approval from the WRC Manager Consents.

4 Geotechnical Conditions

The geology of the Hutt Valley is described in detail by Stevens (1990). The western margin of the valley is the fault trace of the active Wellington Fault located approximately 600 m west of the site. The Hutt Valley has been infilled with alluvial deposits which are up to 300 m thick. There is an artesian gravel aquifer which is recharged from the bed of the Hutt River and extends from north of the bridge site to a point well out in Wellington Harbour. From a point several kilometres upstream of Ewen the aquifer is overlain by finer grained sediments with a much lower permeability. These act as an aquiclude, trapping ground water pressures which are artesian (that is above ground level). The aquiclude materials vary around the valley with the depositional history being dependent on the prehistoric river position, gradient and sea level. In the area of the Ewen Bridge the aquiclude is a 15 m thick bed of dense silty fine sands. Figure 11.8 in Stevens (1990) shows the interrelation of the aquifer, aquiclude and recharge zone.

Historical subsoil data for the area were limited to logs of waterwell bores and some shallow boreholes sunk to investigate the condition and foundations of existing stopbanks. For the Ewen Bridge replacement project, five investigation boreholes were sunk to a depth of 30 m below Mean Sea Level (MSL) at positions shown in Figure 1. Standard Penetration Tests were carried out at nominal 1.5 m centres in all boreholes and showed all materials to be dense to very dense. The investigation holes and specific proving holes sunk at each pier position during construction all intercepted the sequence of materials shown in Figure 2, including a medium sand layer within the aquiclude, referred to as the Upper Aquifer. That layer had artesian pressures but at a lower head than the main Hutt Aquifer. The thickness and levels of the layers were very consistent over the site.

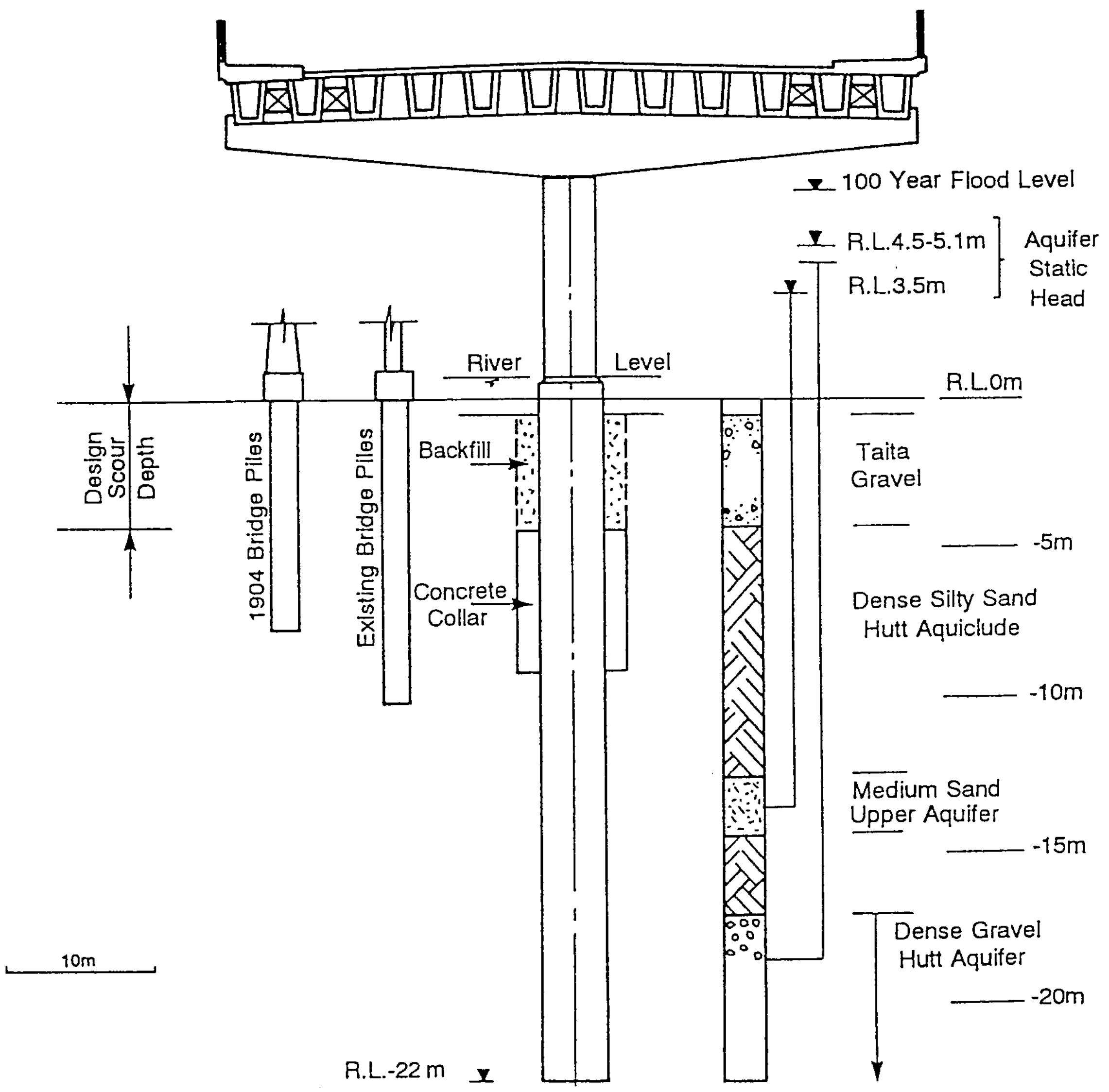


Figure 2: GEOLOGICAL PROFILE AND BRIDGE CROSS-SECTION

In addition to water levels recorded in boreholes during drilling, groundwater levels were recorded in piezometer standpipes installed in boreholes B2 and B5 with tips located in the Hutt Aquifer and Upper Aquifer. Typical ground water levels are shown in Figure 2. At the Ewen Bridge the Hutt River is tidal. Groundwater pressures in both the Hutt Aquifer and the Upper Aquifer fluctuate in phase with the tidal variations in the river level which was continuously monitored. Groundwater levels in the Hutt Aquifer are substantially effected by the rate of pump extraction for water supply purposes.

A borehole was sunk on the eastern side of the bridge to carry out insitu erodibility tests and recover undisturbed samples for laboratory testing of erodibility. These tests were in response to WRC concerns that the aquiclude materials would be very rapidly eroded if exposed by scour during a flood. The insitu tests involved an innovative technique specifically developed by BCHF for the project. A device was manufactured that subjected the sides of an unlined section of borehole to erosion by an upward flow of water at varying flow velocities. A borehole caliper was used to measure the change in the borehole shape after each increment in water flow velocity and hence determine the threshold velocity necessary to cause substantial erosion.

5 Foundation Selection and Design

Three existing bridges have foundations extending down to or into the aquiclude. Of these two have driven piles and one is on caisson foundations. Fussell (1978) has described underpinning of the Ava Railway Bridge, 1.0 km downstream from Ewen, while Morrison (1954) described construction of the Estuary Bridge 2.5 km downstream near the Hutt River mouth.

At Ava the bridge was originally built on short driven piles but later transferred onto deeper piles driven into the aquiclude after concerns that the original piles could be undermined by scour. The underpinning piles were driven steel H piles installed out the bottom of an internally excavated steel casing embedded into the aquifer and filled with bentonite. Fussell did not report any problems.

During construction of the Estuary Bridge there was an incident when a workman pumped out a caisson without approval with the result that the caisson was lifted and tilted by the unbalanced upward pressure from the Aquifer below. Prompt action by a foreman who reflooded the caisson was successful in averting a breath of the aquiclude. The incident is fully reported by Morrison (1954), and demonstrates the potential for problems if construction activities are not carefully thought out and coordinated.

The existing Ewen Bridge and the piers from the demolished 1904 Ewen Bridge are founded respectively on precast concrete and timber piles driven into the aquiclude. There are no records of their installation but as designed details are shown in Figure 2.

The floodway requirements dictated adoption of a single relatively high circular pier shown in Figure 2. This resulted in very high moment loads to the foundations. The "worst case" loadings were moments due to eccentric live load on the bridge which occur when two lanes in one direction are fully loaded and there is no traffic on the other side. Earthquake "code loads" were not critical compared with the eccentric live load. For consideration of the effect of a Maximum Credible Earthquake, the foundation capacity was checked against the overstrength moment capacity of the pier.

It was assumed that the foundations should support the bridge with eccentric live load with river bed level lowered to the maximum general scour level nominated by WRC. Any local scour holes were assumed to be present only while flood loadings were in place.

Three foundation types were considered being:

(i) Single deep cylinder foundation.

(ii) Driven piles connected to a buried pile cap.

(iii) A buried pad footing.

In terms of retaining the integrity of the aquiclude, driven piles were considered the most desirable foundation form as the aquiclude would be in intimate contact with the pile and as a result there would not be any leakage tracks up the piles. However the magnitude of the overturning loads on the bridge foundations would have required a pile cap about 12 m square and scour considerations required that it be built with a top surface 4 m below existing river bed. Construction of either a pile cap or a pad would have required construction of a sheet pile perimeter with sheet piles extending almost the full depth of the aquiclude. The risks to the short and long term integrity of the aquiclude as a result of installation and removal of the sheet piles were considered to be unacceptable.

As a consequence the single deep cylinder foundation was adopted as the only feasible form of foundation. After assessment of end bearing and lateral load capacity, a single 2.5 m diameter cast in situ pile founded at RL -22.5 was proposed. Structural analyses were carried out using horizontal coefficients of subgrade reaction moduli to confirm that the proposed pile size and depth was acceptable in terms of moment capacity and lateral rotations and deflections under service and seismic loads.

Finally the seismic behaviour of the supporting ground was considered. BCHF concluded that the aquiclude materials were too dense and had too high a fines content to liquefy.

6 Intended Construction Method

The construction method and specification contained particular requirements aimed at ensuring that the construction process did not jeopardise the security of Hutt Aquifer aquiclude. It was necessary to ensure the foundations would not heave under the artesian pressures, and that uncontrolled leakage would not occur up the sides of the pile casing penetrating into the aquifer. In addition procedures were specified to prevent or minimise long term leakage up the sides of the permanent pile. The intended construction procedure (including variations proposed by the contractor) is shown in Figure 3. Of particular note are:

(i) The use of the 4 m casing driven a nominal 6 m into the aquiclude and extending above the height of the static water level in the aquifer. To ensure a close aquiclude to casing contact the casing was driven without an external shoe. In addition the aquiclude to casing contact was to be grouted. Before the 2.4 m casing was installed, the 4 m casing was filled with water to the artesian head level. The level was then monitored for 3 days to check the effectiveness of the casing embedment to contain the artesian head with minimal leakage. Typical water level drop rates in the 4 m casings were 5 - 10 mm/hour.

(ii) The requirement that, to ensure a close ground-casing contact, the permanent 2.4 m diameter casing be driven (contrary to normal practice) without an external shoe.

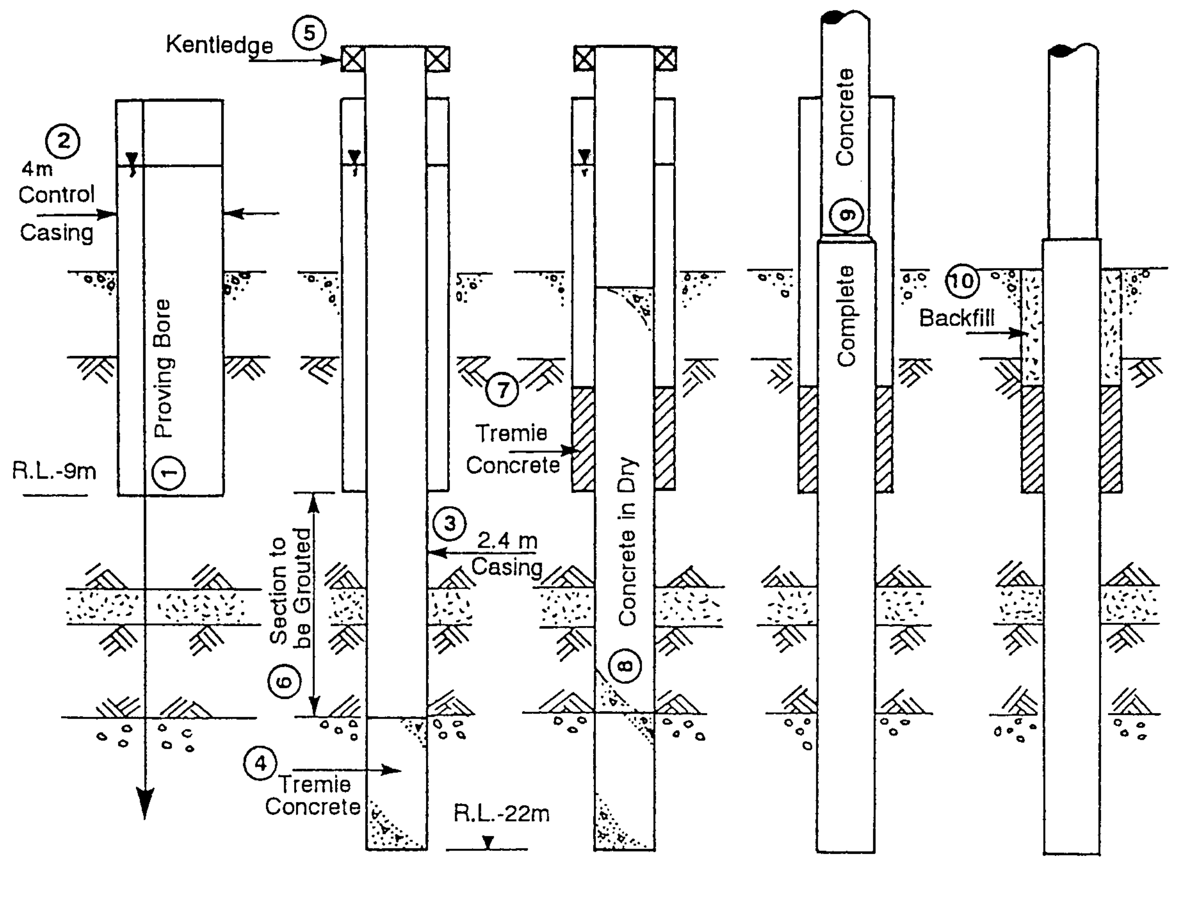


Figure 3: INTENDED CONSTRUCTION METHOD

(iii) The requirement that excavations inside casings were not to extend beyond the casing mouth while the casings were being driven through the aquiclude.

