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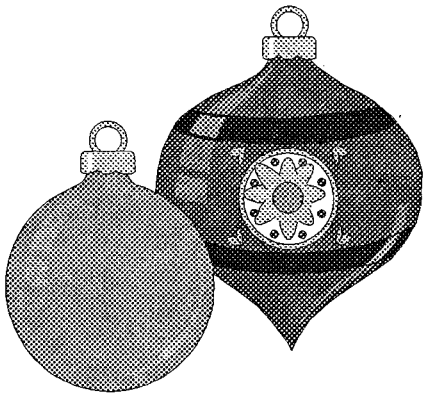
N.Z. GEOMECHANICS NEWS

No. 48

DECEMBER 1994



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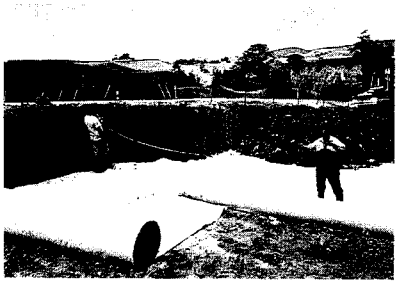

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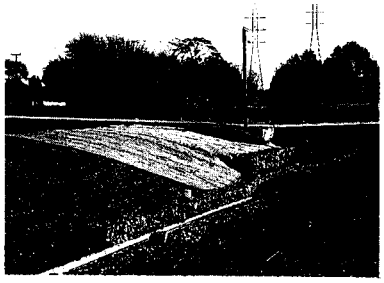

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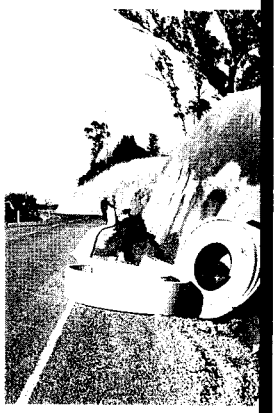

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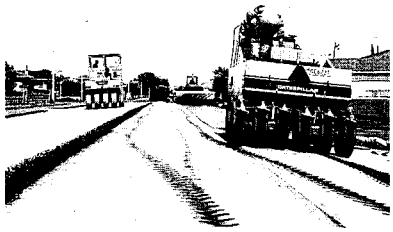

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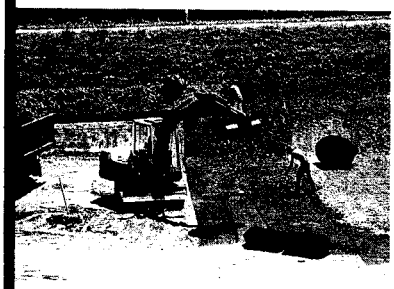

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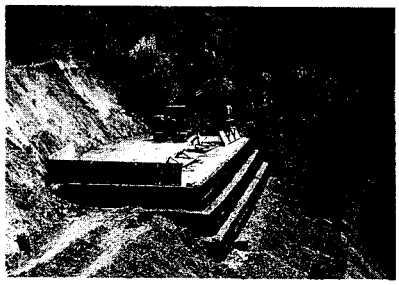

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NZ GEOMECHANICS NEWS

NO.48 DECEMBER 1994

A NEWSLETTER OF THE NZ GEOMECHANICS SOCIETY

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NAME CHANGE

Should the society change its name to reflect modern terminology and the scope of interests now covered by the Society? The Management Committee wants to know your thoughts. Have your say by completing the enclosed questionnaire and returning it to the editor.

GEOMECHANICS NEWS

You will notice changes to the format and style of the magazine. Your impressions and suggestions are welcomed. Is the magazine frequent enough? What sort of articles do you enjoy the most? Take the opportunity to note your thoughts on the back of the Name Change Questionnaire. We are waiting for the results of the Name Change survey before launching a new cover and would like a logo for the magazine (and possibly the Society). The artists among you should send in suggestions with the enclosed Name Change Questionnaire.

SLOPE STABILITY QUESTIONNAIRE

The majority of this issue is devoted to the results of a survey on approaches to slope stability assessments for land development. A questionnaire was sent to every Territorial Local Authority and to the principal geotechnical engineering and engineering geological consultants in NZ. The aim was to initiate a debate on the subject and the results are most interesting. We expect further correspondence on this issue from especially from those who did not take the opportunity to present their views. Thank you to all who responded in the short time frame available. The debate will serve as a useful run-up to the next Symposium in February 1996 on "Geotechnical Issues in Land Development".

LIMIT STATE DESIGN

Six readers of Geomechanics News qualify for chocolate fish, for reading the Pender and Anderson paper on Limit State Design published in the last issue. In trying to read the pages, they deduced that several pages were missing and contacted the editor. To help those readers still trying to understand Limit State Design, the paper is published again in this issue, in full form.

Also included is a review of the recent seminars/workshops on limit state design held throughout the country.

Merry Christmas and best wishes for a happy and prosperous New Year. Take this magazine with you to the beach.

Geoffrey Farquhar & Steve Crawford
EDITORS

PAST NZGS ACTIVITIES

As 1994 draws to a close there are a number of matters of interest to members on which I wish to comment. The past two years have been a period of consolidation and new initiatives for the Society, following the intense focus on the ISL and ANZ conferences in 1991-92. The financial surplus resulting from those efforts has, of course, facilitated such new initiatives as the recent Limit State Workshop, and we expect to run similar workshops in the future either by ourselves or jointly with other Technical Groups. Likewise the very successful two-yearly Society Symposium Series will continue, and I am sure that those members who attended the "Waste Management" symposium in Wellington earlier this year will agree that it "broke new ground" for the Geomechanics profession.

SLOPE STABILITY IN LAND DEVELOPMENT

Most of you will be aware of the Editor's initiatives in promoting discussion on "Slope Stability in Land Development", and I hope that many of you found time to respond to his Questionnaire. The whole question of numerical stability analysis in land developments is to me part of the wider geotechnical issues in urban planning, and I have long advocated a two-tier approach whereby geotechnical constraints are first identified by engineering geological assessment and this is followed by more detailed geotechnical investigations if or where appropriate. To require that all "permanent" slopes have a factor of safety against instability of 1.5, as the Building Industry Authority appears to imply, is to me of considerable concern to the profession. Not only is this an extremely conservative value, especially if applied to "worst case" conditions, but it fails to take into account the fact that most house sites are not even investigated in sufficient detail by geotechnical consultants during the development process to allow such calculations. I am sure that this will be just one of the matters discussed both in future issues of Geomechanics News, and at our next Symposium in early 1996 on "Geotechnical Issues in Land Development".

PROFESSIONAL RECOGNITION OF ENGINEERING GEOLOGISTS

The next matter on which I wish to comment is that of professional recognition of engineering geologists. Many of you will recall the ongoing debate within the Society on what was originally termed the "registration issue", although it has become obvious in recent years that seeking an amendment to the Engineer's Registration Act to facilitate professional recognition of engineering geologists would be an impossible task. Instead attention has focused on ways in which to allow full membership of IPENZ as a means of achieving such professional recognition, and the recent major restructuring within IPENZ has provided just such an opportunity. I was recently co-opted to an IPENZ working group looking at the creation of Technical Member, which would in turn provide a suitable career path for engineering geologists to achieve full membership of IPENZ. Subject to some "fine-tuning" in the next month or two, it is expected that final proposals will go to the IPENZ Board in February of next year and that it will become feasible for engineering geologists (amongst others) to join IPENZ via this route. It is certainly my intention to apply for membership of IPENZ once this option is confirmed, and I look forward to feedback from members on this issue.

LINKS WITH AUSTRALIAN GEOMECHANICS SOCIETY

The final matter that I wish to report on is my recent (October) attendance at the Australian Geomechanics Society (AGS) management meeting in Sydney. Many of you will be aware that AGS functions both as an IEAust National Committee and as a learned body, with funding and

services being provided by IEAust. With the changes presently taking place within IEAust it seems that AGS will evolve quite rapidly into an autonomous organisation very similar to our own Society. The meeting in Australia was very profitable, and it provided an opportunity to renew old friendships as well as to catch up with current office holders. The very successful Young Geotechnical Professionals Conference held in Sydney early this year is to continue on a two-yearly cycle. We have agreed to head up the organising committee, and we look forward to another very successful conference to be held in Auckland in November 1995. The next ANZ Geomechanics Conference is to be held in Adelaide in July 1996, and I hope that a number of our members can make it to one of the most pleasant cities in Australia. Another outcome of my visit is that we should be able to "coat-tail" onto the IEAust Eminent Speakers Programme, and I look forward to much closer working relations between the two societies. It is my hope that this closer contact can be maintained by the Regional Vice-Presidents and future Chairpersons attending respective meetings on a more frequent basis.

I wish all our members the compliments of the Season, and hope to see many of you at the AGM (during the IPENZ Conference) in Palmerston North next year.

David H. Bell
CHAIRMAN

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1. MEMBERSHIP

The following new members are welcomed to the society:

J Cocks	A Maughan	R J Spiers	G Smith
A K George	V M Meyer	A Stiles	N K Jacka

Now that a student grade of membership has been introduced a new membership application form has been prepared and is included in this Newsletter.

2. MANAGEMENT COMMITTEE

The Management Committee held its second and last meeting of the year on 20 October 1994 in Wellington at IPENZ Headquarters. Issues discussed of special interest are mentioned in subsequent paragraphs. The next meeting is scheduled for February 1995, to be held during the IPENZ Conference in Palmerston North.

3. COMPOSITION AND FUNCTION OF THE MANAGEMENT COMMITTEE

A review is to be undertaken of the composition, function and future of the Management Committee. For example, should the composition of the Management Committee be more clearly defined to ensure coverage of all members interests, should there be a limit on the period of time one can serve on the Management Committee. It is considered desirable to set limits and to encourage active participation in the running of the affairs of the Society. A discussion paper is to be presented at the next Management Committee meeting.

4. NAME OF THE SOCIETY

Should the name of the Society be changed to reflect modern terminology and to attract a broader membership base e.g. New Zealand Geotechnical Society (the use of Geotechnical is more commonly used overseas than Geomechanics).

5. NEXT SYMPOSIUM

It was agreed that the next Symposium theme should be "Geotechnical Issues in Land Development". This theme should have broad appeal and would include coverage of land stability and other issues related to the Resource Management Act and Building Act. The preferred venue is Hamilton in February 1996.

6. GEOTECHNICAL LIMIT STATE DESIGN SEMINAR

The Geotechnical Limit State Design Seminar series that myself and Professor M Pender (representing NZGS) and Peter Smith (representing SESOC) presented at Auckland, Taupo, Wellington and Christchurch was extremely well supported. In total some 250 people attended, mostly structural engineers.

There were some interesting discussions on geotechnical limit state design as well as geotechnical problems in general. It was obvious that co-operation between NZGS and SESOC could lead to mutual benefits and as a Society perhaps we should be putting more energy into working more closely with SESOC and other technical groups.

Already SESOC and NZGS are commencing work on a new manual for Design of Retaining Walls and it is intended to update the geotechnical limit state design recommendations given at the Seminar series into a publication at some future date.

7. RESTRUCTURING OF IPENZ

At a Special General Meeting on 21 September 1994 IPENZ passed rule changes that will substantially alter the structure of IPENZ and the way in which it will be managed. These proposals have been widely circulated and include reducing the size of Council, introducing new membership categories, simpler procedures for election, extension of voting rights and scope for self-governing practice colleges.

In practice it will mean that the Technical groups such as NZGS will have a direct link to the Council and play a much more significant role. They could be responsible for responding to public issues (on behalf of IPENZ) in their areas of interest, preparing verification methods or standards for compliance with the Building Act and managing self-governing practice colleges. It is early days but based on discussion with candidates for election to the new Board it seems that we are entering a period of considerable opportunity. However, to respond effectively to these opportunities will require enthusiastic input from members.

Trevor Matuschka
MANAGEMENT SECRETARY

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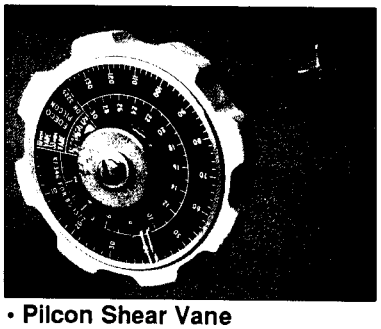

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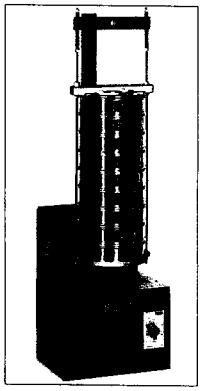

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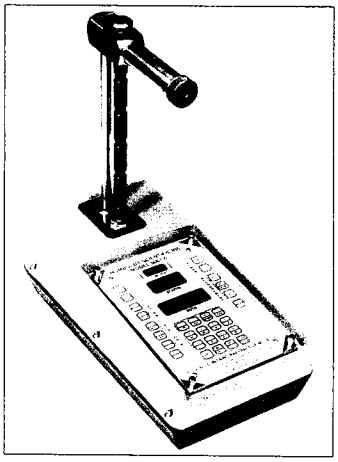

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Société Internationale de Mécanique des Roches

International Society for Rock Mechanics

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Report from ISRM Vice President: NZGS meeting of October 20 1994

(i) Rocha Medal nominations to MJP by December 31

(ii) ISRM Council meeting that was held in Santiago in May. The main things of interest there are the new arrangements for election of the President. The Council passed the recommendation the a President-elect be voted as a full member of the Board about 2 years before taking up office as the President.

(iii) The other item of interest from the Council meeting are moves to have greater cooperation with other bodies - initiatives spearheaded by Norbert Morgenstern when he was President of the ISSMFE and Charles Fairhurst. An initial project in this area is the preparation of a common membership database for the ISRM, IAEG, and ISSMFE. I have received suggestions for the format for this.

(iv) A new ISRM Directory will be published at the time of the Tokyo conference next year. A call has been made for member societies to have up-to-date information which includes fax and e-mail addresses if available.

Mick Pender
AUSTRALASIAN VICE PRESIDENT ISRM

Société Internationale de Mécanique des Roches

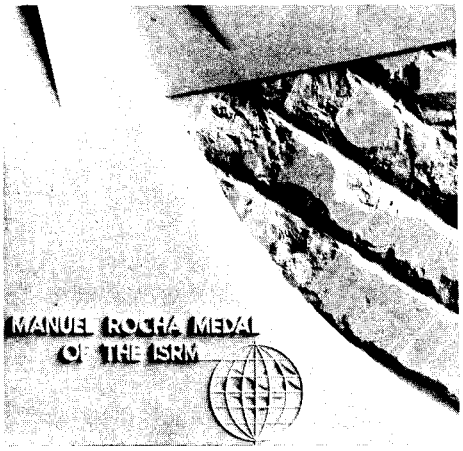
International Society for Rock Mechanics

Internationale Gesellschaft für Felsmechanik

ROCHA MEDAL 1996

SINCE 1982 A BRONZE MEDAL AND A CASH PRIZE HAS BEEN ANNUALLY AWARDED BY THE ISRM TO HONOUR THE MEMORY OF PAST PRESIDENT MANUEL ROCHA WHILE STIMULATING YOUNG RESEARCHERS IN THE FIELD OF ROCK MECHANICS.

AN INVITATION IS NOW EXTENDED TO THE ROCK MECHANICS COMMUNITY, AND SPECIALLY TO FACULTY MEMBERS, FOR NOMINATIONS FOR THE ROCHA MEDAL 1996.



Past Recipients

1982 – A. P. Cunha, Portugal; 1983 – S. Bandis, Greece; 1984 – B. Amadei, France; 1985 – P. M. Dight, Australia; 1986 – W. Purrer, Austria; 1987 – D. Elsworth, UK; 1988 – S. Gentier, France; 1989 – B. Fröhlich, Germany; 1990 – R. K. Brummer, South Africa; 1991 – T. H. Kleine, Australia; 1992 – A. Ghosh, India; 1993 – O. Reyes W., Philippines; 1994 – S. Akutagawa, Japan; 1995 – C. Derek Martin, Canada.

APPLICATION

To be considered for an award the candidate must be nominated within two years of the date of the official doctorate degree certification, and will be eligible for nomination by the individual concerned, by the individual's National Group, or by some other person or organization acquainted with the individual's work.

Nominations shall be addressed to the ISRM Vice-President of the respective geographical region* and they shall contain a one page curriculum vitae which is to include the complete identification of the nominee (full name; nationality; place and date of birth; position held; postal address; telephone and telefax numbers);

A thesis summary in one of the languages of the Society, preferably in English, of about 5 000 words, detailed enough to convey the full impact of the thesis and accompanied by selected tables and figures, with headings and captions also presented in English;

One copy of the complete thesis and one copy of the doctorate degree certificate;

A letter of copyright release, allowing the ISRM to copy the thesis for purposes of review and selection only.

Application Deadline

The nomination shall reach the ISRM Vice-President concerned by registered mail not later than 19941231.

REPORT FROM VICE CHAIRMAN IAEG

COUNCIL MEETING

The Council meeting and the 7th Congress of the I.A.E.G. were held in Lisbon, Portugal from 4th-9th September 1994 and were attended by a record number of delegates and produced a comprehensive and substantial six volume proceedings.

I represented New Zealand at this meeting which brought together the representatives of well over 40 national groups and the executive of the I.A.E.G. The meeting elected to increase subscriptions in order to consolidate the financial affairs of the association which have come through a difficult period. New groups were welcomed and new Vice-Presidential territories agreed, as a result of the political reforms in Europe. There is a need to restore the national groups of the Philippines - Indonesia area and I am looking for useful contacts in that region.

CONGRESS

Over 600 delegates and 100 accompanying persons came to the L.N.E.C. in Lisbon and enjoyed the wonderful facilities, stimulating papers and discussion, many superb social occasions and the very valuable opportunity of international professional contact. I was the only New Zealander present and there were just two Australian delegates, one of whom was John Braybrooke, the incumbent Vice President for Australasia. I am sure we can all look forward to a much larger presence from New Zealand and Australia in Vancouver 1998. The Lisbon Congress was an undoubted success and very much enjoyed by all.

NEW EXECUTIVE

Your new executive for 1995 - 1998 is:-

President Prof Paul Marinos (Greece)
Vice Presidents for:
- Africa: Dr Jacobus Venter (South Africa)
- North America: Mr John Gartner (Canada)
- South America: Jorge Sevilla (Ecuador)
- Asia: Prof Wang Sijing (China)
- Australasia: Dr Warwick Prebble (NZ)
- Europe: Dr Brian Hawkins (UK)
- Europe: Dr Edmund Krauter (Germany)

Secretary General: Dr Louis Primel (France)
Treasurer: Dr Michel Deveughele (France)

THE NEXT CONGRESS

Vancouver, B.C. Canada is the venue for the 1998 Congress. I am in frequent contact with the Canadians. Please give me your wish list for Symposia themes, and make plans now to attend.

COMMENT

The themes and discussion at Lisbon reflect the growing interest in "environmental" matters on the part of the I.A.E.G. Themes such as "natural hazards", and "environmental protection" indicated the activities by delegates in those areas, in addition to the many familiar themes such as "engineering geological mapping" and "construction materials". There was discussion on the need to include many more scientists in our organisation, such as environmental chemists, ecologists and soil scientists. Clearly a wider vision of engineering geology is accepted by many members.

Warwick Prebble
AUSTRALASIAN VICE PRESIDENT IAEG

AUCKLAND BRANCH

Recent Meetings

July — Ian Gunn (University of Auckland) discussed recent developments in Effluent Disposal. This was a joint meeting with the Water Group.

August — Prof Guy Houlsby (Oxford University) talked about Reinforced Earth.

September — Soil Improvement at Museum of NZ in Wellington using dynamic compaction by a falling weight was presented by Peter Millar of Tonkin & Taylor Ltd. Tim Sinclair of Tonkin & Taylor also reported on his sabbatical in Canada. A separate report on the Museum of NZ project is included in this issue of Geomechanics News.

November — Geotechnical aspects of Auckland's large hole in the ground (Sky Tower and Casino development) were presented by Beca Carter Hollings & Ferner Ltd. A separate report on this project is included in this issue of Geomechanics News.

Geoffrey Farquhar
AUCKLAND BRANCH CO-ORDINATOR

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WELLINGTON BRANCH

A steady stream of talks have been arranged during the year by Ian McPherson for Wellington Branch. The talks, generally held at IPENZ rooms about every six weeks, have covered an interesting variety of "Geotechnical" topics, and have attracted a moderate audience, many of whom are "regular supporters". Ian has taken the trouble of advertising the speakers well in advance, both in Geomechanics Society fliers, and in the IPENZ Wellington Branch Newsletter. As Ian has now been attracted to the warmer climes of Malaysia, I have taken over his role as Wellington activities organiser. So if any of you have, or know of a topic suitable for presentation to the local group, please let me know!

I would like to take this opportunity of thanking our speakers who have made considerable efforts on our behalf to present interesting and informative talks. The speakers during the year (1994) have been:

April: Dr John Berrill of Canterbury University presenting the 1994 Geomechanics Lecture on aspects of liquefaction.

May: Mr P. Brabhaharan of Works Consultancy Services spoke on liquefaction and slope failure hazards in the Wellington Region.

July: Mr Greg Saul of Works Consultancy Services spoke on the stabilisation of Cairnmuir landslide in the Cromwell Gorge. Extensive stabilisation measures have included surface drainage using channel works, piping, surface contouring and infilling, slope reinforcement with construction of walls and terraces, subsurface drainage using tunnels and drillholes, plus an extensive network of instruments to monitor any future slope movements. Greg has been involved in the design of these measures and illustrated his talk with a marvellous collection of slides.

Sept: Dr Graham Ramsay and Mr Tom Marshall of Beca Consultants gave a well illustrated talk on the design and construction of the Ewan Bridge Foundation piers. The single column piers extend deep into the artesian Hutt aquifer which supplies a large proportion of the Wellington and Hutt Valley water supply. Thus there were rigorous construction requirements to ensure contamination and leakage of the artesian water did not occur.

October: Dr Ian Brown of Ian Brown Associates, who has been at the forefront of the use and development of computers in geomechanics, gave a talk on the use of computers to manage geological data. Ian has been instrumental in establishing TechBase in New Zealand and keeps up-to-date with computing developments through regular visits to USA. He also has direct experience with their application in his own consultancy.

November: Messrs David Dowrick and Dick Beetham of the Institute of Geological and Nuclear Sciences gave a talk to a combined meeting of Wellington Structural Group, NZ National Society for Earthquake Engineering and the NZ Geomechanics Society, on damage to houses caused by the 1931 Hawkes Bay Earthquake. David had recently discovered the complete archived records of damage and repair costs to houses caused by the earthquake. From these, it was possible to separate out structural damage caused by earthquake induced ground failures.

Talks coming up:

Feb. 1995: Mr Peter Millar of Tonkin & Taylor will give a talk on the foundation investigations and remediation works for the National Museum site in Wellington (see Peter Millar's article later in this issue of NZ Geomechanics News). A date and venue for this talk will be advertised in due course.

In the meantime, we would like to wish you well for Christmas and look forward to seeing you in the New Year.

Dick Beetham
for: Ian McPherson
WELLINGTON BRANCH
COORDINATOR

3. OTAGO BRANCH

Groundwater and Landfills - An Otago Perspective

A successful meeting was held in November with 25 participants who were mostly non-members. Four speakers addressed problems with landfills, changes in practice and implications of not adequately dealing with landfill management as follows :

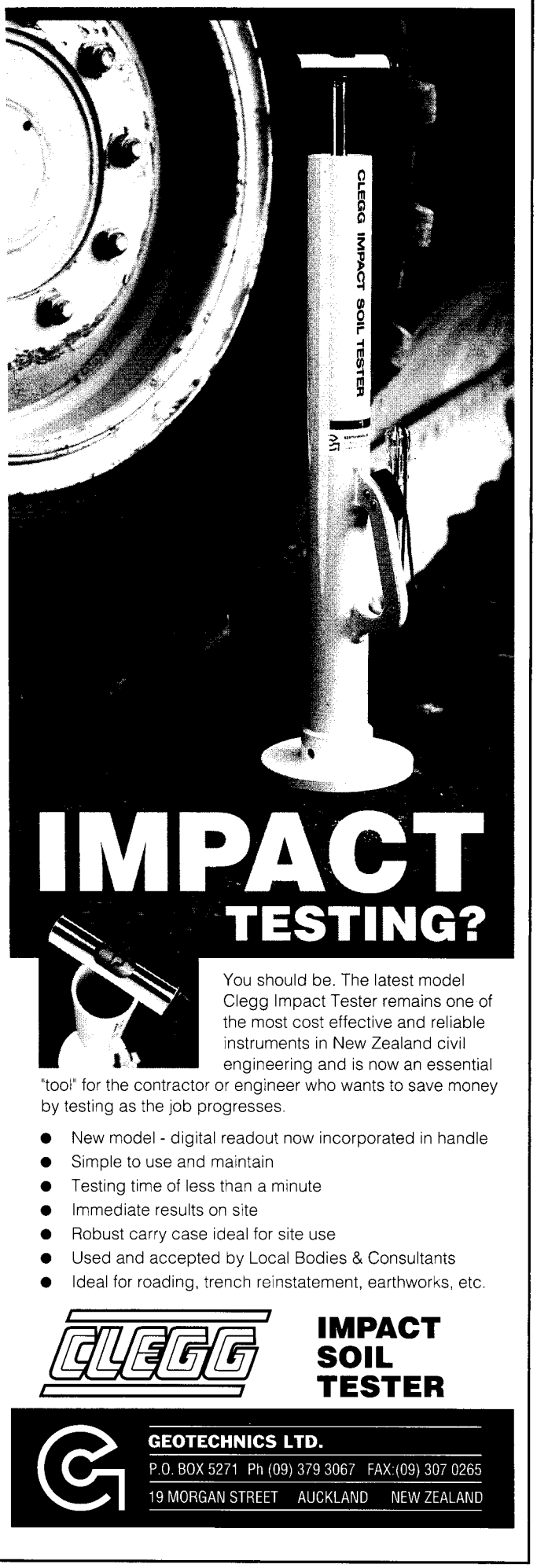
John Cocks (Royds Consulting) spoke on "Landfills, the RMA and regional and district waste management planning."