(iv) The requirement that bentonite be added to the water within the casings. This was at on the advice of WCS. Acceptance of this advice appeared to be central to obtaining agreement from WRC Consents not to oppose technical aspects of the RMA Consent applications. The use of bentonite was therefore accepted even though the designers did not consider it necessary.

(v) The use of a tremie to place the initial concrete under water in the pile.

(vi) The use of kentledge plus the weight of tremie placed concrete in the lower section of the pile to resist uplift pressures.

Contractors tendering the work had reservations about the feasibility of driving the 4 m casing to the specified depth. They also queried the purpose of and need for bentonite in the casings during driving, and commented on anticipated difficulties in developing procedures for effectively grouting the casing - ground interface. Tenderers did not anticipate any difficulties in driving the 2.4 m casing.

7 Monitoring and Contingency Procedures

The designers were fully aware of the complexity of the construction techniques required to install the foundations and the consequences of a failure of the measures put in place to protect the aquifer. Also, it was considered inappropriate to pass responsibility entirely to the Contractor for ensuring the measures were effective and for determining and implementing remedial work in the event of a potential or actual failure of the protection measures. There was also a clear need to establish, before construction commenced, procedures for monitoring the effectiveness of the measures and for responding to any problems. Therefore Monitoring and Contingency Procedures were developed by BCHF as a basis for tendering and obtaining Consents. A detailed Manual (MCPM) was prepared when the contractors methodology and personnel were known. The management procedures involved the establishment of two groups - the Technical Review Group (TRG) and the Contingency Reaction Group (CRG).

The TRG comprised the Foundation Designer (as chairman), the Engineer to the Contract, the Contractors Representative, a WRC Aquifer specialist and specialist geotechnical advisers engaged by the Contractor and by the WRC Manager Consents. The function of the TRG was to review monitoring information and construction related observations and to discuss methodology and progress. The TRG meet fortnightly to monthly for the first months of construction. After that the monitoring and progress results were distributed fortnightly and meetings held at less frequent intervals. The TRG had no power to instruct the Contractor but was able to make recommendations to the Engineer to the Contract. BCHF believe the TRG concept worked well at Ewen and provided a very useful and open technical forum without the pressure from contractual and project management constraints.

The CRG was a group comprising the TRG with the addition of Client, Project Manager and WRC Consents representatives and with the Engineer to the Contract as Chairperson. The CRG was intended to meet when an abnormal situation (termed a Contingency Event) requiring remedial action had been identified. Once a Contingency Event had been declared (as happened on occasions) the direction of the project effectively came under the control of the CRG which decided on the use of the Contractors resources to investigate and/or treat the identified problem. While a Contingency Event was declared, work undertaken by the Contractor related to the event was essentially on a dayworks basis. To assist the CRG, the foundation designer had identified conceivable (but in many cases highly improbable) scenarios (documented in the MCPM as Contingency Event Types) that might jeopardise the security of the aquifer. For each of these Contingency Event Types , the MCPM set out a series of tests to locate the possible leakage paths and criteria for confirming an Event Type. The MCPM also set out a sequence of treatment options.

While no major problems were encountered, the CRG was activated on a number of occasions. BCHF believe it was an effective approach to dealing with problems as all parties necessary to be involved in decision making and action implementation were present. The dayworks involvement of the Contractor enabled easy direction of resources to identify, as quickly as possible, whether a problem existed and, if so, the best means of dealing with it. Also to ensure that prompt remedial action could be taken in the event of a leakage up a casing, or a flood affecting a casing in the riverbed, the Contractor was required to stockpile certain materials (including cement, bentonite and rip rap rock) and hold available equipment and manpower to undertake remedial work at short notice.

8 Monitoring

The designers considered it essential to obtain the earliest possible warning of any failure of the aquifer protection measures or the development of any leakage paths up casing - ground interfaces. A comprehensive monitoring system was therefore specified and the Hydrological Services Group (HSG) of WRC were engaged by HCC and provided the following services:

For a three month period prior to construction a continuous (five minute sampling) record was obtained of water levels in the river and in piezometers installed in the Upper Aquifer and main Hutt Aquifer. This information was assembled and published as a Base Monitoring Data Report which also examined and reported in the influence on the ground water pressures of tidal fluctuation in the river, flood passage and the extent of extraction for water supply from the Hutt Recreation Ground Pumping Station 500 m to the east.

Throughout construction, HSG continued to monitor river water levels. Also, whenever the site was not occupied, monitoring pressure transducers were placed within all casings containing water. These transducers were connected to a telemetry system which relayed readings back to the HSG headquarters Alarm limits were set for the casing water levels as an indication of abnormal drops or rises that could indicate a leakage from a casing or an upward flow in from one of the artesian aquifers. When a telemetred reading moved outside its preset normal range (usually in the early hours of the morning) the HSG system initiated a call to a duty officer who was equipped with a modem and portable computer. On receiving a "call-out", the HSG Duty Officer would remotely interrogate the database to check the pattern of reading movement to identify whether there was a slow trend or a sudden change and also check the movement of other monitored water levels. The MCPM defined a call out sequence which was used by the HSG duty officer to initiate action to respond to abnormal readings not obviously attributable to an equipment malfunction.

Despite the inevitable occasional equipment malfunctions the alarm monitoring system proved very effective, identifying a number of contingency situations. The maintenance of the records on the HSG database also permitted a wide range of reports to be generated for daily, weekly, and monthly review by the foundation designer and the TRG.

9 Construction Experience

Construction started at Pier 6 on the east bank. Despite initial concerns, there were no difficulties in driving the 4 m casing which was advanced with slow but steady progress (typically 2-3 mm/blow) using four 3.2 tonne monkeys dropped simultaneously from 1.2 metres. The internally mounted grout connections were however damaged during the casing installation and internal excavation and only 4 of the 12 grout tubes were operative. No grout takes were measured at any of the operative ports indicating a good aquiclude to casing contact and the subsequent water test showed the 4m casing to be sealed into the aquiclude. During the project no grout takes were experienced in any of the 4 m casing grout injection ports.

The 2.4 m casing at Pier 6 was then pitched and driven from RL -9 to RL -12.5 (Reduced Level Datum is MSL). At this point the casing effectively refused advancing less than 0.2 mm per blow under three 3.2 tonne monkeys dropped simultaneously from 1.2 m. The Contractor did not increase drop energy due to the risk of casing damage. The TRG then initiated investigations to attempt to identify the cause of the refusal. Excavations were made down to and beneath the casing shoe. Divers sent down were unable to detect (by feel) any damage to the casing shoe or the presence of any logs or other hard objects beneath the shoe. Redriving gave increased but still very small sets suggesting the problem was one of skin friction rather than end bearing. An investigation bore hole was sunk inside and beyond the base of the 4m casing which had by then been driven at Pier 5. SPT results were essentially identical to those in the proving bore drilled at Pier 5 before the installation of the 4 m casing. This suggested that the problem was not due to densification resulting from the installation of the 4 m casing.

By the time the investigations and trials at Pier 6 were complete the 2.4 m casing had been advanced down to RL -14, still 8.5 m short of founding depth. It was clear that it would be impractical to drive the 2.4m casing to depth and agreement was obtained from the TRG and WRC Consents to install a 2.2 m casing with an external shoe and drive it to founding depth. The 2.2m casing was driven to founding depth without difficulty, the gap between the ground and the 2.2 m casing left by the external shoe being grouted by flooding the 2.2/2.4 m annulus with grout. Pier 6 concreting was then completed using the sequence in Figure 3.

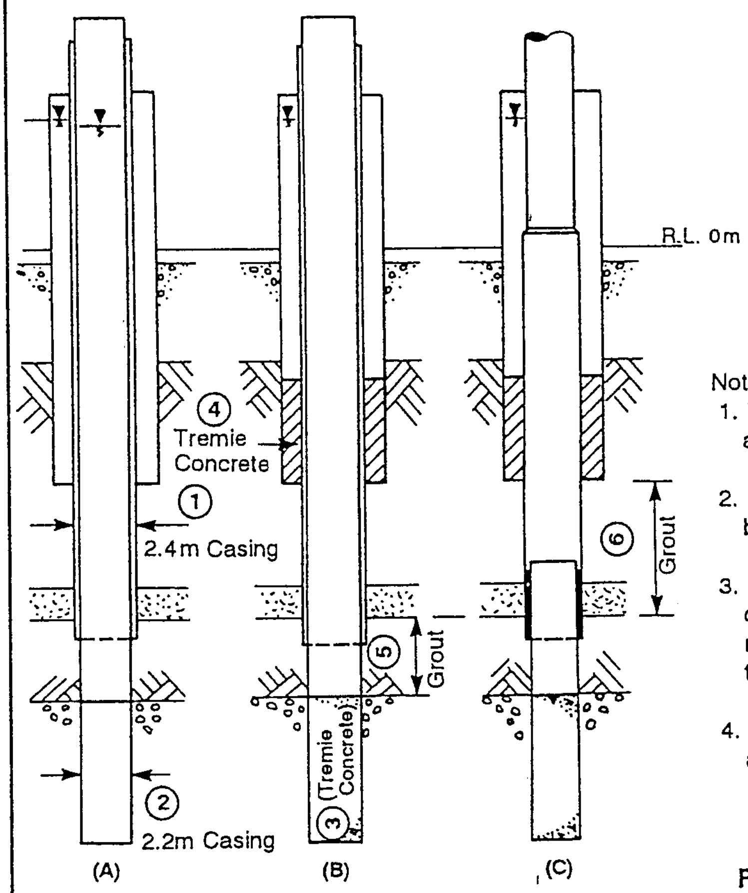
At Pier 5 the 4 m casing would not hold the elevated water levels. Divers were sent down to examine a seal at a mechanical joint in the casing (provided to allow the upper section to be disconnected and lifted out) and to attempt to locate any concentrated outflows. The divers could not detect any concentrated outflows but did ascertain that the internal seal at the mechanical joint had worked out from the joint. Attempts to reseal the joint using Denso paste and tape were unsuccessful. Eventually the leakage from the casing was sealed by flooding the casing base with a very thick bentonite mix (Specific Gravity 1.06) introduced with a lowered water level in the casing. The bentonite water mix was selected after trials of various combinations of cement, bentonite and sodium silicate. The experience at Pier 5 indicated that a low SG (1.012) bentonite dosing in the 4 m casing was not effective in sealing leakages.

The experience at Piers 5 and 6 indicated that both detail and fundamental changes were required to the casing installation procedure and grouting systems. The mechanical joint on the 4m casing was dispensed with, the casing separation on subsequent piers being achieved by cutting underwater. The fixed grout pipes on the inside of the 4 m casing were replaced by grout fittings with non return valves to which a flexible pipe was connected by a diver. It was also apparent that, given the depth of the water in the casings and the presence of silt and bentonite, conditions were such that it was unreasonable to expect that the divers could carry out reliable inspections or measurements. The conclusion was that the use of divers should be avoided where ever possible.

10 Development of Revised Construction Approach

The RMA Consents required that construction proceed to the specification which required the use of casings without external shoes. However after the experience at Pier 6 it was clear that driving the 2.4 m casing to founding depth with internal shoes was likely to be impractical and that, if attempted, there was a major risk of casing damage under the high driving energies employed. The Contract Documents clearly placed the onus on the Contractor to solve this problem; which included the obtaining of approval from the WRC Manager Consents to the modifications to construction procedures. However, the problems encountered had not been expected, and HCC acknowledged that the engineers best placed to assist the Contractor were the HCC consultants (BCHF) who had the most comprehensive knowledge of site conditions and had been party to the negotiations with the WRC Consents associated with the original design. HCC therefore offered the services of BCHF to the Contractor to assist to develop and obtain agreement from the WRC Manager Consents for an alternative approach based on the use of telescoped 2.4 and 2.2m casings with external driving shoes. It was accepted that these casings would have a gap outside them and that the gap would need to be sealed by grouting.

The alternative methodology proposed was fundamentally different from that defined in the Specification, which was a Condition of the RMA Consents. The initial reaction of WRC was that a new RMA Consent Application might be required. However after protracted discussions the alternative proposed was accepted and a grouting procedure agreed, although with a substantially greater number of grout ports than was considered necessary by BCHF. WRC accepted that, if grouting of the first pier suggested that a lesser number of ports would have been effective, then, proposals for reduced number of ports on subsequent piers would be considered.