David Stewart (Otago Regional Council) spoke on "Groundwater Resources - what is known and finding out more."

Dugald MacTavish (Irricon) presented a "Case study - Taieri aquifer and North Taieri Landfill."

David Stewart (Royds Consulting) spoke on "Implications of groundwater contamination on water supplies and ecology."

Philp Glassey
OTAGO BRANCH CO-ORDINATOR



MUSEUM OF NZ

The foundations for the Museum of NZ - Te Papa Tongarewa - have been constructed on a site near the Wellington waterfront using shallow footings constructed on dynamically consolidated reclamation fill. The method has permitted a cost saving of about $3M to the project over the alternative deep pile foundation option in spite of the seismic design criteria being some 40% higher than generally applied in the Wellington region.

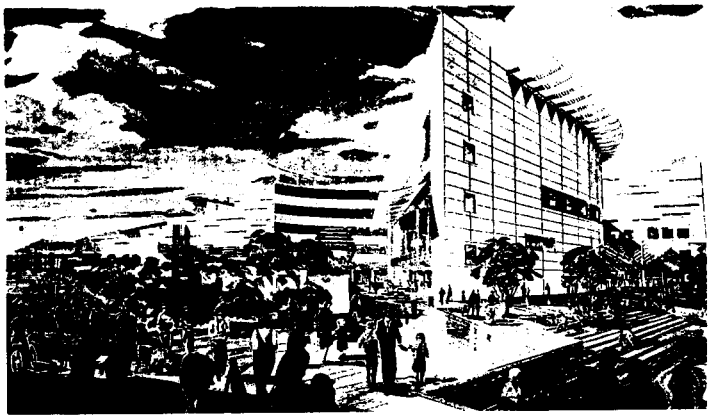
Tonkin & Taylor Ltd undertook foundation investigations in 1992 and identified up to 14 m of loose fill overlaying a thin layer of marine sediments, beach sands and alluvial gravels.

Pile foundation options were considered but the high lateral loadings of the building and fill during seismic events required very large piles or use of a sleeve system. Alternatively ground improvement of the fill would be required. Studies of improvement methods identified dynamic consolidation (DC) as the most effective system and it was possible that sufficient strengthening could be achieved to permit shallow foundation options to be considered.

A successful field trial was undertaken in 1993 which demonstrated the application of the DC method using a grid pattern of drops with weights of 25 tonnes from a drop height of up to 24 m. Strength in the fill increased from an average SPT of 10 in the upper 8 m, to over 30 with a dense surface layer of SPT >50.

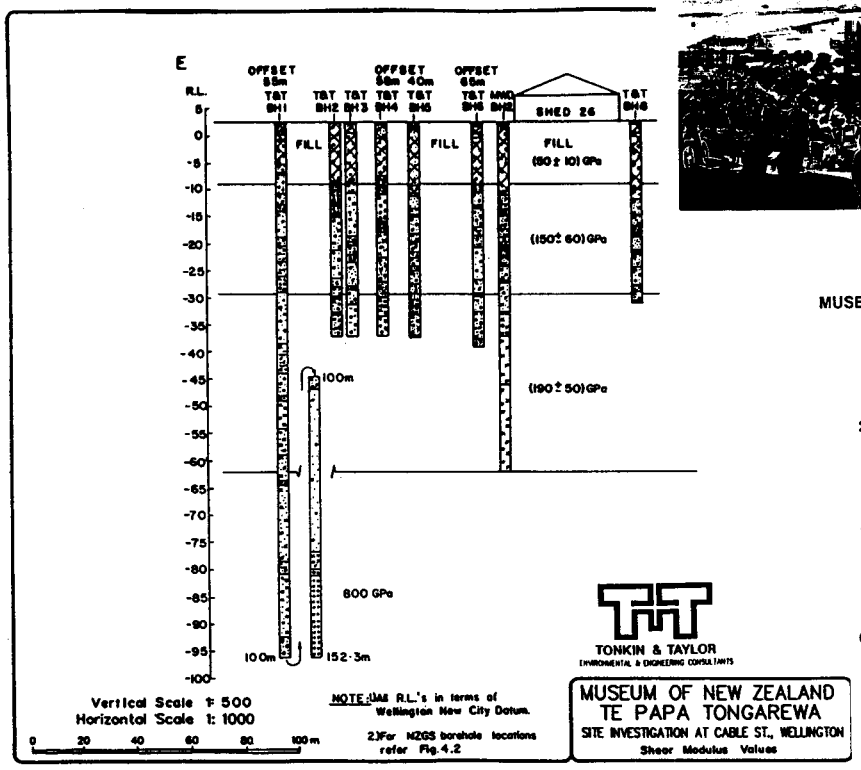
The energy levels were increased for the final design with 30 tonne weights and up to 30 m drops. The method has been very successful in achieving the objectives over most of the site. An area of silt in the fill behind the old seawall required partial excavation and formation of gravel piles. A layer of marine silt at the base of the fill also exhibited limited response to the DC. The raft action of the dense treated fill and tied footings provides adequate resistance under the demanding seismic design loading conditions.

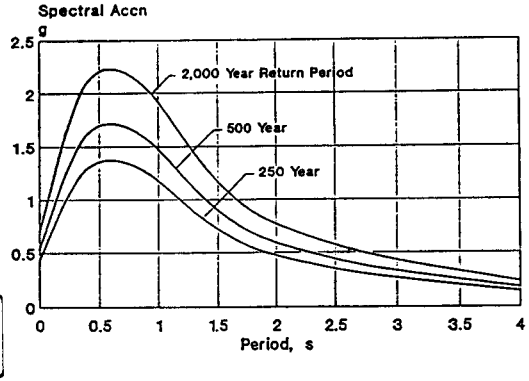
Peter Millar



MUSEUM OF NEW ZEALAND - TE PAPA TONGAREWA

WELLINGTON





GEOTECHNICAL INSTRUMENTATION

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

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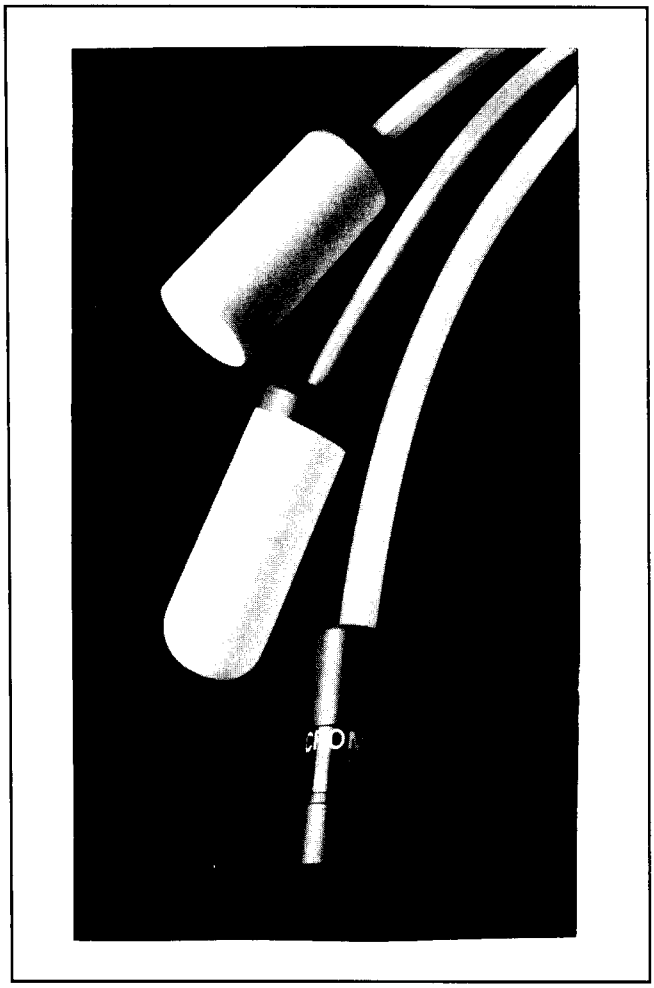
Geosystems (Aust.)
QED - Groundwater Specialists (USA)
BAT - Groundwater Systems
Enviroequip (Aust.)
Solinst (Canada)
RST Instruments (Canada)



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- Filter Sock
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- Soil Probes
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SKY CITY PROJECT

On 10 November a presentation was given by representatives of the Beca Carter Hollings and Ferner Ltd Geotechnical Section, on aspects of the Sky City excavation which was recently completed in central Auckland. The speakers, Ann Williams, Susan Tilsley, and Grant Newby were introduced by Dr D V Toan, the Director of Geotechnical Engineering.

Sky City Hobson Street involves the construction of two main structures on a central city block bound by Hobson Street, Victoria Street, Federal Street and Wellesley Street, together with access by tunnel from Nelson Street to the West. One structure is intended to house a casino together with associated facilities and the second structure is to be a viewing and communications tower known as "Sky Tower". The structures extend over the whole site of some 1.3ha. The substructure of both structures comprises an eight storey carpark, requiring some 26m of excavation.

Subsurface investigations revealed a regular sequence of materials comprising residual soils derived from the Waitemata Group rocks grading through completely, highly and moderately weathered bedded sandstone and mudstone to slightly weathered and fresh sandstone and mudstone at an average depth of 7.0m below ground surface.

Preliminary work for the excavation was carried out as part of the Resource Consent Application and AEE for groundwater. The main concern was that should the regional water table have resided in the residual soils, it would be conceivable that temporary and long-term drawdown of the water table could cause consolidation and settlement around adjacent buildings on shallow footings founded in the residual soils and around services.

Groundwater level monitoring demonstrated the existence of a cascading series of perched water tables above the regional groundwater table; that is, a vertical seepage condition. This was confirmed during excavation: other than ponding rainwater the excavation to full depth was essentially dry.

Detailed logging of the geologic features was carried out as excavation proceeded. While bedding dips predominantly gently to the northeast, the soil/rock interface deepens to up to 20m below existing ground surface in two areas. Additional anchors were installed in these areas and all anchors were grouted a minimum of 3m into fresh Waitemata Group rock.

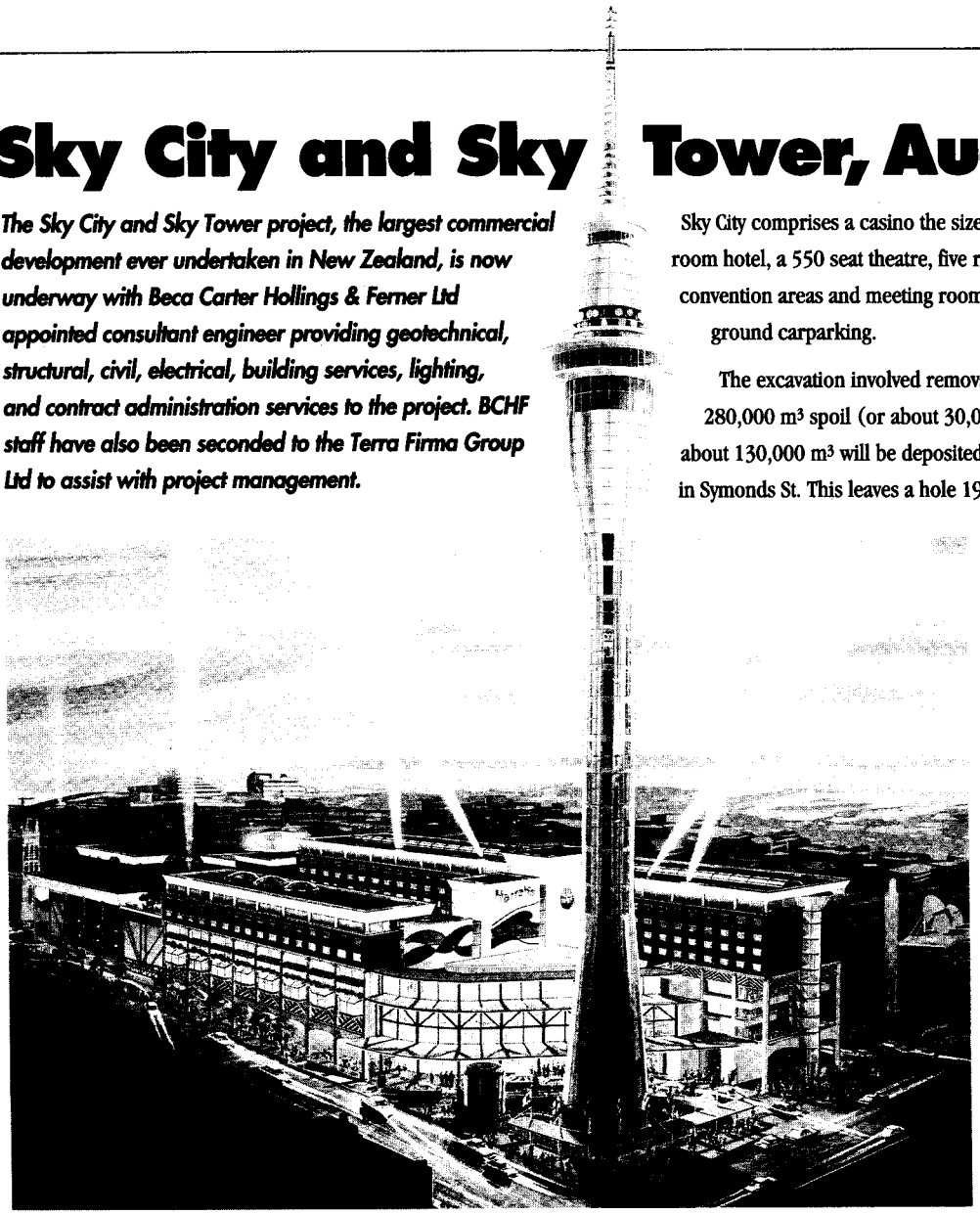
At one location, rock with SPT N-values averaging 35 blows for 300mm occurred in the floor of the excavation. Foundations in this area were redesigned for an ultimate bearing capacity of 1200kPa.