Notes:
1. Installation of 4m Casing as per Figure 3
2. In (B),2.2 and 2.4m Casings connected before dewatering after steps 3 and 4
3. Before step 6;2.2m and 2.4m annulus concreted at base;2.2m and 2.4m casings reconnnected below 2.2m cut off level, then 2.2m cut off
4. Completion of pier concrete and backfilling as per Figure 3

Figure 4: REVISED METHODOLOGY

HCC agreed that the extent of the additional grouting operations was difficult to predict, and that these would be considered as a variation to the Contract. The action of HCC in permitting its designer to develop an alternative construction approach with the Contractor resulted in an effective co-operative relationship forming between the Client, Contractor and Designer. This demonstrated the advantage of close contractor/designer co-operation and interaction in situations where innovative solutions are required,

11 Revised Construction Sequence

The revised construction sequence is shown in Figure 4. The contractor proposed the early pouring of the collar concrete and the connection of the 2.2 and 2.4 m casings to enable the collar and casing weights to be used to resist uplift pressures. This eliminated the need for the kentledge which hampered access to the casings. Otherwise casing installation and concreting sequence was identical to that at Pier 6. The process allowed the casing to be dewatered once concrete had been poured by tremie up to the level of the Hutt Aquifer surface, which was also the bottom of the section to be grouted. It was therefore possible to install grout injection ports from within the dewatered casing provided the installation process could control release of the 20 m head of water behind the casing. Devices were specifically developed to allow holes to be drilled through the casings into high pressure water to create grout ports without releasing the water. A device was also developed for drilling through the grouted ports to measure the thickness of the grout and conduct controlled seepage tests in the ground behind the grouted zone. The grouting process was a complex and closely controlled and measured process and included extensive pre - construction trials. Details are given in the companion paper by Ramsay and Marshall (1995).

12 Conclusions

Construction of subsequent pier foundations was completed without any significant difficulties after implementation of the revised construction methodology. The grouting experience at each pier was different but the results of the grouting and the proving tests provided confidence that the gaps between the casings and the aquiclude had been effectively sealed.

This case history demonstrates the inherent risks in departing from established practice (in this case external casing shoes) even where there are compelling reasons (in this case to ensure a low permeability zone behind the casing to contain the artesian pressures).

It also demonstrated the time, effort and uncertainties that may be involved in obtaining approvals from Consent Authorities for changes in methodology when RMA Consents are referenced to a prescriptive construction methodology Specification.

13 Acknowledgments

The author acknowledges the permission of the Hutt City Council to the publication of this paper and also the contributions made to the development of the revised foundation methodology by the Contractor (J Juno Construction); the foundation subcontractor (Richardson Drilling Company); and Mr R W Irwin, who was engaged to advise on grouting.

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EWEN BRIDGE REPLACEMENT

PILE GROUTING

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Abstract

Construction of a new 170 m long four lane road bridge across the Hutt River at Ewen involved installing 2.4 m diameter cast in situ piles to a depth of 22 metres below river bed. The piles were founded in the Hutt Aquifer which provides 25% of Wellington's water supply and is artesian with a static water level 5 m above river bed level. After it proved impossible to drive a single 2.4 m casing with an internal shoe to founding level, casings with external shoes were adopted. These were expected to leave a gap between the casing and the ground behind the shoe. This paper describes the development and implementation of grouting procedures to effectively seal that gap to prevent release of the aquifer pressure.

1 Introduction

A companion paper by Ramsay (1995) describes fully the investigation, design and construction of foundations for the Ewen Bridge Replacement. The foundations, shown in Figure 1, comprised a 2.4 m diameter cast insitu pile founded at RL -22.5 in an artesian aquifer with a static water pressure head 5 m above river bed level. The original design discussed by Ramsay (1995) involved driving a 2.4 m casing with an internal driving shoe from within a 4 m control casing (which would be filled with water to aquifer head level). However on the first pier it proved impossible to advance the 2.4 m casing below RL - 14 m. An alternative approach was adopted using telescoped 2.2 and 2.4 m casings with external driving shoes, which were expected to leave a gap between the casing and the ground. If significant flows up the casing interface were to occur (after removal of the control casing balancing head), these might cause erosion of the aquiclude and generate a large flow path sufficient to release the aquifer pressure and destabilise the pile. It was therefore necessary to provide a permanent seal in the gap. This paper describes the development and implementation of grouting procedures to seal the gap.

2 Geotechnical Conditions

For the project, five investigation boreholes were sunk to a depth of 30 m below Mean Sea Level (MSL). Standard Penetration Tests were carried out at nominal 1.5 m centres in all boreholes and showed all materials to be dense to very dense. The investigation holes, and specific proving holes sunk at each pier position during construction, all intercepted the sequence of materials shown in Figure 1, which included a medium sand layer (referred to as the Upper Aquifer). The Upper Aquifer was also artesian but had a lower static head than the main Hutt Aquifer. The thickness and levels of the layers were very consistent over the site. The aquiclude has a low permeability, assessed from grading curves at 10^{-7} m/sec, while the Upper Aquifer has a relatively high permeability (measured from insitu pumping tests) of 7×10^{-5} m/sec. Grading curves are presented in Figure 2.

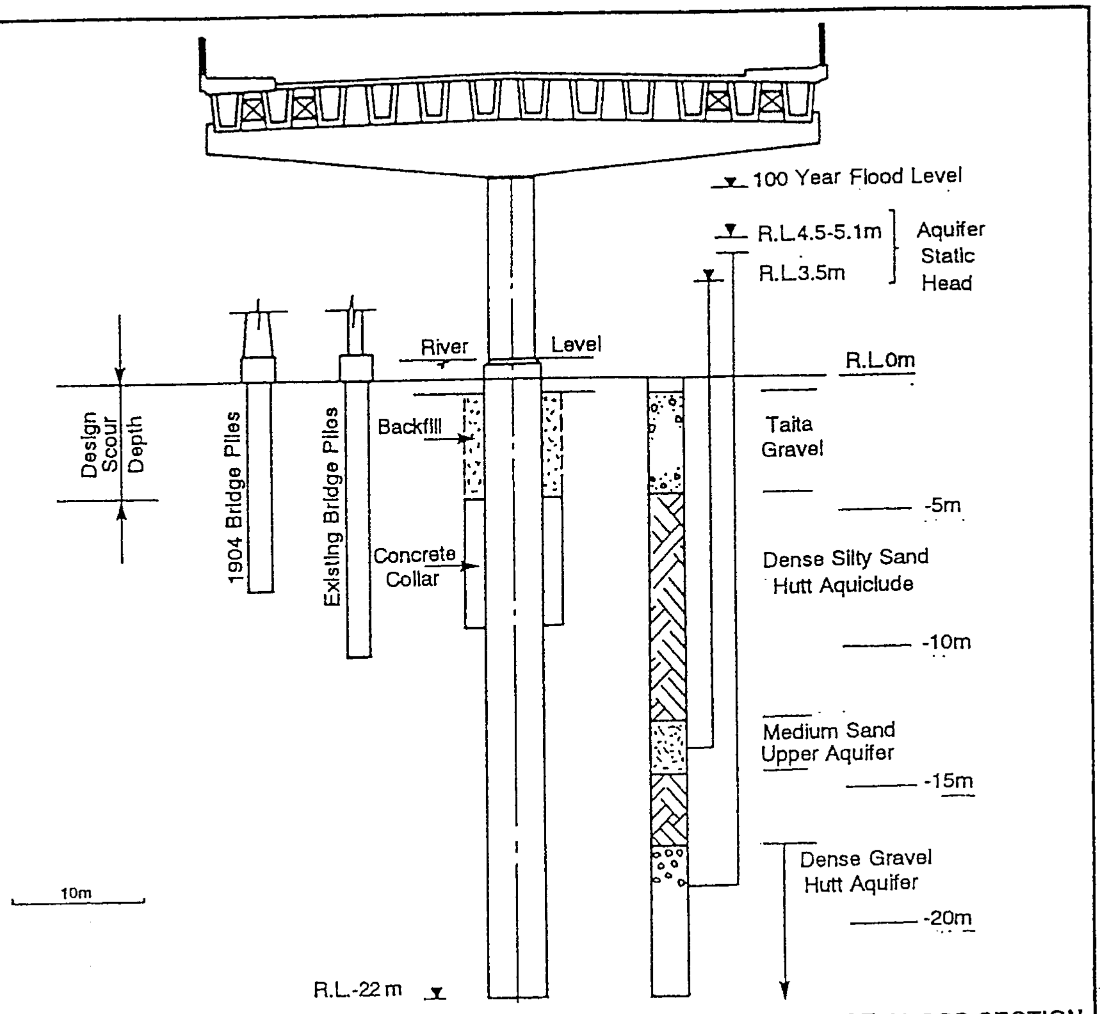

Figure 1: GEOLOGICAL PROFILE AND BRIDGE CROSS-SECTION

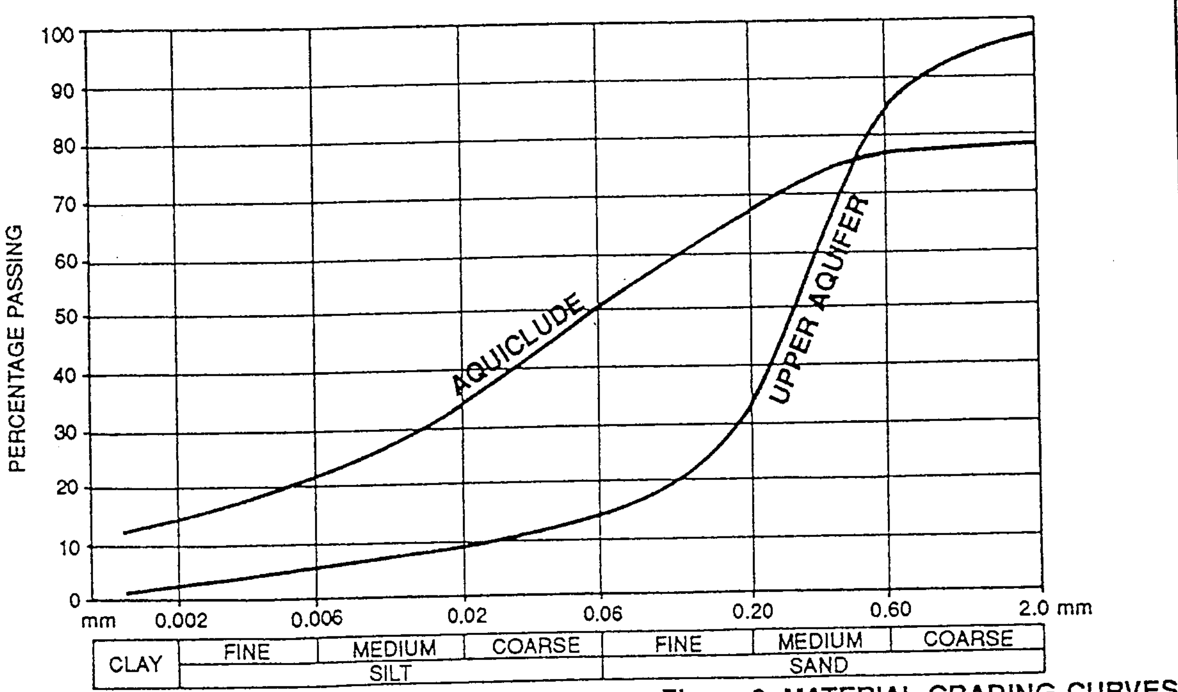

Figure 2: MATERIAL GRADING CURVES

3 Construction Sequence

The modified construction sequence is described in detail in Ramsay (1995) and shown in Figure 3. First a 4 m control casing was driven and internally excavated to RL -9m and filled with water to artesian aquifer head level. The 2.4 m casing was pitched inside the control casing and driven and excavated to a depth around RL -14m. The 2.2 m casing was then pitched inside the 2.4 m casing and driven and excavated to founding level. The bottom section of the 2.2 m casing and the 4 m diameter collar were then concreted underwater by tremie and the casings interconnected so that the combined concrete and casing weights could be used to resist the uplift pressure from the aquifer and allow the 2.2m casing to be dewatered. At this stage it became possible to access the 2.2 m casing for installation of ports and grouting of the gap outside the casing.

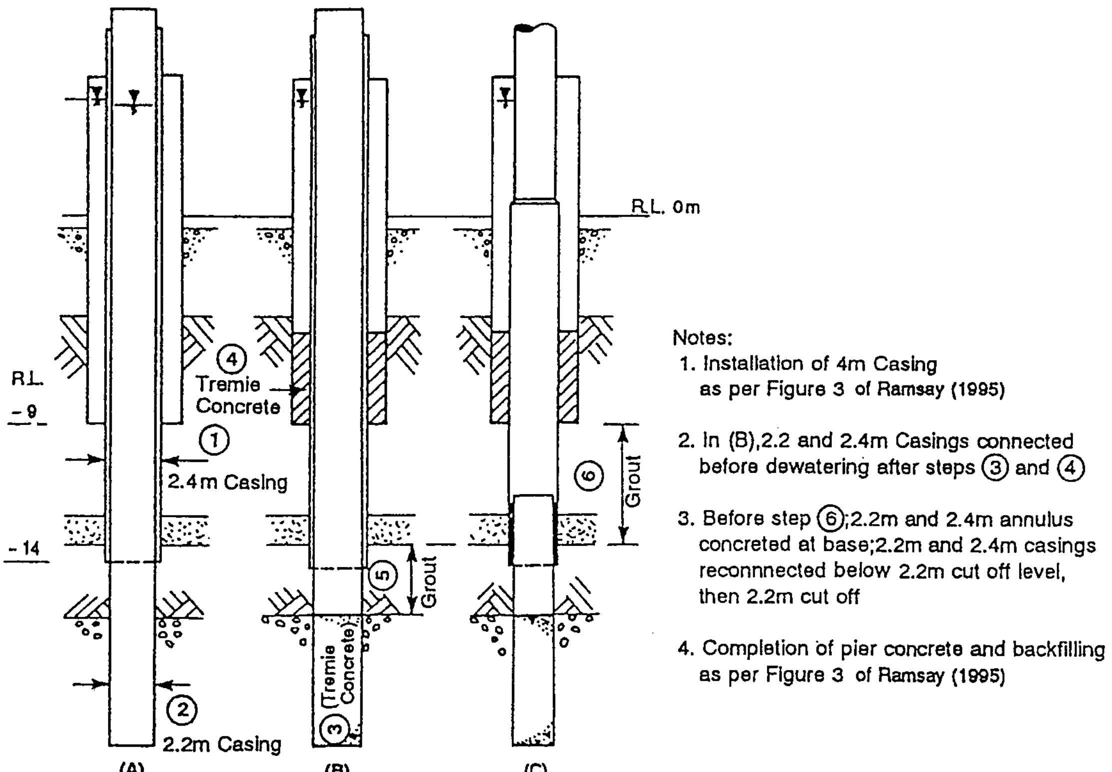

Figure 3 : REVISED METHODOLOGY

4 Grouting Approach

The objective of the grouting was to completely fill any voids outside the casing left after the passage of the driving shoe and also to consolidate any material that had relaxed into the gap left by the driving shoe. A number of grouting methods were considered but only two appeared viable :

(i) Grouting via ports in the casing installed from within the dewatered casing. This provided greatest flexibility as it was based on grout ports being be drilled through the casing after it had been installed. This method enabled isolated voids to be identified and grouted, and permitted testing of the primary grouting effectiveness and also secondary grouting.