The majority of rock defects identified were discontinuous joints which would not contribute significantly to loading on the ground retention system. Two continuous joint sets were identified as having a component of dip into the excavation. Should movement have occurred along these joints, the loading imposed on the ground retention system would have been substantially higher than that assumed in design. For this reason, additional anchors were designed and installed to provide increased restraint to the rock mass in these areas.

Construction of the hotel/carpark block and of the foundation system to the Sky Tower is now well underway.

Sky City and Sky Tower, Auckland

The Sky City and Sky Tower project, the largest commercial development ever undertaken in New Zealand, is now underway with Beca Carter Hollings & Ferner Ltd appointed consultant engineer providing geotechnical, structural, civil, electrical, building services, lighting, and contract administration services to the project. BCHF staff have also been seconded to the Terra Firma Group Ltd to assist with project management.



Perspective of Sky Tower and Sky City

Sky City comprises a casino the size of a rugby field, a 344 room hotel, a 550 seat theatre, five restaurants, retail shops, convention areas and meeting rooms, and six floors of underground carparking.

The excavation involved removal of approximately 280,000 m³ spoil (or about 30,000 truckloads), of which about 130,000 m³ will be deposited on the former Sky City site in Symonds St. This leaves a hole 190 metres long, 72 metres wide and 19 metres deep on average.

The project is estimated to require some 130,000 hours of consulting time with 90,000 hours being devoted to the design stage and 40,000 hours to construction management. Currently 48 BCHF staff are working in a specially established project office, with other staff based elsewhere. Eight Wellington based staff are working on the structural design of the Sky Tower.

Preferred applicant status was conferred on Sky Tower Casino Ltd on December 17 1993, and design and project programming work on this hyper fast-track project began almost immediately to allow the Casino to open within two years of award.

Located in Auckland's Central Business District, the Sky Tower will be 328 metres tall (1080 ft), and it will not only be a distinctive landmark and tourist attraction, but will also provide superior telecommunications coverage due to the elevation and position of the tower. There will be four viewing decks and a revolving bar and brasserie in the tower.

The excavation as it nears completion.

Beca Carter Hollings & Ferner Ltd

Standards New Zealand news

Joint piling Standard

The proposal to convert revision of the 1978 Australian Piling Code, AS 2159, into a joint Standard has been dropped and the project will be completed as an Australian only national Standard. Publication is scheduled for April 1995.

The Standard covers piling - design and installation - and the revision process was already well advanced when the project was considered for joint action in 1992 under the Active Co-operation Agreement with Standards Australia. Appropriate organizations were invited to nominate New Zealand representatives for the project committee but insufficient interest was shown. Standards New Zealand has subsequently consulted with members of the New Zealand Geomechanics Society and with consulting engineers active in geomechanics to determine likely support for a joint revision.

The indications were there is no pressing need for a joint piling Standard and a great deal of work would need to be done to make the current draft suitable for New Zealand use, particularly on provisions for earthquake resistance and compliance with the New Zealand Building Code. The draft is also not suitable for use with the limit state loadings Standard, NZS 4203, and major changes would be necessary to achieve alignment.

The project originally appeared to offer an opportunity for New Zealand input to establish a piling Standard that would be suitable for New Zealand use. However with the significant differences subsequently identified, the feasibility of accommodating New Zealand interests will now need to be evaluated at the next review stage - in about 7 years' time.

Soil testing for engineering purposes

It does not seem likely there will be an immediate move to joint Standards in this area. Comments received on joint drafts circulated in New Zealand indicate there are small but significant differences in much of the equipment specified. This would require New Zealand laboratories to re-equip at some cost for very little gain, apart from having the facility to test to a joint Standard. Some equipment specifications have been derived from original British Standards, but the paths of the Australian and New Zealand soil testers have subsequently diverged.

In New Zealand many soils can be presumed never to have dried out in their natural state. Those tests which start by drying the sample may make structural changes to the soil and are therefore not representative of how such a soil would behave in practice. Thus a fundamental aspect of some of the tests in the Australian Standard is unsuitable for New Zealand conditions.

The New Zealand testing laboratory community has not called for NZS 4402 to be updated. There has also not been sufficient interest for new tests to be adopted by way of a joint Standard for tests which are not available in New Zealand Standards.

With little support evident for this joint work, NZS 4402:1986 will remain in place until at least the next review cycle. Standards New Zealand will provide early notice of any future review of Australian soil test Standards so that an informed decision on New Zealand involvement can be made and New Zealanders involved from the inception of any joint review. Further details from Ian Brewer, Technical Executive (Tel: 0-4-498-5990)

Geosynthetics

New Zealand interest has been shown in developing a joint Australian/New Zealand Standard for standardization in the field of design, testing, and usage of geosynthetics in engineering works.

Standards New Zealand sent out invitations in October for nominations to represent New Zealand on the joint committee, CE/20 Geosynthetics. Committee meetings are expected to be held in Sydney and Wellington. Further details from Ron Humphreys, Technical Executive (Telephone: 0-4-498 5990).

STANDARDS NZ NEWS

First of many

The adjacent column supplied by the staff at Standards New Zealand is to become a regular feature in NZ Geomechanics News. Comments made in this column will generally relate to geotechnical and possibly even engineering geological matters. However, given the geologists' natural aversion to things numerical, I suspect Standards NZ news will appeal more to those of us with an engineering bent.

Be that as it may (now there's a standard phrase), it is a step in the right direction - no more wading through endless unrelated standards updates. The aim is to deliver to you here, the latest improvements in standards for the geomechanically minded.

If you have something to offer, an idea, a criticism or an item of news, please forward it to:

The Editor,
NZ Geomechanics News,
P.O. Box 4241
AUCKLAND, 1
or: Fax: (09) 379 1210
Attn: Mr Geoff Farquhar

STABILITY QUESTIONNAIRE INTRODUCTION

"Slope stability assessment as part of land development consents is well established in our country. However the geotechnical community is aware of difficulties between consultants and Territorial Authorities in agreeing on a consistent approach to stability assessments. The problems are exacerbated by the specification in the Building Code Approved Document of a Factor of Safety of 1.5 for land stability, without any explanation or qualification. This is not a mandatory requirement and alternative solutions may be accepted by Territorial Authorities. Building consents are also issued under Section 36(2) of the Building Act. However, each Authority seems to have its own interpretation of both the application of the factor of safety and the application of Section 36(2) to land stability, and much time has been wasted debating the issues on an ad hoc basis."

The above paragraph summarises some of the difficulties with slope stability in land development. Geomechanics News has initiated a discussion of the subject by sending a letter and questionnaire to every Territorial Local Authority in NZ, the main geotechnical and engineering geological consultants, and other geotechnical persons. It was decided to solicit the views of Territorial Authorities, companies and practices involved in stability assessments, rather than solicit the views of individual members of the Society. Some companies and individuals may have been missed out unintentionally. If you are one and wish to make your views known then please do so by setting them out in a letter to the editor which will be published in the next issue.

The letter and questionnaire are published below together with a summary of the responses. Responses were received from 20 of the 73 Territorial Authorities, and 29 of the Consultants and others. The aim of this first step in the discussion is to publish the approaches to stability used by different groups and to encourage further contributions and debate of the issues in future issues of Geomechanics News. The Society will use the discussion to draw together a common approach to slope stability assessments for Territorial Authorities and Consultants to use. The next Symposium in February 1996 on "Geotechnical Issues in Land Development" may provide a suitable forum to finalise matters.

The responses to the questionnaire are summarised and comments published where respondents gave permission. Most respondents gave permission to publish comments and also their names. Some included letters and additional information which are published.

Responses are published as received and may not necessarily represent the views of all of a particular organisation or company. It is expected that views may be modified upon publication of a wide variety of opinions. The questionnaire was not intended to be comprehensive but rather to open up the subject of stability assessments. Therefore some responses may have been constrained by the particular questions posed.

COVERING LETTER AND QUESTIONNAIRE

Dear Sir

SLOPE STABILITY IN LAND DEVELOPMENT

Slope stability assessment as part of land development consents is well established in our country. However the geotechnical community is aware of difficulties between consultants and Territorial Authorities in agreeing on a consistent approach to stability assessments. The problems are exacerbated by the specification in the Building Code Approved Document of a Factor of Safety of 1.5 for land stability, without any explanation or qualification. This is not a mandatory requirement and alternative solutions may be accepted by Territorial Authorities. Building consents are also issued under Section 36(2) of the Building Act. However, each Authority seems to have its own interpretation of both the application of the factor of safety and the application of Section 36(2) to land stability, and much time has been wasted debating the issues on an ad hoc basis.

The Geomechanics Society, which embodies the geotechnical community in NZ, is initiating a discussion of the subject through its newsletter (Geomechanics News). You are invited to fill out the attached questionnaire and return it to the editor of Geomechanics News. Territorial Authorities are invited also to send a copy of their policy for slope stability assessments or a letter outlining their policy. Consultants and others are invited also to send a letter outlining their approach to slope stability assessments particularly as they apply to land development.

It is intended to summarise the results of the questionnaire and publish any comments made (with your permission). A copy of the newsletter will be sent to you if you do not already receive one.

This letter is being sent to every Territorial Authority in NZ, the main geotechnical and engineering geological consultants, and other geotechnical persons. The society will use the results of this questionnaire to draw together a common approach to slope stability assessments for Territorial Authorities and consultants to use. **Please return your questionnaire to the editor by 11 November.**

Please phone me on 0-9-379 1200 if you need clarification of any aspects.

Yours faithfully,

GEOFFREY FARQUHAR
EDITOR GEOMECHANICS NEWS

SLOPE STABILITY ASSESSMENTS FOR LAND DEVELOPMENT QUESTIONNAIRE

1. From whom are assessments acceptable?

   □ Registered Engineer (civil engineer) □ Registered Engineer (geotechnical specialist)
   □ Engineering Geologist □ Firm of Engineering Geologists
   □ Firm of Consulting Engineers (no geotechnical specialist in firm)
   □ Firm of Consulting Engineers (geotechnical specialist in firm)

   Do you have any comments? .....

2. Is an engineering geological assessment of stability (i.e. a non analytical approach based on study of the previous performance of the slope) acceptable?

   □ Yes □ No □ Sometimes (please give circumstances) .....

3. Is a numerical slope stability analysis that produces a factor of safety always necessary?

   □ Yes □ No (please give circumstances) .....

4. A minimum factor of safety of 1.5 is acceptable for the following conditions :

   Groundwater profile : □ High □ Average □ Low
   Soil strength : □ Upper bound □ Average □ Lower bound

   Please give any comments .....

5. Is strength testing of soils required to establish strength parameters for each specific site?

   ☐ No ☐ Yes ☐ Sometimes (please specify) .....

   Can parameters established by testing of similar soils from other sites be used?

   ☐ No ☐ Yes (please specify) .....

6. What factor of safety is required for earthquake loading?
   .....

7. What earthquake return period is required?
   .....

8. Is a factor of safety less than 1.5 acceptable for extreme conditions (excluding earthquake)?

   ☐ No ☐ Yes (please specify the extreme conditions and factors of safety) .....

9. If drainage works are used to stabilise land, should their failure to operate be considered?

   ☐ No ☐ Yes (please give circumstances) .....

10. What role does peer review of assessments play? .....

11. How is Section 36(2) of the Building Act applied? .....

12. Please make any other comments .....

13. Name of Territorial Authority, company or person whose views are represented in this questionnaire .....

14. Can the comments you have made to the various questions be published in Geomechanics News?

    ☐ Yes ☐ No

    If yes, can your name be given alongside them?

    ☐ Yes ☐ No

    If no, can your comments be identified as "Territorial Authority" or "Other"?

    ☐ Yes ☐ No

15. If you are attaching a letter setting out either a TA policy or your approach to stability assessments, can this be published as is, i.e. photocopied?

    ☐ Yes ☐ No ☐ Not applicable

    If no, can the text of the letter be published and identified as from a "Territorial Authority" or "Other"?

    ☐ Yes ☐ No

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QUESTION (1): From whom are assessments acceptable?

Summary of Results	Territorial Authority	Other		
		Geotechnical Engineers	Engineering Geologists	Firm of Engineers & Geologists
Registered Engineer (civil)	12	3	3	0
Registered Engineer (geotechnical specialist)	18	18	6	5
Engineering Geologist	12	12	6	5
Firm of Engineering Geologists	10	11	6	4
Firm of Consulting Engineers:				
Without geotechnical specialist	9	4	1	0
With geotechnical specialist	14	14	6	4
TOTAL NO. of Questionnaire Replies	**21**	**18**	**6**	**5**

Comments:

	Would accept assessments from ...	Comments	Identification
Territorial Authorities	geotechnical specialists, engineering geologists or firms with these staff	**On difficult sites, Council usually seeks its own assessment from a third party (Geotechnical Specialist)**	Territorial Authority
	all options presented in questionnaire	**Nelson City Council has been a leader in NZ for requiring stability certificates with new subdivisions. We require a certificate from an experienced geotechnical engineer, however, on larger lots or more stable land, certificates can with prior approval be provided by registered civil engineers.**	Nelson City Council
	all options presented in questionnaire	**Acceptance of any of the above would depend on the degree of difficulty, risk and assessed need for competency**	Territorial Authority
	registered (civil) engineer and Firm of Consulting Engineers with no geotechnical specialist in firm	**There are no local "geotechnical" engineers and generally a letter from a Registered Engineer is accepted by WDC regulatory people.**	Wanganui District Council

	Would accept assessments from ...	Comments	Identification
	registered engineer (geotechnical specialist)	**see attached letter**	Waitakere City Council, senior subdivision engineer (personnel view)
	all options presented in questionnaire	**Council's Code of Practice requires a Registered Engineer experienced in soils engineering or such other person as the Council may specifically approve upon the recommendation of Council's Engineer.**	Territorial Authority
	all options presented in questionnaire	**It would depend on the nature, magnitude, and risk associated with the work. If it were a large job with considerable financial risk, a firm of consulting engineers with geotechnical specialists in the firm would be required.**	Bruce Dobson, Chief Engineer, Ruapehu District Council
	geotechnical specialist only	**We prefer reports from forms specialised in geotechnical engineering**	Territorial Authority
Geotechnical Engineers	geotechnical specialists (a firm or alone) or engineering geologist)	**The question is case specific, and probably more important to be person specific. Individuals need to work within their sphere of expertise.**	C.J. Newton
	geotechnical assessments from geotechnical specialists or engineering geologists whether in a firm or sole practitioners)	**Slope stability assessments are a specialty science and should only be carried out by specialists in this field**	Mark Mitchell
	Registered Engineer (geotechnical specialist) whether in a firm or sole practitioner	**Engineering geologist (or firm) if it is the first level of assessment only (geological/ geomorphological assessment) that is required (see my response to question 2). (However, even then there may be engineering issues that need to be evaluated with the judgement as to satisfactory performance, in which case we need an engineer in the picture somewhere).**	Prof. M.J. Pender
	geotechnical specialist engineers whether in a firm or sole practitioners	**Often subjective judgements required and reliability of assessment is dependent on the experience of the person undertaking the assessment, and in case of a larger firm, the senior reviewer of the assessment**	A.W. Smith, Jackson Clapperton & Partners

	Would accept assessments from ...	Comments	Identification
	registered engineer (geotechnical specialist) or engineering geologist	**Only accepted by NCC (Nelson City Council) subject to prior approval of credentials**	Paul Russell, Royds Consulting Nelson
	geotechnical specialists and possibly engineering geologists, whether in a firm or sole practitioners	**Whether the engineer is registered or not is irrelevant; the important thing is being properly qualified and experienced in the geotechnical field. Some engineering geologists would be acceptable, but others would not**	Laurie Wesley
	geotechnical specialists whether sole practitioners or in a firm	**The assessor must be an individual with appropriate specialist experience**	D.K. Taylor
	all options presented in questionnaire	**As with all engineering judgement the degree of specialisation needs to reflect the degree of risk/difficulty and engineering registration puts limits on how registered engineers approach their work.**	Peter Geddes, Hawthorn Geddes - Civil and Structural
	geotechnical specialists or engineering geologists, whether in a firm or sole practitioners	**We are concerned at the number of slope stability assessments carried out by unqualified people**	Soil & Rock Consultants
	geotechnical specialists or engineering geologists whether in a firm or sole practitioners	**Provided assessment is carried out/approved/ reviewed by the geotechnical specialist**	Brian Patterson
	geotechnical specialists or engineering geologists whether in a firm or sole practitioners	**Slope stability is a specialist science and as such should be dealt with by a specialist**	Babbage Consultants Ltd - N.S. Luxford
	geotechnical specialists or engineering geologists whether in a firm or sole practitioners	**Engineering geologist would be acceptable if registered under a recognised system similar to engineer's registration or member of recognised institution. Should utilise specialist unless straightforward site**	Fraser Thomas Ltd

	Would accept assessments from ...	Comments	Identification
Engineering Geologists	all options presented in questionnaire except a firm of consulting engineers without geotechnical specialist	**Assessments should be from people with appropriate expertise**	R.D. Beetham
	all options presented in the questionnaire	**I have ticked all boxes simply because it is unprofessional for anyone to operate outside their field of expertise. Under some circumstances, any one of the above could carry out a suitable assessment, however that is defined**	Dr D.H. Bell
	all options presented in the questionnaire except a firm of consulting engineers without a geotechnical specialist	**Not necessarily any of the above unless done by suitably qualified and experienced professionals**	Independent Self Employed Consultant
	geotechnical specialists or engineering geologists whether in a firm or sole practitioners	**Firm of engineers with an engineering geologist or geotechnical engineer should be acceptable so long as report is reviewed/ written by the geotechnical engineer or engineering geologist with required expertise**	Other
Firm employing both geotechnical engineers and engineering geologists	geotechnical specialists or engineering geologists whether from a firm or sole practitioner	**Assessments are of a specialist nature and it is not appropriate for a generalist practitioner (unless there is specific local experience)**	D.N. Jennings, Works Consultancy Services (Hamilton)
	geotechnical specialists or engineering geologists whether in a firm or sole practitioners	**Slope stability assessments require specialist skills and therefore should only be undertaken by appropriately qualified and experienced people**	Worley Consultants Ltd
	geotechnical specialists or engineering geologists whether in a firm or sole practitioners	**A multi-disciplinary approach is preferable.**	Tonkin & Taylor Ltd (Auckland, Wellington, Christchurch & Dunedin)

QUESTION (2) **Is an engineering geological assessment of stability (i.e. a non-analytical approach based on study of the previous performance of the slope) acceptable? (If "No", give circumstances).**

Summary of Results	Territorial Authority	OTHER		
		Geotechnical Engineers	Engineering Geologists	Firm of Engineers & Geologists
YES	4	6	5	1
NO	4	0	0	1
Sometimes	12	13	4	3
TOTAL NO. of Replies	**20**	**18**	**6**	**4**

Comments:

	Answer (Yes/No/ Sometimes)	Comments	Identification
Territorial Authorities	Sometimes	**In areas of the city known not to experience difficulties over the last 20 or so years.**	Territorial Authority
	Sometimes	**The normal situation would be a non-analytical approach as only two or three firms provide the certificates and they have done the ground work for the Nelson soils and know how they perform in the conditions.**	Nelson City Council
	Sometimes	**Particularly where there is a low degree of risk**	Territorial Authority
	No	**(see attachment - letter)**	Personal View - Waitakere City Council Senior Subdivision Engineer
	Sometimes	**In addition to the factors outlined in question 1, the degree of certainty that we have with regard to the problems in a particular locality would influence the judgement.**	Bruce Dobson, Chief Engineer, Ruapehu District Council

	Answer (Yes/No/ Sometimes)	Comments	Identification
	Sometimes	**This entirely depends on the type of the slope i.e. slope angle, slope height, soil strength information. Example:** **1. A slope of 14° or less in weathered Waitematas or volcanic residuals may only need an engineering geological assessment i.e. boreholes to confirm soil strength parameters.** **2. A flat site in developed region may not even need a geotechnical assessment provided that there are no other evidence of unsuitable soil such as uncontrolled fill or peat.**	Territorial Authority
Geotechnical Engineers	Sometimes	**In many situations, clear cut decisions can be made on unstable or stable land, however, it is the boreline case that need specialist involvement**	C.J. Newton
	Sometimes	**Particularly to provide general guidelines for land zoning**	Mark Mitchell
	Sometimes	**1. Uniform geology** **2. history of performance on similar nearby areas** **3. availability of nearby areas for inspection**	Bruce Horide
	Sometimes	**Where formation is variable and characteristic cannot be easily modelled (i.e. not deep uniform formation) with easily measurable characteristics)**	Other
	Sometimes	**I like to think of a hierarchy of three levels: geomorphological (geological?) assessment, stability assessment, and stability analysis. This has a progression of increasing depth (an expense) of investigation. For straightforward applications (in an area where there is no history of ground instability) a geomorphological assessment will be adequate.**	Prof. M.J. Pender
	Sometimes	**Dependent on ground slope, nature of assessment and proposed development non-analytical approach may be suitable for a preliminary appraisal, but any major development should be assessed by testing and analysis**	A.W. Smith, Jackson Clapperton & Partners
	Sometimes	**Usually providing the proposed development is not worsening the situation or field evidence from adjacent sites can be applied**	Paul Russell, Royds Consulting Nelson
	Yes	**But not based entirely on the previous performance of the particular slope. Performance of other slopes in the same material would come into the assessment**	Laurie Wesley

	Answer (Yes/No/ Sometimes)	Comments	Identification
	Sometimes	**As a means of zoning land with various degrees of risk and to indicate the extent of subsequent investigations, assessment or analysis appropriate to specific sites/areas.**	D.K. Taylor
	Yes	**Where instability is manifested, then a more rigorous approach would be needed.**	Peter Geddes, Hawthorn Geddes Civil & Structural
	Sometimes	**A qualitative assessment of slope stability is an essential part of the overall assessment that should be carried out in all cases. We consider the qualitative assessment to be of major, if not greater, importance than the quantitative (or analytical) assessment.**	Soil & Rock Consultants
	Sometimes	**Normally a study of previous performance of the slope is only part of the engineering geological assessment. If the slopes are not going to be modified or natural drainage interfered with, a simple study of slope movement history is adequate provided the nature and general performance of materials are acceptable.**	Brian Patterson
	Sometimes	**Only rarely for reasons given in (comments to) Question 3**	R. Melville-Smith, Foundation Engineering Ltd
	Yes	**Where stresses on slope are undergoing negligible future changes**	N.S. Luxford, Babbage Consultants Ltd
	Sometimes	**(a) When the slope is obviously unstable; or** **(b) When slope is shallow and obviously stable, based on study of past performance;** **(c) When done in conjunction with a numerical analysis**	Fraser Thomas Ltd

	Answer (Yes/No/ Sometimes)	Comments	Identification
Engineering Geologists	Yes/Sometimes	**Sometimes it is the most realistic method for assessing stability - particularly if there is little subsurface investigations data.**	R.D. Beetham
	Yes/Most Times	**Design by precedent is entirely acceptable in all cases where it can be demonstrated that there is no existing slope instability, and where all codes etc., are complied with in relation to long-term site stability. Where active ground movement is occurring some form of analytical approach is probably required, but again I would caution against total reliance on the numbers obtained. Without an accurate site model and correct input parameters a quantitative analysis can be misleading**	Dr D.H. Bell
	Sometimes	**Analogy can be drawn (similar geological foundations and loading conditions). NB: engineering geological assessments do not have to be non-analytical at definition**	Independent Self Employed Consultant
	Yes/Sometimes	**Should be used in conjunction with analytical methods to provide a better assessment of a site and enable incorporation of observations/ measurements in any back analysis of a slope**	Other
Firms employing both geotechnical engineers and engineering geologists	No	**Little confidence can be given to an engineering geology approach if the mechanisms of failure are not understood and similar little confidence can be given to an analytical analyses without a realistic geological model.** **The quantum of each is site selective, i.e.:** **(i) If the slope is in weathered soil/rock with a stability condition that is structurally controlled there is little point in obtaining a safety factor from a series of soil test strength parameters and software that assumes an uncontrolled failure surface. The analytical analyses must be limited to the topography and the soil/rock structure conditions and a "worst case" model of these conditions "back-analysed" to obtain an appreciation of the likely range of strength properties. The analysis may be a relatively simple manual type. These values can then be used for the development evaluation where the topography is to be altered.**	Glyn R.W. East, Works Consultancy Services Ltd, (Auckland)

	Answer (Yes/No/ Sometimes)	Comments	Identification
		(ii) Likewise if the slope is in relatively homogeneous and monotonous clay type soils or a fill/embankment over weak soils there is little the geology can aid to the evaluation apart from identifying a non-structurally controlled soil. In this case, an analytical analysis must be carried out on strength tested soils. **My experience is such that the former is the more typical in the land development situation.**	
	Sometimes	• **where there is clear evidence of geological conditions and long term stability; and** • **where there are no proposed changes in the environmental conditions (e.g. run-off, soakage, loading of slopes)**	D.N. Jennings, Works Consultancy Services Ltd (Hamilton)
	Sometimes	**Where careful assessment of slope angle, outcrops and borehole information by an experienced person suggests stability is unlikely to be critical**	Engineering Geology Ltd
	Yes	• **generally, the "past performance" approach should be the prime assessment procedure. Obviously this requires an appropriately skilled assessor.** • **numerical analysis may be warranted if appropriate data is available and/or engineering works are contemplated to decrease unacceptable risks. The analyses can help assessment of the degree of improvement**	Worley Consultants Ltd
	Sometimes	**Only in situations where there is <u>no</u> precedent of instability and risk to structures and life is low. In all other situations, subsurface investigation is required to establish the soil strength profile and groundwater regime, so that it can be compared with situations where instability has occurred.** **The Engineering Geologist approach is likely to be at least as useful/necessary as pure analysis, and an analytical approach alone is not acceptable. The engineering geologist approach should be the first stage of any assessment, i.e. should precede a numerical analysis - however, only as a stand-alone report where no instability is identified.**	Tonkin & Taylor Ltd (Auckland, Wellington, Christchurch & Dunedin)

QUESTION (3): Is a numerical slope stability analysis that produces a factor of safety always necessary?