(ii) Grouting by flooding the annuli between the casings with grout. This method had been used at the first pier constructed. In that case, the 2.4 m casing had been driven with an internal shoe, and it was only the 2.2m casing to aquiclude interface where there was a gap to be grouted. After using the flooding technique at that pier, it had been noted that it was difficult to control the operation and determine the final position of the grout. As the grout column has a higher density than water, the grout tends to flow downward quickly. Also with this method there was possibility of grout not reaching lower gaps if material had collapsed back against the casing at a higher level.

Method (i) was accepted as the only one which provided the necessary control and flexibility and ability to prove the effectiveness of the grouting. The following are key features of the grouting procedure proposed :

(a) Ports would be drilled through the casing using a device, to be developed, that would prevent any flow of the high pressure water (20m head) into the casing and the inevitable erosion of aquiclude material into the casing.

(b) Prior to each grout injection phase, dyed water would be injected through intended grout injection ports and the pattern of dyed water return at other ports noted. The observations from these water tests would be used to confirm the number and location of the ports to be used for grout injection.

(c) The casing - aquiclude interfaces to be grouted would be subdivided into three sections (referred to as 'levels'). Passage of grout beyond the top and bottom of each level would be controlled either by a casing driving shoe or a band welded to the casing (and referred to as a 'collar').

(d) Grouting would proceed in three steps :

- Primary grouting at lowest possible heads (but not greater than 70% of total overburden stress) with bleeding at other ports to achieve, wherever possible, a full pattern of grout return at other primary ports. For grouting purposes the ports on each level were subdivided into rings.

- Grout effectiveness checking through secondary ports (spaced between the primary ports) installed after completion of primary grouting. These ports would be used to check for the presence of grout and hence the penetration of the primary grouting. A device was also to be used in selected ports to check the rate of seepage from the ground behind the grouted gap. This would act as a check on the effectiveness of the primary grouting in consolidating and reducing the permeability of ground that had relaxed into the gap left behind the driving shoe.

- Consolidation grouting carried out through the secondary ports at pressures typically 80% but up to 90% of total overburden stress.

(e) All grout injection pressures and backpressure at bleed ports would be controlled by header tanks and recording procedures would be established to accurately identify the grout take through each port and the pattern and location of ports through which grout return was achieved.

The trials indicated that the bentonite caused a marked reduction in the fluidity and that it was preferable to overcome this by addition of higher dosing with Febgrout (an aluminium powder based plasticiser, water reducing agent and expanding agent) rather than by increasing the water-cement ratio. A grout mix was adopted with a water:cement ratio of 0.5 and 2.5% (Rheogel L) bentonite plus 0.5% Febgrout. This mix exhibited good fluidity (12 seconds ASTM flow time), moderate expansion and acceptable setting times. To ensure thorough mixing of the bentonite through the grout, the sequence of adding materials during mixing was water-bentonite-cement-Febgrout. Specific Gravity (mud balance) and flow cone tests were performed after mixing and at regular intervals during the grouting operation, samples being tested from the agitator and at injection points. Grout consistency of return bleed flows was also checked to ensure the grout had travelled from the injection point to the bleed point without significant mixing with water. Once return flows exhibited a normal grout consistency that bleed port would be closed off.

6 Grouting Equipment

A conventional mixing system was used comprising a fast 150 litre capacity (Craelius 175) colloidal mixer, a 300 litre capacity (Cemag 150) Agitator, and a 70 litre/minute (Craelius) Grout Pump. A clear 25mm plastic standpipe was connected to the side of the agitator to indicate the grout level in the agitator This enabled the volume of grout in the agitator to be measured. An accurate record of the flow of grout into particular ports was obtained by measuring the volume in the agitator (and also the amounts added from the pump and the volumes periodically discarded from the agitator as having been mixed too long as indicated by an unacceptably long flow time).

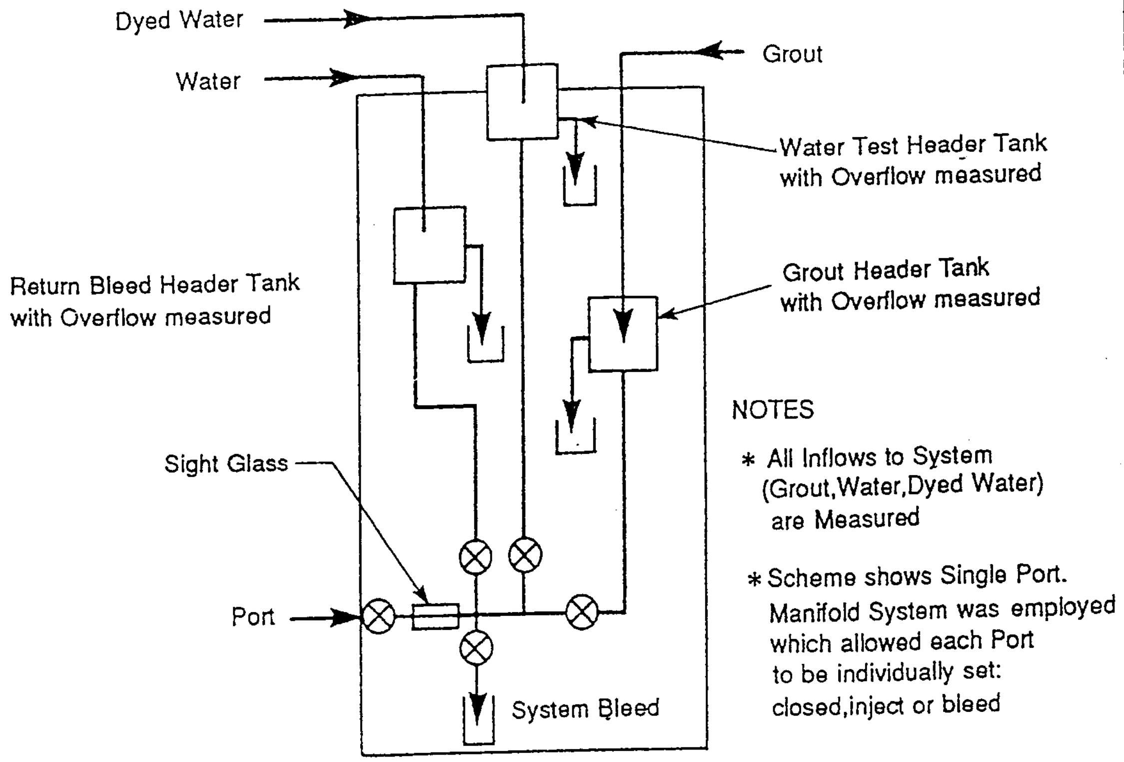

Figure 4: GROUTING DELIVERY ARRANGEMENTS

(f) Grout mix trials would be carried out and simulated trials conducted using the selected grout mix and the proposed equipment to confirm the procedures were workeable before actual casing - aquiclude interface grouting was undertaken.

(g) All grouting operations were to be directed by a Beca Carter Hollings and Ferner (BCHF) engineer. For this purpose comprehensive checksheets and recording sheets were to be prepared for each phase of the grouting. A report was to be prepared on the completion of grouting at each Pier.

5 Grout Mix Design

The grout to be used was required to contain bentonite (to comply with WRC specifications) and was also expected to have the following properties :

- minimal bleed
- expansive rather than shrinkage characteristics
- a consistency that would permit easy injection but limit mixing with water. A flow time of between 11 and 14 seconds for an ASTM 1725ml Flow Cone was adopted as the acceptance criterion.
- a setting time such that the grout would remain fluid sufficiently long allow flow to bleed ports but not so long that tidal fluctuations in aquifer static head would be able to flush grout from Level I. The setting time was assessed by laboratory testing and controlled on site by measuring the flow time of the mix at 10 minute intervals until flow time increased above 14 seconds, at which stage the grout was discarded.

Enquiries disclosed that there was little information on available on the combined effects of bentonite and common proprietary fluidifying and expansive additives, and it was therefore necessary to carry out trials. Results of the trials are included in Table 1.

TABLE 1

GROUT MIX TESTING SUMMARY

TRIAL MIX No		1	2	3	4	5	RELEVENT TEST STANDARD
COMPOSITION							
WATER	(l)	90	75	75	60	60	
BENTONITE	(gm)	2400	2400	4500	3000	1500	
	%	2.00%	2.00%	3.75%	2.50%	1.25%	
CEMENT	(Kg)	120	120	120	120	120	
	w/c	0.75	0.625	0.625	0.5	0.5	
FEBGROUT	(gm)	375	375	375	600	600	
	%	0.31%	0.31%	0.31%	0.50%	0.50%	
SPECIFIC GRAVITY		1.56	1.65	1.70	1.75	1.85	
FLOW TIME	0mins	9.5	10	11	12.2	11.1	ASTM 1725ml FLOW CONE
	10mins	9.5	10	11.9	11.7	10.9	
	20mins	9.5	10	12.5	12		
	30mins	9.5	10	13	12		
	40mins	9.5	10	13	12.4	11.4	
	50mins	-	10	13.5	14	11.5	
	60mins	-	10	14.5	14.2		
LAB TESTS							
BLEED	%			0.50%	0.50%	1 to 1.5%	NZS 3112:1986
EXPANSION	%			0.00%	1 to 1.5%	1 to 1.5%	NZS 3112:1986
SETTING TIME 1	initial (hours)				05:05	05:30	NZS 3122:1990
	final (hours)				08:05	08:40	
COMMENTS		flow too fast	flow too fast	no expansion	OK	OK	

A gravity header tank system was used to control grout delivery pressures and a header tank system provided to control and measure outflow from bleed ports. During grouting, a manifold system was used that enabled ports to be used either for injection; or for bleed to atmosphere (to check grout return consistency); or for bleed under balanced pressure to a header tank. These arrangements are shown schematically in Figure 4. The bleed header tank was a suspended 40 litre drum which was drained out periodically. Return volumes were measured by recording levels in the return header tank. The grout header tank was a 9.8 m long 80mm diameter pipe suspended from the casing. Very small volumes of grout take could be measured by recording level drops in the grout header tank.

Grout lines were 25mm bore pvc with a 500mm long clear plastic 'sight glass' near the port . Valves in the grout lines were 20mm ball type stop cocks and connections were of the self-sealing Camlock type.