Summary of Results	Territorial Authority	OTHER		
		Geotechnical Engineers	Engineering Geologists	Firm of Engineers & Geologists
YES	5	3	1	1
NO	14	15	6	4
TOTAL NO. of Replies	**19**	**18**	**7**	**5**

Comments:

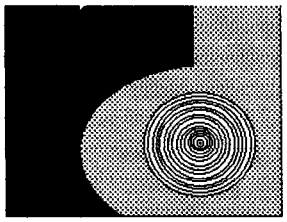
	Answer (Yes/No)	Comments	Identification
Territorial Authorities	No	**As per comments to question 2**	Territorial Authority
	No	**Usually only require such an analysis when structures are involved or it involves more steep or difficult hillsides**	Nelson City Council
	No	**Depends on the history, topography and geology of the site**	Territorial Authority
	No	**Where it can be demonstrated that a particular treatment, including a stable slope, has proved successful in identical circumstances**	Territorial Authority
	No	**District Council takes the view that the specialist would carry out the necessary level of investigation appropriate to the site**	Territorial Authority
	No	**Generally not required and never given if ever requested**	Wanganui District Council
	No	**Surface slope - slight** **Depth of weathered material - not deep** **G.W.L. - deep and not able to rise to surface** **Borehole Investigation - shows high shear strengths** **- strength rising with depth**	Waitakere City Council, Senior Subdivision Engineer - Personal view

	Answer (Yes/No)	Comments	Identification
	No	The factors outlined in (responses to questions) 1 and 2 would be taken into account.	Bruce Dobson, Chief Engineer, Ruapehu District Council
	No	For slopes such as that mentioned in (response to) question 2.	Territorial Authority
Geotechnical Engineers	Yes	The numerical answer is the combination of all the inputs and it will show up any oversights that may have occurred	C.J. Newton
	No	When soil strengths are relatively high, experience will indicate that adequate factors of safety occur	Mark Mitchell
	No	Underlying rock with erodible surface layers causing slab failures cannot be easily assumed	Other
	No	"Calculated" factors of safety are quite misleading without appropriate investigation. The numbers are worthless (they can even create a misleading air of "scientific" rigour) if a proper and thorough investigation has not been done. Generally the appropriate investigation is too expensive. Thus a stability assessment is about as far as one could expect to go.	Prof. M.J. Pender
	No	It is sufficient to comply with the performance provisions of clause B1-Structure, i.e. clauses B1.3.1, 3.2 and 3.3	Paul Russell, Royds Consulting Nelson
	No	I believe the production of a safety factor by numerical analysis would contribute about 5% to the data pool on which a slope stability assessment would, or should, be based	Laurie Wesley
	No	Not necessary if an engineering geologist's, or experienced geotechnical engineer's, assessment of the area is that the risk of instability is low	D.K. Taylor
	No	(Not necessary when) knowledge of local conditions previously experienced	Other
	No	See (comments to question 2)	Peter Geddes, Hawthorn Geddes Civil & Structural

	Answer (Yes/No)	Comments	Identification
	No	Slope stability analysis only provides useful and reliable results if the geology and soil profiles, groundwater levels and pressures, and soil strength parameters are all reasonably well known. In many circumstances, the investigation required to determine this information is uneconomic, and unnecessary, unless stability is seen as critical from other aspects of the site assessment.	Other
	No	The qualitative assessment considered in conjunction with the development considerations may often be sufficient, e.g.: • site development (retaining walls/basements/etc) provide the necessary stabilisation • thin veneer of weathered residual soil on hard less weathered base • well drained side slope of spur or ridge, etc.	Soil & Rock Consultants
	No	Generally an investigation leading to determination of a factor of safety is not required unless there is a serious concern about the stability, and the nature and size of the development warrants it	Brian Patterson
	No	Nearly always, but sometimes the slope geometry and/or the shape of the failure plane makes computer analyses unrealistic. I like to run analyses to test sensitivities to groundwater levels and to changes in soil properties	R. Melville-Smith, Foundation Engineering Ltd
	No	Many situations dealing with natural soils where ground strengths vary considerably are best assessed using regional experience, careful/detailed site inspection and review of existing data such as air photos and geological maps	N.S. Luxford, Babbage Consultants Ltd
	No	(a) When the slope is obviously unstable (b) When slope is shallow and obviously stable, based on study of past performance	Fraser Thomas Ltd
Engineering Geologists	No	Some slopes are too complex for numerical analysis to be meaningful	Riddolls & Grocott Ltd
	No	Some analyses are quite inappropriate because of lack of data, etc.	R.D. Beetham

	Answer (Yes/No)	Comments	Identification
	No	As noted above there is no requirement for quantitative analysis in situations where existing stability is maintained, and the associated costs of obtaining "correct" site data cannot be justified. Even where existing slope stability is identified it will not always be necessary to analyse slope stability quantitatively as s36(2) allows for approval provided that stability is not worsened by the development. In my view, qualitative (i.e. not quantitative) assessment is justified in the great majority of cases.	Dr D.H. Bell
	No	As (per comment to Question 2). Also probabilistic based analysis may be preferable	Independent self employed consultant
	Yes	Useful in providing a benchmark, however, every specialist will do it slightly differently. Should still only be used as an indication rather than as it is currently assumed to be an exact answer	Other
Firm employing both geotechnical engineers and engineering geologists	Yes	Some type of numerical analyses is always necessary for an appreciation of the stability conditions and any methods to improve the situation. If the topography is to be altered, there is no way a terrain evaluation without some analytical input can satisfy any stability criteria (on the assumption that the slope in question is to be steepened or loaded). As mentioned (in my response to question 2), this analysis may be confined to "back-analysis" or may be a relatively simple manual type	Glyn R.W. East, Works Consultancy Services (Auckland)
	No	Where the slope is obviously stable (based on known geology, nature of development, and previous experience/analysis in similar situations) - c.f. (answers to question 2).	D.N. Jennings, Works Consultancy Services Ltd (Hamilton)
	No	Only required in cases where an experienced judgement suggests that stability could be critical or to check remedial works necessary to improve an existing instability situation.	Engineering Geology Ltd
	No	See comments for (question 2). In addition, the analytical approach, particularly sensitivity analyses, can be useful to assess relative risk	Worley Consultants Ltd

	Answer (Yes/No)	Comments	Identification
	No	**(Refer comments for Q.2). Where there is a significant risk to life or structures and/or there is a precedent of instability (i.e. landslips in similar materials or similar slopes) and/or where the slope is to be modified, i.e. cuts, fills, retaining structures, surcharge, raising of groundwater levels. Very often it is not possible to determine parameters and geological model sufficiently accurately to warrant numerical analyses in which case analysis can show improvement by proposed works.**	Tonkin & Taylor Ltd (Auckland, Wellington, Christchurch and Dunedin)



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QUESTION (4): A minimum factor of safety of 1.5 is acceptable for the following conditions:

Summary of Results	Territorial Authority	OTHER		
		Geotechnical Engineers	Engineering Geologists	Firm of Engineers & Geologists
Groundwater Profile:				
High	2	4	1	3
Average	8	6½	0	2
Low	0	2½	1	0
Soil Strength:				
Upper Bound	1	½	½	0
Average	10	9½	½	2
Lower Bound	0	2	0	3
TOTAL NO. of Replies	**18*1**	**18*2**	**6*3**	**5*4**

* Notes:

1. 8No. Territorial Authority respondents commented that for these conditions they rely on independent specialist advice

2. 2No. respondent stated these conditions are site specific
   2No. respondents replied "don't know" or "didn't understand the question"
   3No. respondents stated a sensitivity analysis is required

3. 3No. respondents stated these conditions are site specific
   1No. respondent stated a sensitivity analysis is required
   1No. respondent stated disagreement with options presented

4. 2No. respondents stated these conditions are site specific

Comments:

	Answer		Comments	Identification
	GWT	Soil Strength		
Territorial Authorities	-	-	**This is a site specific issue, general answer not appropriate**	Territorial Authority
	Average	Average	**We have no guidelines on this issue at present and the answer given is intuitive**	Territorial Authority
	-	-	**Act on recommendations of consulting engineer**	Territorial Authority
	-	-	**Refer (comments to question) 3 and (question) 12**	Territorial Authority

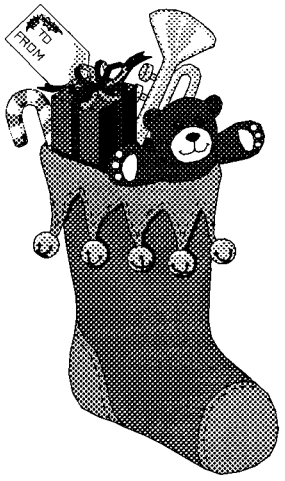
	Answer		Comments	Identification
	GWT	Soil Strength		
	-	-	**Refer (comments to question) 3**	Wanganui District Council
	-	-	**Shear vane tests and boreholes done at time of subdivisions. Registered covenants placed against title if necessary.**	Territorial Authority
	Average	Average	**Probably of more importance would be the extent to which we could be satisfied with the consistency of the groundwater profile and soil strengths. If there was a high degree of (precision) and low financial risk a factor of safety of 1.5 would probably be acceptable**	Bruce Dobson, Chief Engineer, Ruapehu District Council
	-	-	**See attachment (personnel view in letter from WCC Senior Subdivision Engineer)**	Waitakere City Council Senior Subdivision Engineer - personal view
	High	Average	**The slope stability analysis is carried out for realistic maximum ground water level and agreeable realistic values of c′ and ϕ'. The realistic values of c′ and ϕ' should be either established from laboratory tests or estimated from in-situ tests such as shear vane reading in a test bore.** **The groundwater level should be considered saturated unless hydrological evidence is produced to justify a lower level.**	Territorial Authority
Geotechnical Engineers	Low	Average	**Need to ensure groundwater remains low and the failure mass is relatively small. Large ancient landslides are unlikely to obtain factors of safety of 1.5**	C.J. Newton
	Average	Average	**If you are working with lower bound soil strengths, you may be able to accept a FS of 1.3, particularly if the soil strata is well established with test borings, etc.**	Mark Mitchell
	-	-	**Depends on the application and the site conditions. The consequences of failure, the uncertainty in the groundwater conditions, how much local knowledge is available about the soil, the variability of the soil deposit (e.g. volcanic worse than sedimentary).**	Prof. M.J. Pender

	Answer		Comments	Identification
	GWT	Soil Strength		
	Average	Average	Geotechnical assessment and design should be moving towards (say) "10 percentile" strengths and load factors as used in structural design and NZS 4203!	Bruce Horide
	-	-	Load factors define ultimate state ϕ factors materials (NZS 4203 1992)	Other
	Average	Average	Use limit state. For groundwater factor depends on "permanency" of profile. Temporary conditions could use FS 1.3	Paul Russell, Royds Consulting Nelson
	-	-	Whether it is acceptable depends on how detailed the site investigation has been, and how carefully the samples for $c'\phi'$ measurements have been selected etc.	Laurie Wesley
	High	Average	Reliance on analysis may lead to equally misleading results as reliance on engineering assessment. Both approaches need to be made as appropriate	Peter Geddes, Hawthorn Geddes (Civil & Structural)
	Average	Average	Factors of safety can only be realistically used if a sensitivity analysis is made for variations in parameters so that a judgement can be made as to whether average values are unduly conservative, or optimistic	Other Engineers
	High	Average	High GWT - reasonably regular seasonal event. Strength parameters - not easily defined re Upper, average, lower for different soil deposits. Computer analyses allow sensitivity of slope stability to variations in strength (particularly c') to be investigated	Soil & Rock Consultants
	Average	Average	The requirement to determine a minimum factor of safety for every site assessment is unnecessary and unrealistic. I suggest that there will be only a few sites that require sufficient investigations and lab tests etc., to obtain a realistic and meaningful factor of safety determination	Brian Patterson
	Average	Average	See (comments to question) 12	N.S. Luxford, Babbage Consultants Ltd