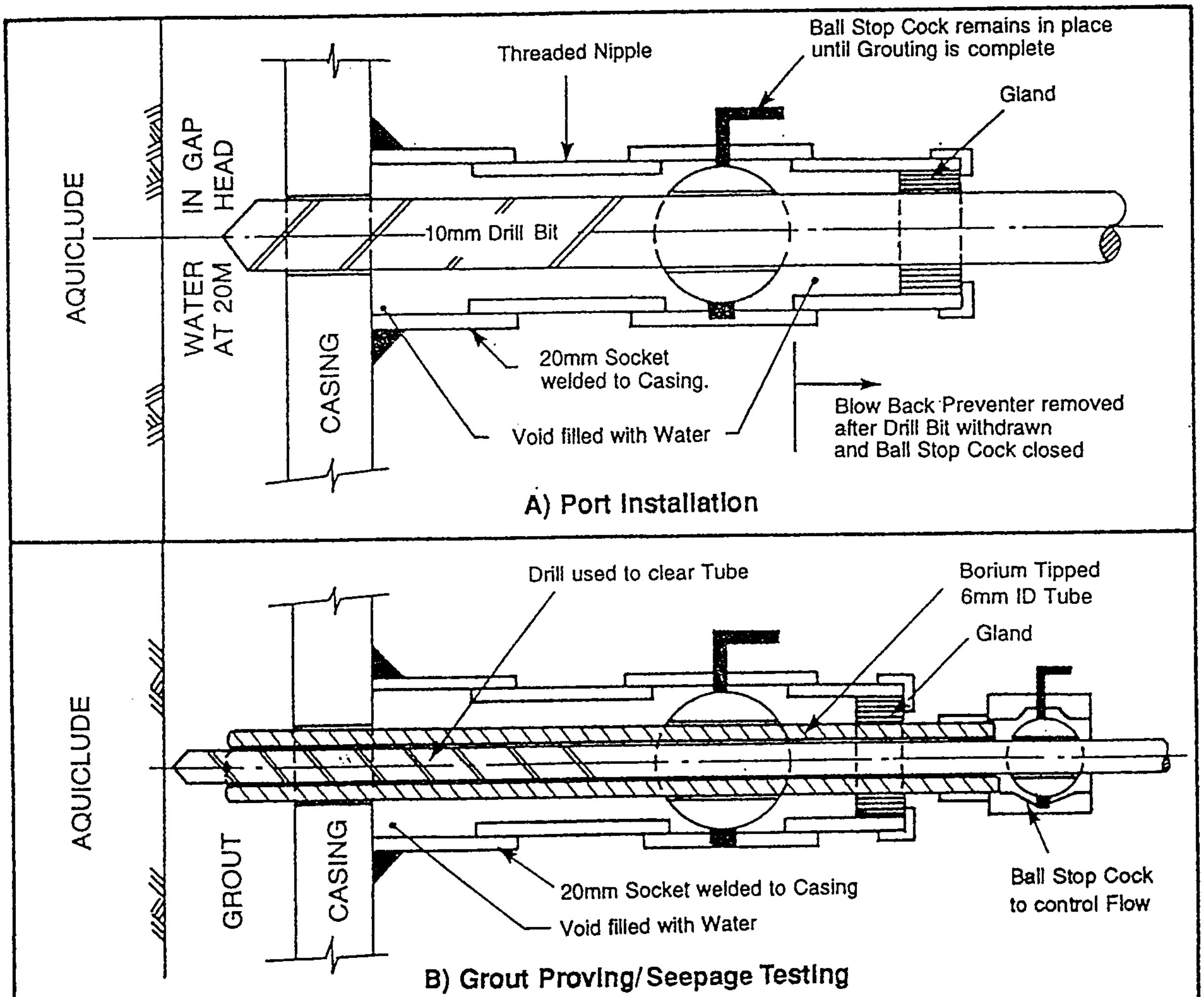

Figure 5: SPECIALISED EQUIPMENT DEVELOPED FOR PROJECT

7 Grout Port Installation

The grout ports were created by welding onto the inside of the casing a socket to which a ball valve was then fitted. A blow back preventer device, specifically developed for the project, was then connected to the ball valve. The device, shown in Figure 5, enabled a hole to be drilled through the casing, using a long drill extending through a gland and then a water filled compartment. This ensured that the high pressure water (200kPa) outside the casing would not flow in after the casing was penetrated. After drilling the hole, the drill was withdrawn through the ball valve, which was then closed allowing the blow back preventer to be removed for use on the next port.

A second device also shown in Figure 5, was specifically developed for drilling through a port after grouting to check the grout thickness and conduct controlled seepage tests in the ground beyond the grouted gap. With this device, a 10 mm OD x 6mm ID tube with a borium faced end was drilled through the grouted port using a blowback prevention arrangement similar to that for port installation. The tube was then removed and the grout sample extracted. After reinserting the tube, a drill was used to clear the tube before performing the seepage tests. After removal of the drill, flow out of the cleared hole was measured under control using a ball valve on the tube. It was possible with the device to drill in the tube in several stages and check seepage at a number of depths beyond the back of the casing.

8 Grouting Sequence and Procedures

The construction of the bridge foundations was carried out under Resource Management Act (RMA) Consents which were subject to Conditions, one of which was that any departure from the specified construction method would require the approval of the Wellington Regional Council (WRC) Manager Consents. The alternative methodology proposed (which included the grouting approach described in this paper) was a departure from the original specification, and hence the methods required WRC approval. The initial reaction of WRC was that a new RMA Consent Application might be required. However, after protracted discussions the alternative proposed was accepted and a grouting procedure agreed, although with a substantially greater number of grout ports than was considered necessary by the designers. WRC accepted that, if grouting of the first pier suggested that a lesser number of ports would have been effective, then, proposals for reduced number of ports on subsequent piers would be considered.

Full Grouting Procedures Manuals were developed after grout mix trials and the development and simulated testing of equipment specifically developed for the project. For grouting purposes, the aquiclude to casing interface was divided into three levels as shown in Figure 6. Level II was set to encompass the Upper Aquifer which, because of its higher permeability, had different grouting characteristics to Levels I and III.

Each section was bounded at top and bottom by shoes and collars to limit grout travel. The general configuration of casings and sealing bands/shoes is shown in Figure 6. SPT tests were performed in the proving holes at 500 mm centres through zones where sealing bands were to be located to determine the appropriate elevation of the bands for each individual pier.

Grouting was performed in a closely defined procedure with comprehensive readings of grout pressures and takes through each port. Cellular phones were able to receive and send from 25m down the casing and were used for communications between the BCHF site supervisors and the Geotechnical Engineer based in the office. At one minute intervals, surface staff recorded fluid levels in header tanks while staff down the casing recorded the status of each valve (open or closed). From these records, which were recorded in the field directly onto preformatted sheets, it was possible to reconstruct the pattern and volumes of grout or water take from each injection port and the associated return of water or grout from bleed ports.

Prior to each grout injection operation a trial was carried out with dyed water to check groundwater pressure behind the casing and enable the flowpath connections between ports to be established.

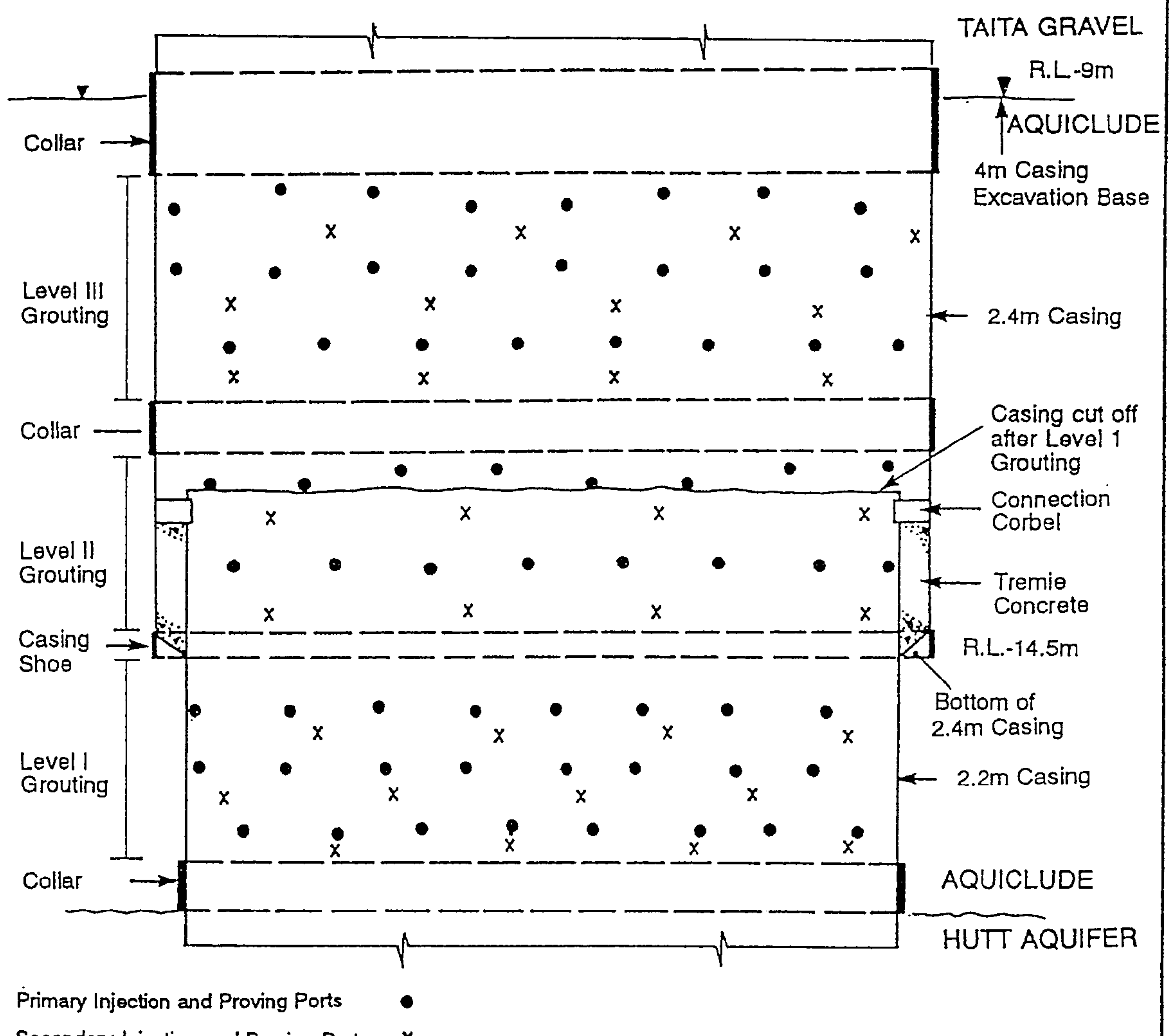

Figure 6: FULL GROUTING PORT PATTERN

For the first pier grouted using these procedures, a pattern of primary and secondary injection ports indicated in Figure 6 was adopted. As discussed above this pattern contained more ports than was considered necessary by the designers.

After grouting of the first pier it was apparent that satisfactory results could have been obtained with far fewer ports. WRC agreed that on subsequent piers a reduced pattern (with 50% of the ports) could be initially used on the remaining piers provided that detailed checking procedures were undertaken to ascertain that the gaps had been adequately filled with grout. Wherever the checks did not conclusively show whether a gap existed or that all gaps had been filled with grout, the procedures required installation of the full grout pattern shown in Figure 6.

Decision trees were agreed with WRC for establishing whether or not reduced port arrangements were acceptable. Figure 7 shows a typical example. The decision trees accepted were more demanding than the designers considered necessary.

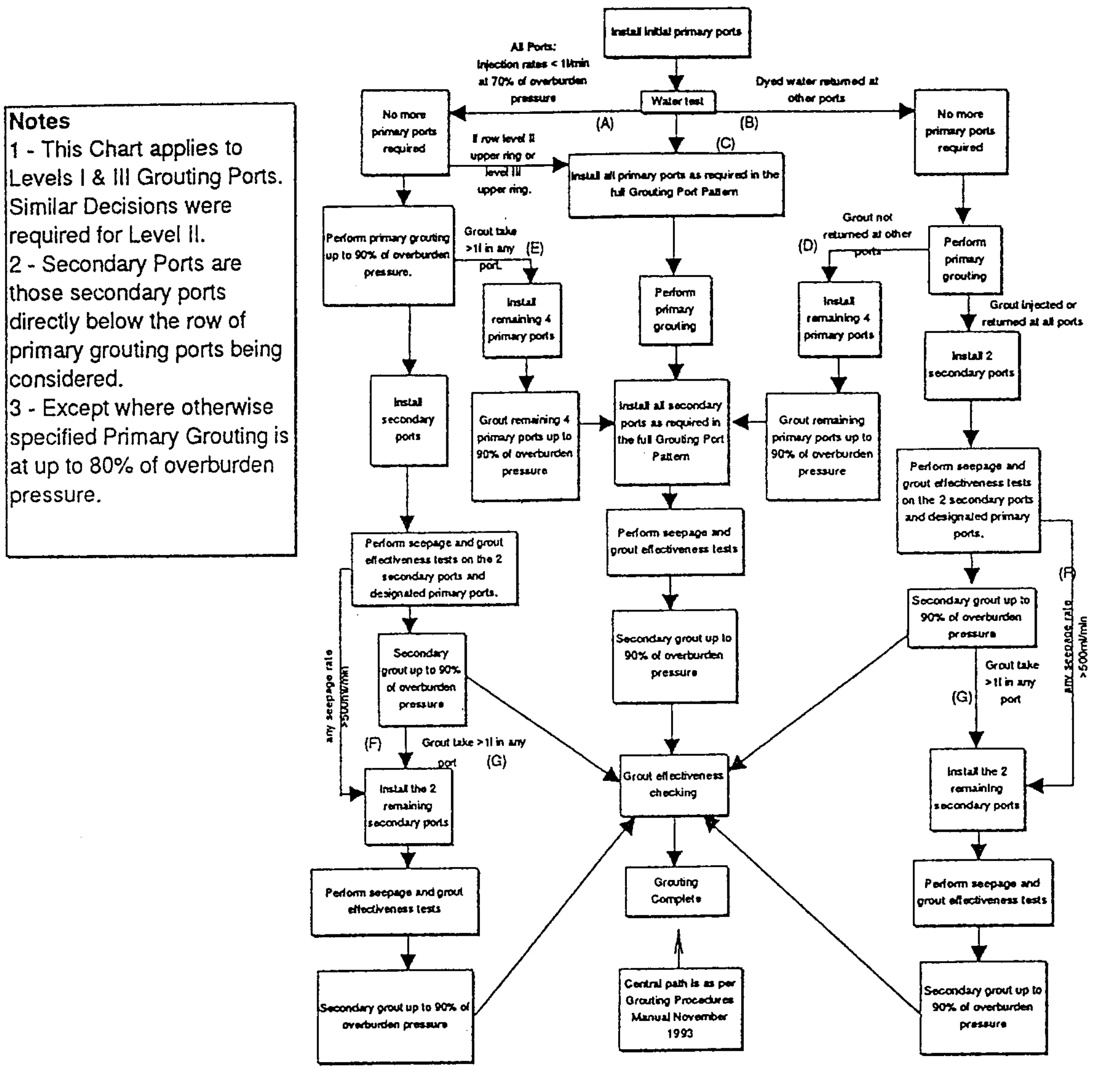

FIGURE 7. TYPICAL GROUTING DECISION TREE

9 Grouting Results

The grouting exercise was successfully accomplished on four piers without any major difficulties. Each pier and each grouting Level behaved slightly differently. The different gradings and permeabilities resulted in different behaviours of the aquiclude and Upper Aquifer materials both with respect to grout patterns and also the extent of the gap between the ground and the casing. In the dense and silty aquiclude material, the gap left behind the driving shoes appeared to be almost the full width of the shoe (16mm), and the ground would not take either water or grout injection. By contrast the Upper Aquifer, which was a cleaner coarser sand, appeared in many instances to have collapsed back against the casing after the passage of the driving shoe. Also the Upper Aquifer, while too fine to take grout, took water readily.