	Answer		Comments	Identification
	GWT	Soil Strength		
Engineering Geologists	Low	Upper bound/Av	**Very conservative requirement for many hill areas**	Mark Yetton, Geotech. Consulting
	-	-	**This question pre-supposes that a minimum factor of safety is a pre-requisite. The use of a minimum safety factor (regardless of the conditions) leads to the possibility for manipulation of the input data to comply with standards**	Riddolls & Grocott Ltd
	-	-	**A factor of safety of 1.5 would be very conservative in some conditions. A factor of safety has to be tailored for the particular problem and knowledge of conditions**	R.D. Beetham
	-	-	**I don't agree with any (of the groundwater or soil strength options presented). I consider that a factor of safety of 1.5 is too conservative under almost all conditions, and it therefore becomes highly debatable as to whether or not the analysis should be carried out under worst case conditions. My approach would be to consider a factor greater than 1.0 for the worst case conditions likely within the design life of the structure, but only of course in situations where a quantitative analysis is deemed necessary.**	Dr D.H. Bell
	-	-	**Extremely sensitive sites or structures, or where failure modes/mechanisms/input parameters are uncertain and have not been properly accommodated in the analysis**	Independent Self Employed Consultant
	High	-	**1. Groundwater: each site needs to be treated separately and potential for development of a high groundwater table addressed, e.g.: ridge crest or slope toe site. This blanket requirement for saturated conditions assessment is not always realistic. However, by not doing so (it) makes it more difficult for Councils to review a report.** **2. Where strengths are very high, it best to do some back analyses to assess realistic soils parameters for the assessment. Where strengths are low, it is similarly as important. Basically, each site requires specific assessment to address constraints.**	Other

	Answer		Comments	Identification
	GWT	Soil Strength		
Firm employing both geotechnical engineers and engineering geologists	High	Lower Bound	**Reasonable cf 4402 gw for 100 yr return period. However, for extreme parameters FS=1.2 would be acceptable.** **The factor of safety adopted needs to reflect:** • **the level of knowledge (e.g. estimate v measured gw and strength parameters)** • **hazard and risk (cf proposed development)** **A more rational approach would reflect the Hong Kong Geotechnical Manual for slopes approach (see attached extract) i.e. a graduated factor of safety for input parameters and hazard.**	D.N. Jennings, Works Consultancy Services Ltd (Hamilton)
	Average	Average	**"Normal acceptable practice" suggests that low probability events can have a lower factor of safety reflecting either transient, long return period, or lower bound conditions**	Worley Consultants Ltd
	High	Lower Bound	**With proviso that:** **(a) the SF > 1 with the appropriate pseudo horizontal seismic force for the site; and** **(b) the SF >1 with maximum possible groundwater profile** **What is the safety factor of a slope? Apart from man-made structures or perhaps embankment over weak soil any first order prediction analyses of a factor of safety must be viewed with suspicion within the normal bounds of field and laboratory testing and the precision of the geological model.** **It would be more appropriate that there was a qualified graduated scale for:** **(a) Level of knowledge; and** **(b) The hazard risk of failure.** **In some cases, this may require a SF >1.5.**	Glyn R.W. East, Works Consultancy Services Ltd (Auckland)

	Answer		Comments	Identification
	GWT	Soil Strength		
	High	Lower Bound	**A FOS of 1.5 should be applied under soil/rock and groundwater conditions which can reasonably be expected to occur in the lifetime of the development, (100-150 years), i.e. does not necessarily mean maximum credible groundwater condition or groundwater at the ground surface.** **The factor of safety should depend on degree of confidence in the model and the design parameters.** **If investigation or monitoring of groundwater levels is carried out to prove very wet winter conditions, then these should be adopted along with FOS=1.5.** **Consequences of failure should also be addressed where determining FOS, i.e. if low consequence of failure then FOS > 1.3 could be used, e.g. rural property and potential failure not over property boundaries. For urban works FOS > 1.5 suitable for design conditions and FOS > 1.2 for potential failure of drainage systems and extreme events of full saturation (if this is not the design condition) and earthquake.**	Tonkin & Taylor, (Auckland, Wellington, Christchurch & Dunedin)



QUESTION (5):

Is strength testing of soils required to establish strength parameters for each specific site? (If "Sometimes", please specify)

Summary of Results	Territorial Authority	OTHER		
		Geotechnical Engineers	Engineering Geologists	Firm of Engineers & Geologists
YES	4	3	0	0
NO	7	3	1	0
Sometimes	9	11	4	5
TOTAL NO. of Replies	**20*1**	**18**	**5**	**5**

Notes: *1 2No. respondents seek specialist advice

Can parameters established by testing of similar soils from other sites be used?
(If "Yes", please specify)

Summary of Results	Territorial Authority	OTHER		
		Geotechnical Engineers	Engineering Geologists	Firm of Engineers & Geologists
YES	14	17	4	5
NO	5	0	0	0
TOTAL NO. of Replies	**20*1**	**18**	**4**	**5**

Notes: *1 1No. respondent considers this question is not applicable

Comments:

	Answers		Comments	Identification
	1st Part Yes/No/ Sometimes	2nd Part Yes/No		
Territorial Authorities	No	Yes	**(2nd Part): Provided they are of a similar nature**	Territorial Authority
	Sometimes	Yes	**(1st Part): Depends on the location of the site and how it is intended to be developed** **(2nd Part): Where soil type is similar then parameters previously established are used**	Nelson City Council
	Sometimes	Yes	**(1st Part): Test results from similar soils could be used**	Territorial Authority
	Sometimes	Yes	**(1st Part): (act on recommendation of consulting engineer)** **(2nd Part): depending on proximity**	Territorial Authority
	Yes	Yes	**(1st Part): (specialist would carry out the necessary level of investigation appropriate to the site)** **(2nd Part): Adjacent sites only**	Territorial Authority
	Sometimes	Yes	**(1st Part): Where only one site is possible on a lot and it has a less than certain stability. Notable exceptions are 5 ha blocks that have many possible building sites and at least one is suitable for NZS 3604** **(2nd Part): These are part of the experienced Geotechnical Engineer's store of knowledge**	Personal View - Waitakere City Council Senior Subdivision Engineer
	Sometimes	Yes	**(1st Part): It would depend on the factors covered in responses to questions 3 and 4.** **(2nd Part): Depending on the degree of predictability and financial risk**	Bruce Dobson, Chief Engineer, Ruapehu District Council
	Sometimes	Yes	**(1st Part): Test bores and a minimum of shear vane readings will be required.** **(2nd Part): Sites which are assessed as complying with NZS 3604 need no further geotechnical investigation.**	Territorial Authority

	Answers		Comments	Identification
	1st Part Yes/No/ Sometimes	2nd Part Yes/No		
Geotechnical Engineers	No	Yes	(1st Part): Average soil strength tests can be used with a degree of confidence provided (the soils) are identified correctly. (2nd Part): Provided a number of test results are used and engineering judgement is used	C.J. Newton
	Yes	Yes	(1st Part): But may only be field tests (2nd Part): So as to provide guidelines. But always need to carry out some testing	Mark Mitchell
	Sometimes	Yes	(1st Part): If reasonably able to be measured with modern methods and reliably (2nd Part): If engineering geological assessment suggests that this is reasonable	Bruce Horide
	-	-	Refer to response to question 3. More often the question is can the job (client) afford to do what is required to support a proper investigation. Data from other sites permissible if local knowledge suggests that this is a valid approach. The validity of this should be confirmed by classification testing.	Prof. M.J. Pender
	No/Sometimes	Yes	(1st Part): If unfamiliar materials are encountered, otherwise conservative values can be used for smaller projects (2nd Part): ref above	Other
	Yes	Yes	(2nd Part): Adoption of typical but conservative soil parameters c', ϕ' and bulk density from similar soils after an assessment by a Geotechnical Specialist of the character of the site soils	A.W. Smith, Jackson Clapperton & Partners

	Answers		Comments	Identification
	1st Part Yes/No/ Sometimes	2nd Part Yes/No		
	No	Yes	**(1st Part): Net economically viable for domestic sites. We would adopt conservative values based on experience. Lab testing would apply where costs ensured construction savings or safety demanded testing (2nd Part): As an intermediate option between zero and full testing this approach is viable providing a geological or simple lab test correlation (e.g. Atterberg limits) can be found**	Paul Russell, Royds Consulting Nelson
	Sometimes	Yes	**(1st Part): When no knowledge of a particular soil's properties is held, when the significance of slope failure is high in terms of the hazard to life and property. (2nd Part): For uniform soil types for which extensive knowledge of strength parameters is held, preferably, but not essentially, when classification parameters are available to allow correlation.**	Fraser Thomas Ltd
	Sometimes	Yes	**(1st Part): Normally I would not consider slip circle or other methods of analytical analysis to be of much importance for assessing house site or subdivision stability. However, on questionable sites, especially sites where past movement has clearly occurred, some test data on c', ϕ' values may be of assistance (2nd Part): Sometimes, if it is clear that soil conditions at the other site are essentially identical to those at the site under consideration**	Laurie Wesley

	Answers		Comments	Identification
	1st Part Yes/No/ Sometimes	2nd Part Yes/No		
	Sometimes	Yes	**(1st Part): See below. Index tests (limits, moisture content) are necessary to confirm correlation with other sites.** **(2nd Part): Provided those test results, including index tests to correlate soil types, are presented with the report, or identified by specific reference to the "TA's" files where the results are available to the TA relay.**	D.K. Taylor
	Sometimes	Yes	**(1st Part): Depends on soil type and uniformity of soils** **(2nd Part):** **• if origin of soils known to be the same** **• if previous testing has verified this**	Other
	Yes	Yes	**(1st Part): Unless there are circumstances which make it obviously unnecessary i.e. back analysis of an adjacent slip.** **(2nd Part): Depending on how critical the results are to the final outcome.**	Peter Geddes, Hawthorn Geddes (Civil & Structural)
	Sometimes	Yes	**(1st Part): This depends on local geology and previous testing in (the) region, e.g. Port Hills loess in Christchurch has been extensively tested and shows little variation over a wide area.** **(2nd Part): as above**	Other
	Sometimes	Yes	**(1st Part): Valid for large scale projects - strength testing expensive - conservation in design can have major cost implications** **(2nd Part): Requires good statistical data base - implies co-operation between Consultants & TA's**	Soil & Rock Consultants

	Answers		Comments	Identification
	1st Part Yes/No/ Sometimes	2nd Part Yes/No		
	Sometimes	Yes	**(1st part): Strength testing may be required if the stability of the site is a serious concern because of the performance of the natural slope or the intended major slope modification, nature of development e.g. water retaining structures. (2nd Part): Need to be confident about the similarity and should only use parameters to determine sensitivity of site stability**	Brian Patterson
	Sometimes	Yes	**(1st Part): If site conditions are unusual or Engineer lacks experience, tests should be done. We have a Telarc laboratory and have built up an extensive library of test results. We often rely on these, especially low budget jobs. (2nd Part): If Engineer is sure conditions are similar**	R. Melville-Smith, Foundation Engineering Ltd
	No	Yes	**(1st part): Experience is often much better than testing for actual soils. Back analysis too and parametric analyses can give a much better understanding of how a slope may perform under a range of conditions than rigid adherence to test results. (2nd Part): Where conditions can be correlated with index properties and the geological setting is consistent.**	N.S. Luxford, Babbage Consultants Ltd
Engineering Geologists	Sometimes	-	**(1st Part): For loess, ϕ is relatively constant, but the huge wild card is cohesion ranging from 1-2 kPa to > 100 kPa. Most is apparent cohesion from soil suction**	Mark Yetton
	Sometimes	Yes	**(1st Part): 1 - When the importance of the site warrants. 2 - When the geological and groundwater conditions dictate that testing of specific strata is warranted. (2nd Part): When the regional geology is well known and when a large body of representative strength data has been collected over time.**	Riddolls & Grocott Ltd