Reduced procedures were applied at approximately 50% of locations. On several piers there were substantial takes on the lowest ring of ports, attributed to the existence of a flow path downwards past the collar on the casing to the Hutt Aquifer. In these cases the initial grout was allowed to go off, after which grouting of the ring above was completed with normal takes.

On Level I and III the grout takes measured indicated that a gap with a width close to the thickness of the casing shoe/collar had been filled. In a number of cases the gap was sufficiently continuous that water testing and primary grouting was accomplished using only one or two injection ports to produce return flows at all other primary ports.

On Level II no returns of dyed water were achieved during the water tests and it was concluded that the injected dyed water was lost into the relatively permeable Upper Aquifer. During primary grouting on Level II, grout takes were variable with no bleed return of grout in some instances. The absence of returns and the generally smaller grout takes compared with those at Levels I and III were attributed to the less cohesive material in Level II relaxing back against the casing.

On Level I it was possible to positively check the effectiveness of the grouting by lowering the water level in the 2.2/2.4 m casing annulus and observing whether the water level returned to the aquifer head level. On several piers there was still an apparent leakage path from the Upper Aquifer. As in all cases rates of water level rise in the annulus could be attributed to seepage through the aquiclude, the test demonstrated that the Level I grouting had effectively blocked any leakage paths up the outside of the casing from the Hutt Aquifer.

The grout effectiveness testing demonstrated the presence of grout behind the casing at virtually all locations over Levels I and III after the primary grouting. At Level II, where the ground was the less cohesive and more permeable Upper Aquifer, grout was not always present and the flow measurements from behind the casing through the 6mm ID tube were in the range 0 - 2 litres per minute (which is small for a head difference across the casing of 20m). At Levels I and III the seepage test flows were less than 500 ml/minute and flows were clear water. This demonstrated the absence of any paths directly connecting the tested areas to the Hutt Aquifer or Upper Aquifer.

Observations were made of whether the flow from the seepage testing contained sand or not and, where it did, the port was immediately closed. These locations were generally at level II or at the few instances at Levels I and III where grout was not present, suggesting an isolated ungrouted area behind the casing. Testing was repeated at distances up to 100mm behind the casing without any significant increases in flows.

Secondary grouting with a few exceptions resulted in very small takes. Significant takes were only experienced in the few locations where the effectiveness testing did not detect grout from the primary grouting and the flow from the tube contained sand and was closed off after only a few hundred millilitres had been allowed to flow.

10 Conclusions

Project specific grouting procedures were successfully developed and applied to grout the gap left outside a casing with an external shoe driven through a confining aquiclude into an artesian aquifer. Systems were developed for installing grout ports without releasing flows from the high pressure water outside the casing. Methods for determining the presence of grout and the effectiveness of the grout in sealing any leakage paths up the outside of the casing were also developed and applied. As the grouting was a departure from the specified pier construction methodology which was referenced in a Condition to the Consents, it was nececessary to obtain the approval of WRC as the RMA Consent Authority. The development and documentation of the grouting procedures to the satisfaction of the RMA Consent Authority was a major exercise, and while the basic procedure adopted was as proposed by the designer, it was necessary to use more ports than the designer considered necessary in order to obtain the approval.

11 Acknowledgments

The authors acknowledges the permission of the Hutt City Council to the publication of this paper and also the contributions made to the development and implementation of the grouting procedures by the foundation subcontractor (Richardson Drilling Company) whose staff provided many of the ideas for the special equipment developed and co-operated wholeheartedly in the execution of the work under BCHF direction. The authors also acknowledge the invaluable advice on mix design and mixing equipment provided by Mr R W Irwin.

12 References

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LIMIT STATE DESIGN OF FOUNDATION AND RETAINING WALLS

STUART PALMER
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1 INTRODUCTION

A series of seminars took place in Christchurch, Wellington, Auckland and Taupo during November last year to present a method and parameters for limit state design of foundations and retaining walls. Some of the information presented at those seminars was published in the last issue of Geomechanics News.

In the following discussion a comparison of the effective overall factors of safety produced by the limit state design methods proposed at the seminars is made with the overall factors of safety which have been commonly used. On the basis of this comparison some amendments to the limit state design method are suggested.

2 COMPARISON OF FACTORS OF SAFETY

In the following table the overall factors of safety provided by the proposed ultimate limit state design method are presented. These overall factors of safety have been estimated by combining load factors and strength reduction factors where appropriate and by working a limited number of examples. The table also presents the overall factors of safety commonly used in geotechnical engineering.

Table 1: COMPARISON OF FACTORS OF SAFETY

Situation	Proposed Limit State Design			Commonly Used Overall FOS
	NZS 4203:1992 Load Combination	Strength Reduction	Effective Overall FOS	
1 SHALLOW FOUNDATIONS				
(a) Static Loading	1.4G, 1.2G + 1.6Q	0.5	2.8-3.2	3
(b) Seismic Loading	G + Qu + Eu	0.5	2	2
(c) Capacity Design	Actions as calculated	0.9	1.1	1.1
2 PILE FOUNDATIONS				
2.1 Static Analysis				
(a) Static Loading	1.4G, 1.2G + 1.6Q	0.4-0.65	3.8-2.1	3
(b) Seismic Loading	G + Qu + Eu	0.4-0.65	2.5-1.5	2
(c) Capacity Design	Actions as calculated	0.9	1.1	1.1
2.2 Driving Formulae				
(a) Static Loading	1.4G, 1.2G + 1.6Q	0.45-0.55	3.3-2.7	3
(b) Seismic Loading	G + Qu + Eu	0.45-0.55	2.2-1.8	2
(c) Capacity Design	Actions as calculated	0.9	1.1	1.1
2.3 Static Load Tests				
(a) Static Loading	1.4G, 1.2G + 1.6Q	0.65-0.9	2.1-1.6	2
(b) Seismic Loading	G + Qu + Eu	0.65-0.9	1.5.1.1	1.5
(c) Capacity Design	Actions as calculated	0.9	1.1	1.1

Situation	Proposed Limit State Design			Commonly Used Overall FOS
	NZS 4203:1992 Load Combination	Strength Reduction	Effective Overall FOS	
3 RETAINING WALLS				
3.1 Gravity Retaining Walls				
(a) Overturning	0.9G, 1.6P*	-	1.8	2
(b) Sliding	0.9G, 1.6P	0.8	2.0-2.2	1.5
(c) Bearing	0.9G, 1.6P	0.5	3?-6?	3
3.2 Cantilever with Passive Soil Resistance				
(a) Overturning	1.6P	0.5	3.2	2

*P - lateral soil (and water?) pressure. All other symbols as per NZS 4203:1992

3 DISCUSSION

The comparison of factors of safety presented in Table 1 is discussed in the following sections.

3.1 SHALLOW FOUNDATIONS

In 1976 Peter Taylor proposed factors of safety to be used in conjunction with NZS 4203:1976 factored loads to give overall factors of safety equivalent to those traditionally used. Peter Taylor's factors of safety became widely accepted. What has been proposed here is similar (refer Table 1). The approach and the values of strength reduction factors proposed are considered appropriate and it is suggested that it be extended to include wind and other load combinations specified in NZS 4203:1992.

In the latest amendment to "Verification Method B1/VM4" (Amendment 2, 19 August 1994) (contained within the BIA Approved Document, B1 Structure) a strength reduction factor of Ø = 0.8 (FOS = 1.3) is proposed instead of the previously used FOS = 1.1 (Ø = 0.9). The selection of Ø = 0.8 or 0.9 remains a point of debate. It is noted that the level of conservatism applied in selecting the soil or rock strength parameter may have a greater influence on the final design than the selection of Ø = 0.8 or 0.9.

3.2 PILE FOUNDATIONS

The Ø values (strength reduction factors) proposed at the seminars were taken from the Draft Australian Piling Standard CE/18, September 1994. A range of Ø values was given for each of 12 different methods of assessing a pile ultimate capacity. In table 1 above these Ø values have been summarised in three groups (ie, ultimate capacity assessed by:

- static analysis
- driving formulae
- static load test

The Draft Australian Piling Standard also provides guidance in selecting the Ø value within each range.

The complexity of this system is questioned. The geotechnical engineer has a great deal of flexibility in applying his or her judgement to select the parameters used in determining the pile's ultimate capacity. It therefore seems to be contradictory and unnecessary to dictate Ø values in this "cook book" fashion. It is suggested that a simpler selection of Ø values may be more appropriate. Such a simpler selection of Ø values is presented in the conclusions below.

With reference to Table 1 it is noted that the effective overall factors of safety (determined by combining the NZS 4203:1992 load factors and the proposed Ø values) range to considerably lower values than those commonly used.

The Ø values suggested in the conclusions below provide overall factors of safety consistent with those commonly used.

The comments made in 3.1 above (shallow foundations) with respect to a ∅ = 0.8 or 0.9 for capacity design also apply for pile foundations.

3.3 RETAINING WALLS

(i) Gravity Retaining Walls

The overturning and sliding procedures proposed at the seminars have been assessed by combining load factors and strength reduction factors to give overall factors of safety (refer to Table 1). It is noted that the overturning analysis is less conservative than that previously commonly used and the sliding more conservative.

The factoring of the lateral soil pressure in the bearing capacity assessment makes it very difficult to compare the proposed method with that traditionally used. A few examples have been worked and it has been found that the effect of the factored lateral pressure is to reduce the effective width of the eccentricly loaded foundation. The effect of this significant, particularly for cohesionless soils. The proposed procedure will produce some designs quite different to traditional methods. The proposed procedure for the geotechnical design of gravity retaining walls is therefore not considered appropriate.

(b) Cantilever with Passive Soil Resistance

This relates to timber pole retaining walls and sheetpile walls, etc.

There are a number of methods in common use for the analysis of such walls, eg:

- Passive resisting moments > FOS x active mobilising moments
- Net passive moments > FOS x net active moments
- Factored depths of embedment
- Partial factors of safety
- Broms for ultimate capacity of laterally loaded pile

The factors of safety calculated depend on the method of analysis used. To quote Mick Pender (refer issue 48 of Geomechanics News) this is a case of "apples, oranges and factors of safety". We cannot specify factors of safety, or load factors and strength reduction factors without specifying the method of analysis to be used. Therefore if we wish to apply ultimate limit state design to these types of retaining walls Ø values need to be specified for each method of retaining wall analysis. This is similar to what is proposed in the case of piles.

(c) Structural Design

Previous versions of NZS 4203 have used a load factor of 1.7 for lateral earth pressure which has been applied in structural design but not geotechnical design. It is considered that it is appropriate that we continue to use a similar load factor (1.7 or 1.6) in structural design.

4 CONCLUSIONS

The following amendments to the limit state design methods presented at the seminars are suggested to address the concerns outlined in the foregoing discussion. These suggested amendments are generally consistent with the August 1994 amendment to B1/VM4 (contained within the BIA approved document, B1 Structure).

4.1 SHALLOW FOUNDATIONS

The Ø values proposed at the seminar are considered appropriate for application in conjunction with all the factored load combinations presented in NZS 4203:1992 except those including earthquake overstrength (capacity design). Those Ø values were 0.5 where the resistance is provided by mobilising soil or rock shear strength and 0.9 where the resistance is provided by the dead weight of the foundations or soil.

The Ø value to be applied in capacity design deserves further debate. This debate should consider how the soil or rock strength parameters are selected in design and also the mode of failure, ie, would the foundation failure be brittle and result in collapse of the structure or would it be ductile with limited movement?

4.2 PILE FOUNDATIONS

The following Ø values are suggested for use in combination with all NZS 4203:1992 factored load combinations except those including earthquake overstrength (capacity design). The effective overall factor of safety provided for the static case is given in the table for comparison.

Table 2: SUGGESTED Ø VALUES FOR PILE FOUNDATION DESIGN

Method of Assessment of Ultimate Geotechnical Capacity	Strength Reduction Factor	Effective Overall FOS Static Case 1.4G, 1.2G + 1.6Q
Static Analysis	0.5	3
Driving Formulae	0.5	3
Static Load Test*	0.7*	2

* This assumes not less than 3 or 5% of piles (whichever is greater) are tested.

The comments on Ø values for capacity design presented in 4.1 for shallow foundations equally applies here for pile foundations.

4.3 RETAINING WALLS

The use of a load factor of 1.6 on lateral earth pressure and 1.2 on lateral water pressure is suggested as being appropriate for the structural design of retaining wall members. The water level chosen by the designer for assessing lateral water pressure must be one which is unlikely to be exceeded during the life of the wall.

It is suggested that the geotechnical design of retaining walls should continue to use the traditional methods and associated overall factors of safety until appropriate alternative limit state design state design methods are evolved. The ultimate limit state design methods proposed at the seminars are not considered appropriate for the geotechnical design retaining walls.

It is understood that SESOC and NZGS are working on some guidelines for the design of retaining walls. Ultimate limit state design should be considered by that group.

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EXPERIENCE OF GEOTECHNICAL LIMIT STATE DESIGN IN RUSSIA

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INTRODUCTION

Limit state design in geotechnical engineering has become an issue which is widely discussed in New Zealand and therefore I thought that it could be interesting for New Zealand Geotechnical and Structural Engineers to learn about the limit state design approach we use in Russia. Until 1994 I worked as an Associate Professor at the Chelyabinsk State Technical University in Russia and was involved in preparation of several Russian standards on foundation settlement calculation and laboratory test methods.

Before 1960 a total unified factor of safety was used in geotechnical calculations but it became evident that possible variability of strength parameters of soils, loads and working conditions, different importance of structures and buildings (or consequences of their failure) and the different extent of simplification of design models could not be assessed correctly in one total factor of safety.

From 1960 the Limit State Design approach based on partial factors of safety became widely used in Russia and was adopted and remains mandatary in our building code. The main principles of the Limit State Design and a very brief history of the development of shallow foundation limit state design methods in Russia are given below.

ULTIMATE LIMIT STATE DESIGN (ULSD)

ULSD is the limit state design method where the ultimate bearing capacity of foundations is compared with the applied loads.

The main design condition is:

$$\sum N_n \gamma_{f1} \gamma_{ts} \leq Q \left[A, \frac{r_n}{\gamma_{m1}} \frac{\gamma_1 \gamma_{d1}}{\gamma_n} \right] \tag{1}$$

Where $\sum Nn\, \gamma_{f1}\, \gamma_{ts}$ - design loads combination (or factored loads);

$$Q\left[A, \frac{r_n}{\gamma_{ml}} \frac{\gamma_1\, \gamma_{dl}}{\gamma_n}\right]$$ - reduced or allowable bearing capacity function;

Nn - applied loads;

γ_{f1} - load factors for ULSD which allow for the variability of the applied loads;

γ_{ts} - combination coefficients which reduce loads taking into account probability of simultaneous application of their maximum values;

A - geometric parameters of a foundation;

r_n - soil strength parameters;

γ_{m1} - soil safety factors which depend on the variability of soil strength parameters obtained during field and laboratory testing;

γ_n - structure reliability factor which depend on industrial importance of building or structure and consequences of its failure;

γ_1 - working condition factor for ULSD which depends on the complexity of soil conditions and method of manufacturing of the foundation;

γ_d - reduction factor which allows for inaccuracy of adopted design model (simplification of the soil-superstructure interaction, inaccuracy of chosen calculation methods).

This approach assesses all possible inaccuracies at different stages of the design procedure and therefore makes it possible to achieve an appropriate level of reliability of the structure. It is interesting to note that the effective overall factor of safety produced by this method is generally in the range of 2 to 3.

For a long time ULDS was the main design procedure which governed the size of foundations. However, in more recent time a change in the design philosophy has developed whereby for shallow foundations only serviceability limit state (SLSD) is considered. ULSD of shallow foundations is now only considered in the following specific cases:

1 Foundations loaded with large horizontal forces or overturning moments.

2 Foundations of buildings and structures located on slopes.

3 Foundations on soft saturated clays and silts (undrained failure).

4 Foundations on rock.

5 Foundations subject to seismic loading.

Although the Russian code requires that for foundations listed above ultimate limit state must be checked, the sizes of the foundations very often are governed by the SLSD.

All other foundations are designed only on the basis of Serviceability Limit State Design.

SERVICEABILITY LIMIT STATE DESIGN (SLSD)

Figure 1 presents the load-settlement diagram for a vertically loaded shallow foundation (see Figure 1), where Q is ultimate bearing capacity, Qa - allowable bearing capacity (in the case of the total FOS approach Qa = Q/FOS), S_t - tolerable foundation settlement which is governed by the structural flexibility of a building or structure.

From Figure 1 it is obvious that for foundations which transmit mainly vertical load on the subgrade the failure or ultimate limit state does not represent a real danger because the load Q_t at which the ultimate tolerable settlement S_t is exceeded is normally much less than the ultimate bearing capacity Q. If S_t is larger than Sa (where Sa is foundation settlement at load Qa) Ultimate Limit State Design can lead to a very conservative foundation design.

To allow the efficient use of foundation material and more complete utilization of soil strength, the SLSD should be used.

The main SLSD principle can be written as:

$$S_i \leq S_{ti}, \qquad (2)$$

Where

S_i - predicted deformations of foundation, for example S_1 - settlement of a particular foundation, S_2 - average settlement of the building or structure, S_3 - differential settlement, S_4 - tilt, S_5 - horizontal displacement of a foundation etc;

S_{ti} - respective tolerable deformations of a foundation.

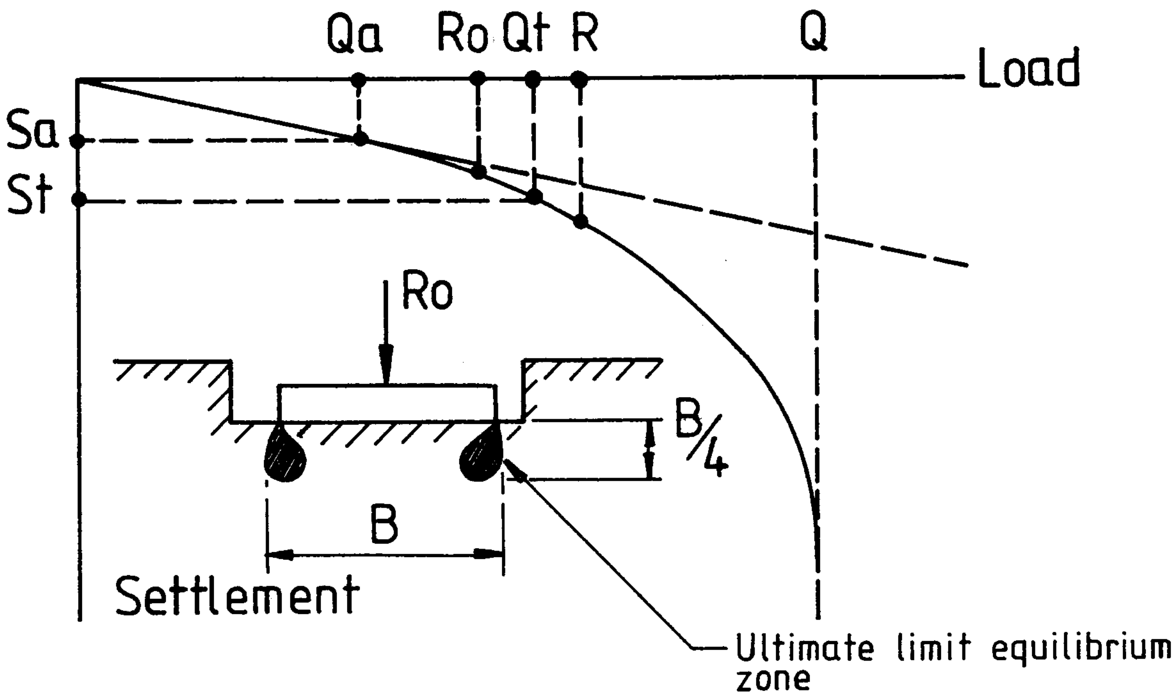
Although the Finite Element method is now widely used in Russia and elsewhere it is still time consuming and therefore sometimes it is not practical to use complicated design models to take into account superstructure - foundation - subgrade interaction in SLSD. To simplify the problem Russian codes allowed predicted deformations calculated on the basis of simple foundation - subgrade interaction models to be compared with tolerable deformations. A standard specification of tolerable deformations for the design of buildings and structures was developed in Russia using the results of long term monitoring of the settlements of many different buildings and structures.

The most complicated part of the SLSD is settlement prediction. Although the main principles of settlement calculations are very similar in all countries, the Russian geotechnical school has a slightly different approach to this problem. In 1930 we started to use theory of elasticity to predict stress-strain state of the soil beneath foundations. Although on the West the theory of elasticity was used with great caution, in Russia during the 55 years since 1930 a system of geotechnical standards based on the theory of elasticity was developed. Two main problems were dealt with to bring the theory of elasticity into practical use.

The first problem was how to restrict the load at which it is still possible to assume that the settlement - load curve is approximately straight line and the theory of elasticity is applicable. In other words to identify the load up to which we can confidently use the theory of elasticity. (It must be noted that this problem exists as long as we use this theory and immediately disappears if nonlinear soil models are used).

Initially this confidence limit load R was conventionally assumed to be equal the load R_o at which the ultimate limit equilibrium zones (or zones where the shearing stresses in the soil elements are equal to their ultimate value) beneath the edges of a foundation have the depth of B/4 (see Figure 1), where B is foundation width. Later on this load R was refined on the basis of numerous comparisons between predicted and actual monitored settlements. In the new geotechnical standard the confidence limit load R varies from $1.1R_o$ for soft clays to $1.9R_o$ for dense gravels.

Figure 1: LOAD-SETTLEMENT DIAGRAM FOR A VERTICALLY LOADED SHALLOW FOUNDATION



This restriction imposed by the theory of elasticity led to a paradoxical situation when during SLSD not only deformations but bearing pressures must be checked:

$$\sum N_n \gamma_{f2} \gamma_{lc} \leq R \left[A, \frac{r_n}{\gamma_{m2}}, \frac{\gamma_2 \gamma_{d2}}{\gamma_n} \right], \tag{3}$$

Where factors γ_{f2}, γ_{m2}, γ_2, γ_{d2} have the same meaning as factors γ_{f1}, γ_{m1}, γ_1, γ_{d1} in condition (1) but have values which are different from those used for ULSD. For example $\gamma_{f2} = 1.0$ while for ULSD $\gamma_{f1} = 1.1 - 1.4$; factors γ_{m2} are calculated on the basis of statistical analysis of test data for level of confidence of 0.85 while level of confidence of 0.95 is used to calculate γ_{m1}.

Second problem was the method of determining "elasticity" modulus (or deformation modulus as it is called in Russia because the elastic part of the soil deformation is normally small) and to adjust the theoretical solutions obtained from the theory of elasticity to practical purposes. It must be noted that in the Russian approach this deformation modulus is used to calculate only final total settlements and therefore immediate, consolidation and secondary settlement can not be calculated separately if the modulus is employed.

Russian research has shown that when the deformation modulus is obtained from oedometer tests the predicted settlements are normally much higher than the actual monitored settlements. Better settlement prediction is obtained if the deformation modulus is derived from field plate test. However even in this case it is necessary to correct the elastic half-space model by assuming that only those layers of soil (zone of influence) where vertical stress σzp due to the external load is still quite large in comparison with overburden pressure σzg are compressed.

It is normally assumed that $\sigma\ zp = 0.1 - 0.2\ \sigma\ zg$ in the deepest layer of the zone of influence. This assumption was proved by field test results which showed that the layers of soils where external load does not cause any vertical deformation (or the deformation is negligible) are located at the more shallow depth then it is predicted by elastic half-space models.

This artificially corrected elastic half-space model gives the settlement prediction with satisfactory accuracy but for broader slab foundation (with width of more than 10 m) the more accurate settlement prediction can be obtained if the model in the form of elastic layer on absolutely rigid stratum is used.

TEST METHODS

The test methods used in Russia are similar to New Zealand test methods. Some typically used tests are: field plate tests (0.5 m^2 plate in pits or 0.06 m^2 plate in boreholes are normally used), field shear test (where three large soil samples left in place during the pit excavation of a pit and encased in a steel box and then sheared along the subgrade surface at different vertical loads), dynamic penetration test (similar to SPT), static penetration test, shear vane test, pressuremeter test and geophysical methods. Laboratory tests frequently used are: triaxial test, shear test, cone penetration test.

Probably the only basic difference between Russian and New Zealand approaches is that in Russia more attention is paid to the accuracy of deformation modules and strength parameters measurements because all foundations are designed on the basis of SLSD. Therefore it is mandatory according to Russian standard to perform field plate and shear tests plus laboratory tests for all major projects. In other words only direct measurements of soil parameters is accepted rather than use of any correlations. If the category of building or structure importance (given in standard description) is of a lower level the Russian codes do not require time consuming field plate and shear tests. In these cases only static and dynamic penetration tests plus laboratory tests are performed and many different correlations can be applied to obtain deformation and strength parameters of soils. Some of these correlations are similar in concept to the correlations which are used in New Zealand. However in Russia the codes also specify a number of additional correlations between oedometer deformation modulus and plate deformation modulus for different types of soils and several other correlations between physical and mechanical soil properties.

DISCUSSION AND ACHIEVEMENTS

Although for more than 60 years many Russian scientists and engineers criticised the theory of elasticity because it did not take into account non-linear soil properties, time factor, scale factor and many other effects, no one was able to suggest anything better for routine design. As a result SLSD approach based on elasticity theory was refined and improved on the basis of new monitoring data and theoretical investigations and by 1974 became well established in a major system of standard codes for foundation design in Russia.

When this approach was used in Russia for the first time in 1960 to design strip footings for multistorey buildings, it was discovered that the allowable bearing pressures could be significantly increased and the footing width could be reduced. In some design cases the footing width was reduced from one half to one third of that calculated on the basis of previously existing standard which adopted total factor of safety approach. In one case bearing pressures beneath strip footings (designed on the basis of SLSD) were increased by 150% and achieved 0.4 - 0.55 MPa instead of 0.2-03 MPa used before. The monitored settlements of these new footings were still in tolerable limits. Using this method 35-40% savings in concrete volume were achieved when hundreds of 12-16 storey houses were built in Moscow. The use of narrow footings enabled not only a considerable saving of concrete but also made it possible in some cases to use precast strip footings instead of expensive pile foundations.

Once the settlement calculation problem was resolved with a reasonably accuracy and, as a result, the soil stiffness coefficients were refined; more attention was directed to the design of continuous slab foundations used when loads were high and the soil was soft and non-uniform. Various foundation - base models were examined for slab foundation design: Winkler's model, elastic half space and elastic layer on absolutely rigid stratum. In order to evaluate the reliability of these methods the settlements and deflections of several multistorey building slab foundations were monitored. The comparison between predicted and monitored slab settlements indicated that model of elastic layer corresponds to the monitoring data much better than all others.

To avoid the complexity of a foundation - subgrade finite element design where subgrade in the form of elastic layer is used and to allow for non-uniformity of the soil in plan, the international stiffness coefficient method has been developed in Russia.

In this method variable stiffness coefficients are established at each point of the subgrade from the relationship between bearing pressure and predicted settlement (at this stage the slab being assumed to be infinitely flexible). After that an iteration calculation technique is used to allow for the slab rigidity. The calculated slab settlements obtained in the first trial are different from those previously used to calculate the stiffness coefficients because of the slab stiffness and therefore several trials with corrected stiffness coefficients are required to establish the distribution of the stiffness coefficients in plan.

Later on non-linear deformations of the soil and reinforced concrete as well as foundation - superstructure interaction were taken into account to refine slab foundation calculation methods.

Initially slab foundations were used in Russia with a great caution and therefore were mainly ribbed-type slabs. However in 1970 a large number of multistorey buildings (for example 22 storey Intourist Hotel in Gorkey Street, Moscow and Television Centre Building in Moscow) were constructed on 1-1.5 m thick ribless slab foundations, and slab foundations became the principal design solution in multistorey building construction practice. A major example is the 61 m diameter ring slab foundation (with a ring width of 9.5 m) resting on a compressible subgrade used to support the 533 m high Ostankinskii television tower design in Moscow.

CONCLUSION

- The Limit State Designed approach based on partial factors of safety has been successfully used in Russia for the last 35 years.

- The SLSD method described above makes it possible to achieve more efficient and economical foundation design.

- The theory of elasticity can be used to predict final total foundation settlements with satisfactory accuracy. Non-linear soil properties make it necessary to restrict the loads for which the theory of elasticity is applicable and to correct elastic half-space model of the soil base by restricting the zone of influence or to use elastic layer model to achieve more accurate settlement prediction.

- Reliable deformation and strength parameters of soils can be obtained from the direct plate and shear field tests for application in SLSD.

ACKNOWLEDGEMENTS

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1995

MAY 28 - JUNE 1, 1995
Copenhagen, Denmark
11TH EUROPEAN CONFERENCE ON SOIL MECHANICS AND FOUNDATION ENGINEERING
Theme: The interplay between geotechnical engineering and engineering geology.

JUNE 4-7, 1995
Lake Tahoe, Nevada, USA
35TH US SYMPOSIUM ON ROCK MECHANICS
Topics: Rock Mechanics and Waste Disposal; Laboratory Testing; Weak Rock Engineering; Slope Stability; Neotectonics; Seismicity; Mining; Rock Dynamics-Drilling and Blasting.

JUNE 11-16, 1995
The Hague, The Netherlands
OFFSHORE AND POLAR ENGINEERING CONFERENCE
Topics: Offshore Technology and Ocean Engineering; Energy and Resources; Geotechnical Engineering Pipelines; Offshore Mechanics; Materials, Tubular Structures and Welding; Polar Engineering and Russian Arctic; Advanced Ship technologies Superconducting Propulsion; Marine Environmental Policy Environmental Techniques.

AUGUST 1-2, 1995
Kuala Lumpur, Malaysia
DAM ENGINEERING '95
Topics: Dam design and construction; monitoring and instrumentation; dam materials; dam maintenance and management; safety and reliability; environmental aspects.

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 2-4, 1995
Brighton, UK.
CHANNEL TUNNEL - ENGINEERING GEOLOGY SYMPOSIUM
Topics: All aspects of tunnelling in chalk and associated rocks, including classification of chalk for tunnelling.

SEPTEMBER 3-6, 1995
Adelaide, South Australia
11TH GEOPHYSICAL CONFERENCE AND EXHIBITION OF THE AUSTRALIAN SOCIETY OF EXPLORATION GEOPHYSICISTS

SEPTEMBER 25-29, 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.

SEPTEMBER 30, 1995
Tokyo, Japan
INTERNATIONAL WORKSHOP ON ROCK FOUNDATION OF LARGE-SCALED STRUCTURES
Themes: Investigation and Testing of Rock Masses; Evaluation of Deformability and Strength of Rock Masses; Design and Analysis of Rock Foundation; Construction and Measuring of Rock Masses.
Topics: Bearing capacity of rock foundation; Deformation of settlement of rock foundation; Dynamic or seismic behaviour of rock foundation; In-situ test or laboratory test of rock masses and rock specimens; Rock mass classification and scale effect; Discontinuity and anisotrophy of rock masses; Stability or deformation analysis of rock foundation.

OCTOBER 4-5, 1995
Linkoping, Sweden
CPT '95, INT. SYMPOSIUM ON CONE PENETRATION TESTING
Themes: Equipment and Testing, Interpretation of Results, Solution of Practical Problems; design of earth structures, deep and shallow foundations, ground supports, slope stability.

OCTOBER 30 - NOVEMBER 4, 1995
Guadalajara, Mexico
10TH PAN AMERICAN CONFERENCE ON SOIL MECHANICS AND FOUNDATION ENGINEERING.

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

NOVEMBER 29-DECEMBER 2, 1995
Auckland, New Zealand
2ND AUSTRALIA-NEW ZEALAND YOUNG GEOTECHNICAL PROFESSIONALS CONFERENCE
(see article earlier in this issue of NZ Geomechanics News)

DECEMBER 11-15, 1995
Cairo, Egypt
XI AFRICAN REGIONAL CONFERENCE ON SOIL MECHANICS AND FOUNDATION ENGINEERING
Theme: Geotechnical Engineering and Community Development.
Topics: Foundation Engineering for low cost structures; Infrastructures for new communities; Problematic soils; Soil improvement; Heritage preservation; Earth embankments; Retaining and buried structures.
Papers: by November 1, 1994.
Languages: English and French

1996

FEBRUARY 16-18, 1996
Hamilton, New Zealand
SYMPOSIUM - "GEOTECHNICAL ISSUES IN LAND DEVELOPMENT"
Topics: Slope Stability, Geotechnical Risk and Hazard Assessment, Local Government control on Land development; Stabilisation and slope improvement (see article earlier in this issue of NZ Geomechanics News).

MAY 6-10, 1996
Kuala Lumpur, Malaysia
12th SOUTHEAST ASIAN GEOTECHNICAL CONFERENCE
Themes: Environmental Geotechnics, Ground Improvement, Foundations in Limestone Areas, Tropical Soils.
Language: English

MAY 14-17, 1996
Tokyo, Japan
2ND INTERNATIONAL CONFERENCE ON GROUND IMPROVEMENT SYSTEMS - Grouting & Deep Mixing
Topics: Engineering Properties of Materials and Improved Soils; Equipment, Execution and Process Control; Design Guideline and Engineering Manual Evaluation; Applications; New Technologies.

MAY 29-31, 1996
Montreal, Canada
GEOFILTERS '96
Topics: Properties relevant to filtration and drainage; Filter and drainage design criteria; Quality control and assurance; Case studies.
Abstract: by February 15, 1995
Languages: English and French

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

JULY 1-6, 1996
Adelaide, South Australia
7TH AUSTRALIA-NEW ZEALAND CONFERENCE ON GEOMECHANICS - GEOMECHANICS IN A CHANGING WORLD
Call for papers: April 1995 with synopses required by July 1995 and final papers by January 1996. (See article earlier in this issue of Geomechanics News)

SEPTEMBER 3-6, 1996
Torino, Italy
EUROCK '96, ISRM INTERNATIONAL SYMPOSIUM - PREDICTION AND PERFORMANCE IN ROCK MECHANICS & ROCK ENGINEERING
Topics: Foundation of dams, bridges, oil field platforms and other large structures; Natural and excavated slopes; Tunnels, oil wells and caverns; Mining structures; Environmental engineering (including fluid-rock interaction, prediction of contamination radioactive waste repositories, subsidence above oil and gas fields, etc.), Historical sites and monuments.

NOVEMBER 5-8, 1996
Osaka, Japan
2nd INTERNATIONAL CONGRESS ON ENVIRONMENTAL GEOTECHNICS
Themes: Site Investigation, Speciation and Characterisation; Modelling and Numerical Analysis; Geotechnics of Mines Waste Management; Geotechnics of Municipal Waste Management; Waste Disposal and Containment; Geotechnical Recycle or Reuse of Waste Materials; Remediation of Contaminated Ground; Dredging and Sediments; Geo-Environmental Risks: Assessment and Mitigation; Regulations: Trends and Vision for the Future.
Language: English

1997

SEPTEMBER 6-12, 1997
Hamburg, Germany
XIV INTERNATIONAL CONFERENCE ON SOIL MECHANICS AND FOUNDATION ENGINEERING
Themes:
Plenary Sessions:
- Soil Testing & Ground Property Characterisation
- Recent Developments in Foundation Techniques
- Retaining Structures and Excavated Slopes
- Underground Works in Urban Environment
- Soil Improvement & Reinforcement
- Waste Disposal and Contaminated Sites

Parallel Sessions:
- Recent Developments in Laboratory Stress-Strain Testing in Geomaterials
- Ground Property Characterisation by Means of Insitu Tests
- Interplay between Physical and Numerical Models as Applied in Engineering Practice
- Soil Structure Infraction for Shallow Foundations under Static Dynamic Loadings
- Design and Performance of Piled Rafts
- Limit States Concept in Design of Shallow and Deep Foundations
- Design Construction and Performance of Anchored Walls and Strutted Excavations
- Large Excavations with Dewatering in Urban Environment
- Subsidence as Related to Various Tunnelling Techniques
- Performance and Monitoring of Underground Works
- Soil Improvements for Tunnel Works
- Deep in Place Mixing Methods including Jet-Grouting
- Use of Geosynthetics and Geotextiles in Geotechnical Engineering
- Pollutants Containment via Passive Barriers
- Active Pollutants Control and Remediation of Contaminated Sites
- Dredging Sludge and Tailings Impoundments
- Teaching and Education in Geotechnical Engineering

Abstracts by 1 Nov. 1995
Manuscripts by 31 Dec 1996
Registration by 30 June 1996

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

PUBLICATIONS OF THE SOCIETY

The following publications of the Society are available from the Secretary, IPENZ, P.O. Box 12241, Wellington North. Some publications have been reduced in price to members to clear excess stocks. All prices exclude postage and GST.

	LIST PRICE		REDUCED PRICE TO MEMBERS
	MEMBERS	NON	
Australia-NZ Conferences on Geomechanics			
Proceedings of the Sixth Australia-NZ Conference on Geomechanics, Christchurch, February 1992	$100.00	$100.00	$50.00
Proceedings of the Third Australia-NZ Conference on Geomechanics, Wellington, May 1980	$ 20.00	$ 30.00	$10.00
Proceedings of the Second Australia-NZ Conference on Geomechanics, Brisbane, July 1975	$ 25.00	$ 25.00	N/A
NZ Geomechanics Society Symposia			
Proceedings of the Wellington Symposium "Geotechnical Aspects of Waste Management", May 1994	$ 25.00	$ 35.00	N/A
Proceedings of the Auckland Symposium "Groundwater and Seepage", May 1990	$ 25.00	$ 45.00	$10.00
Proceedings of the Hamilton Symposium "Piled Foundations", September 1986	$ 20.00	$ 25.00	$10.00
Proceedings of the Alexandra Symposium "Engineering for Dams and Canals", November 1983 (a joint Symposia with NZSOLD)	$ 40.00	$ 50.00	$10.00
Proceedings of the Palmerston North Symposium "Geomechanics in Urban Planning", May 1981	$ 20.00	$ 20.00	N/A
Proceedings of the Wanganui Symposium "Using Geomechanics in Foundation Engineering", September 1972 (xerox copy)	$ 8.00	$ 10.00	N/A
Other Publications			
Guidelines for the Field Description of Soils and Rocks in Engineering Use	$ 10.00	$ 13.00	N/A
"Stability of House Sites and Foundations - Advice to Prospective House and Section Owners"	$ 1.00	$ 1.00	N/A
IEA Guidelines for Provision of Geotechnical Information, etc.	$ 10.00	$ 10.00	N/A
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OBJECTS

(a) To advance the study and application of soil mechanics, rock mechanics and engineering geology among engineers and scientists.

(b) To advance the practice and application of these disciplines in engineering.

(c) To implement the statutes of the respective international societies in so far as they are applicable in New Zealand.

MEMBERSHIP

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

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

ANNUAL SUBSCRIPTION

Annual subscriptions, which include the newsletter are (for 1995)

Members	*(IPENZ members)*	*$25.00*
	(others)	*$40.00*
Students	*(IPENZ members)*	*$15.00*
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International Association of Engineering Geology	*(IAEG)*	*$10.00*
	(with bulletin)	*$37.00*

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

NZ Geomechanics Society
P O Box 12 241
WELLINGTON

The Secretary
NZ Geomechanics Society
The Institution of Professional Engineers New Zealand (Inc)
P.O.Box 12-241
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NEW ZEALAND GEOMECHANICS SOCIETY
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(A Technical Group of the Institution of Professional Engineers New Zealand (Inc))

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

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I wish to affiliate to:

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International Society for Rock Mechanics	(ISRM)	Yes/No
International Association of Engineering Geology	(IAEG)	Yes/No
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DECLARATION: If admitted to membership, I agree to abide by the rules of the New Zealand Geomechanics Society

Signed . . . .
Date . . . .

ANNUAL BASIC SUBSCRIPTION: Due on notification of acceptance for membership, thereafter on 1st of January, student fees are half full membership fees. Please do not send subscriptions with this application form. You will be invoiced on acceptance into the Society

(for office use only)

Received by the Society . . . .
Recommended by the Management Committee of the Society . . . .
Approved by the Council of the Institution . . . .

ADVERTISING IN NZ GEOMECHANICS NEWS

NZ Geomechanics News is published at least twice a year and distributed to the Society's 400 members throughout New Zealand.

This magazine is issued to society members who comprise professional geotechnical and civil engineers and engineering geologists from a wide range of consulting, contracting and university organisations as well as those involved in laboratory and instrumentation services.

Advertisement Location	Single Issue
Back Cover	\$250*2
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