	Answers		Comments	Identification
	1st Part Yes/No/ Sometimes	2nd Part Yes/No		
	No/Sometimes	Yes	**(1st Part): Back analysis can be used and is often better than testing if failure surface is well known, and present stability condition**	R.D. Beetham
	Yes/Sometimes	Yes/ Sometimes	**(1st Part): If quantitative analysis is being carried out then strength testing should be carried out unless exactly similar materials have already been tested. There are situations when generalised parameters (based on experience) can be used. (2nd Part): It is quite feasible to carry out back analysis of a failed slope using assumed parameters based on experience, but for forward analysis this is debatable.**	Dr D.H. Bell
	Sometimes	Yes	**(1st Part): Where the following are absent: not where geological analogy can be drawn and that experience/ database can be applied to the site or where back analyses can be used. Back analysis can be worth a 1,000 Test! (2nd Part): As above (analogy?)**	Independent Self Employed Consultant
	Sometimes	Yes	**(1st Part): Important for marginal sites however, in general observation/ measurement of existing slopes in the vicinity should provide the necessary parameters. (2nd Part): Soils are heterogeneous so all additional data to assist in an assessment is useful.**	Other
Firm employing both geotechnical engineers and engineering geologists	Sometimes	Yes	**In the majority of cases terrain evaluation and back analyses of a know geological unit provides suitable effective strength parameters for analyses but one would need to be careful about assigning an explicit safety factor. If an explicit non qualified safety factor is required (as in 4/) there seems no way out but to test each site.**	Glyn R.W. East, Works Consultancy Services Ltd (Auckland)

	Answers		Comments	Identification
	1st Part Yes/No/ Sometimes	2nd Part Yes/No		
	Sometimes	Yes	**(1st Part): Yes, if site does not satisfy the coarse sieve observation/precedent assessment. Yes, if there is no local information/experience. (2nd Part): Yes, provided the geology and stratigraphy are similar. Need to establish ground conditions/geology with confidence.**	D.N. Jennings, Works Consultancy Services Ltd (Hamilton)
	Sometimes	Yes	**(1st Part): For critical slopes, difficult sites or multi-layer soil profiles may need triaxial tests, otherwise undrained shear strengths are adequate as a guide. (2nd Part): For well known soils with "typical" properties. Budgets often do not allow for expensive triaxial tests.**	Engineering Geology Ltd
	Sometimes	Yes	**(1st Part): If sensitivity analyses indicate marginal stability for average conditions, then actual strengths are useful. This also applies to other elements of the model (groundwater, geology) (2nd Part): All testing represents a point sample and is therefore only representative of conditions. If material identification is adequate, then using "off-site" results can also be "representative".**	Worley Consultants Ltd

	Answers		Comments	Identification
	1st Part Yes/No/ Sometimes	2nd Part Yes/No		
	Sometimes	Yes	**(1st Part): This is not practical or appropriate for the majority of small site investigations. Correlations should be established between conventional field testing (shear vane/Scala penetrometer) and effective strength parameters. If testing not done, then parameters must err on conservative side and factor of safety must be higher than if testing done. Back analysis of existing failures in similar conditions is very useful and the use of sensitivity analyses is encouraged. (2nd Part): Needs judgement and experience. Parameters can be derived by:** • **Back-analysis of existing slips in similar soils** • **Correlation of conventional parameters** • **However, important to establish shear strengths of materials critical to stability for identified failure mode**	Tonkin & Taylor Ltd, (Auckland, Wellington, Christchurch & Dunedin)

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(Refer page 149 for more information)

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QUESTION (6): What factor of safety is required for earthquake loading?

QUESTION (7): What earthquake return period is required?

	Comments	Identification
Territorial Authorities	6. Variable 7. Elastic design	Territorial Authority (TA)
	6. Code Requirement 7. We are in a "B" rated area	TA
	6. 1.5 7. 150 years	Nelson City Council
	6. nil 7. not required	Wanganui District Council - T. Moran
	6. NA 7. NA	TA
	6. as per code 7. as per code	TA
	6. 1.2 7.	Waitakere City Council - Senior Subdivision Engineer - personal view)
	6. 1.2 7. 150 years	TA
	6. 1.2? 7. 450 years?	TA
	6. It would depend on all of the above factors including the sophistication of the analysis, 2.0 - 4.0 say 7. Would depend on above factors, 50-120 years	Bruce Dobson, Ruapehu District Council
Geotechnical Engineers	6. 1.1 7. 100 years	Babbage Consultants Ltd - N.S. Luxford
	6. 1.2 7. 150 years	Foundation Engineering Ltd
	6. 1.2-1.3 7. > 100 years	Soil & Rock Consultants
	6. 1.0-1.2 depending on location, importance, etc. 7. 450 year to be consistent with loadings code	Other
	6. - 7. Location dependent	Hawthorn Geddes Civil & Structural - P. Geddes

	Comments	Identification
	6. 1.1 7. ?	Fraser Thomas Ltd
	6. Building Code - 1.2 7. not specified	Royds Consulting Ltd - Paul Russell
	6. 1.2 dependent on adopted loading 7. relative to economic life of anticipated use	Jackson Clapperton & Partners - A. Smith
	6. Residual displacement may be a better criteria 7. Depends on site location and consequences of failure	Other
	6. 1.3 7. 150 years	Mark Mitchell
	6. 1.2 7. 150 years	Bruce Horide
Engineering Geologists	6.& 7. Depends on sensitivity of site/structure, etc.	Independent Self Employed Consultant
	6. Depends how well other parameters are known 7. Varies depending on site etc.	R.D. Beetham
	6. - 7. 100 years = design life of structure	Brian Patterson
	6. > F5 = 1.0 7. 50 years	Riddolls & Grocott Ltd
	6. > 1.1 7. 50 year	Geotech Consulting Ltd - M. Yetton
	6. 1.3? 7. 100-150 years	Dr D.H. Bell
Firms employing both geotechnical engineers & engineering geologists	6. recommend 1.3 for 7. 150 year return period	Engineering Geology Ltd
	6. 1.2 7. nominally 150 years	Worley Consultants Ltd

	Comments	Identification
	6. Depends on the sensitivity of the structure to damage, consequences of failure and also the soil type • for MCE (max. credible earthquake), FOS ≥ 1.0 • for DBE (design basis earthquake), FOS > 1.2 7. What earthquake return period is required: • for DBE (design basis earthquake) return period FOS ≥ 150 years • again the return period adopted is dependent on type of structure and the consequences of failure	Tonkin & Taylor Ltd, (Auckland, Wellington, Christchurch, Dunedin)
	6. > 1.0, Displacement approach (Newmark sliding block) not acceptable for a building 7. 450 year (cf NZS 4203)	D.N. Jennings, Works Consultancy Services Ltd, Hamilton
	6. SF (earthquake) > 1 7. Return period = 150- years (qualified by the hazard condition)	Glyn R.W. East, Works Consultancy Services (Auckland)

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QUESTION (8) Is a factor of safety less than 1.5 acceptable for extreme conditions (excluding earthquake)? (If "Yes", please specify the extreme conditions and factor of safety)

Summary of Results	Territorial Authority	Other Geotechnical Engineers	Engineering Geologists	Firms of Engineers & Geologists
No	11	1	0	0
Yes	5	13	6	6
TOTAL NO. of Replies	**16**	**14**	**6**	**6**

	Answer (Yes/No/ Sometimes)	Comments	Identification
Territorial Authorities	No	**But depends on definition of 'extreme' i.e. generally NOT acceptable but prepared to listen to argument re 'extreme' case**	TA
	No (generally)	**Or at least I can't think of the circumstances, although we would be fairly relaxed if the application was to build a "chook house"**	Bruce Dobson, Ruapehu District Council
Geotechnical Engineers	Yes	**A factor of safety of 1.5 is set to allow for variations from anticipated conditions. Using worst credible soil strength parameters and ground water conditions a Factor of Safety = 1.0 - 1.2 may be acceptable depending on the perceived likelihood of occurrence/exceedance and the consequences of failure.**	Babbage Consultants Ltd - N.S. Luxford
	Yes	**Fully saturated ground conditions case when such conditions are extremely likely to occur or can only occur for short periods of time. Here I use 1.2 (or would if Council agreed)**	Foundation Engineering Ltd - R. Melville-Smith
	Yes	**(i) Temporary works 1.3-1.4** **(ii) Rural blocks - slopes away from development 1.3** **(iii) In certain cases, fully saturated slope 1.3-1.4** **(iv) Lower bound shear strength & high GWT combination 1.3-1.4** **(v) Where qualitative assessment largely positive e.g. mature bush on slope**	Soil & Rock Consultants
	Yes	**Assuming lower bound conditions for water table, soil strengths, etc., say 1.3**	Other

	Answer (Yes/No/ Sometimes)	Comments	Identification
	Yes	I think the assessment of a slope should be that it has "adequate stability to be acceptable as a house site", which implies it has a minimal (but not zero) risk of failure in any reasonably foreseeable circumstances. Trying to go beyond such an assessment is futile; it will produce numbers not much related to reality.	Laurie Wesley
	Yes	Not tested in this area Live load, vehicle loads	Royds Consulting Ltd - Paul Russell
	Yes	Possibly 1.3 to 1.4 for 100 year return period event, but dependent on confidence in adopted soil parameters	Jackson Clapperton & Partners - A. Smith
	Yes	Under a 1:150 year return period earthquake a f.,o.s. of 1.3 would seem adequate	C. Newton
	Yes	1.20 low significance - e.g. rural road 1.30 moderate significance 1.40 high significance - e.g. hospital acceptable for extreme conditions say 1 in 10 year return period storm	Fraser Thomas Ltd
	Yes	When have good reason to conclude that a good understanding of the situation is available. Control of the soil present - a compacted embankment rather than a natural slope. Consequences of failure - an embankment for a minor road not as serious as a building foundation. When it is possible to work from the back analysis of an existing failure. When very good groundwater monitoring data is available. (The cost of groundwater monitoring could be set against the gains from using a lower factor of safety.)	Prof. M.J. Pender
	Yes	1.3 OK if soil parameters were established, e.g. man made dam Also OK if average strength parameters used rather than minimum values	Mark Mitchell
	Yes	1. Earthquake - NZS4203 - design : 1.1 2. g.w.l. - upper bound and soil strength lower bound together: 1.1	Bruce Horide

	Answer (Yes/No/ Sometimes)	Comments	Identification
Engineering Geologists	Yes	**Provided earthquake and other eventualities have been ... for in the analysis**	Independent Self Employed Consultant
	Yes	**FS < 1.5 was used in <u>all</u> the Cromwell Gorge slide stabilisation works. Often ~ 5% increase in stability was all that could realistically be achieved**	R.D. Beetham
	Yes	**Short duration high intensity rainfall events**	Riddolls & Grocott Ltd
		In Auckland we do not normally design for earthquake loadings	Other
	Yes	**Because most established hill areas in Christchurch with no significant slope problems when back analysed have FS < 1.5 for prolonged rainfall**	Geotech Consult. Ltd - M. Yetton
	Yes	**I would suggest that FS = 1.5 is far too high for most situations, especially given the numbers of properties located on landslides in New Zealand. Where structures such as retaining walls form part of the site design. I would agree that 1.5 is realistic with earthquake loading**	Dr D.H. Bell
Both geotechnical engineers & engineering geologists	Yes	**Unrealistically high groundwater FOS = 1.3 adequate when remediating existing landslide**	Engineering Geology Ltd
	Yes	**- Extreme groundwater levels - Long term downcutting/erosion of slope toe by stream action**	Worley Consultants Ltd
	Yes	**Extreme conditions are defined as the most unfavourable soil/rock and groundwater conditions which can be reasonably expected to occur in the lifetime of the structure. The factor of safety should be related to the probability of groundwater conditions.** **Where sufficient investigation/monitoring shows that a groundwater level at surface condition is very unlikely, then could accept FOS < 1.5 (but > 1.2 for extreme conditions). This also applies to failure of drainage systems, particularly horizontal drains which are prone to blockage during the lifetime of a structure.**	Tonkin & Taylor Ltd, (Auckland, Wellington, Christchurch, Dunedin)

<table>
<tr><th></th><th>Answer (Yes/No/ Sometimes)</th><th>Comments</th><th>Identification</th></tr>
<tr><td rowspan="3"></td><td></td><td>Also where an existing failure occurs and requires remedial work, back analysis provides greater confidence and therefore a FOS > 1.3 could be acceptable under some circumstances e.g. large mass landslide. Effects of creep need to be considered for such cases and transient piped water tables causing shallow instability. Also dependent on consequences of failure.</td><td></td></tr>
<tr><td>Yes</td><td>cf the Hong Kong approach
<table>
<tr><th>Geology</th><th>Soil Strength</th><th>Ground-water</th><th>Dev. Definition/ Slope</th><th>FS</th></tr>
<tr><td>Known
Known</td><td>Lower bound
Lower bound</td><td>Average
High (> 100 yr)</td><td>Known
Known</td><td>1.5
1.2</td></tr>
</table></td><td>D.N. Jennings, Works Consultancy Services Ltd, Hamilton</td></tr>
<tr><td>Yes</td><td>Under conditions that the safety factor of a cut/natural slope can be satisfactorily controlled to 1.5 with drainage. Or in the case of an embankment with weak foundation soils that will increase to 1.5 with time.

As mentioned in 4/ a graduated scale for (a) Level of knowledge; and (b) The hazard risk of failure, may be more appropriate</td><td>Glyn R.W. East, Works Consultancy Services (Auckland)</td></tr>
</table>

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(see Page 149 for more information)

QUESTION (9) **If drainage works are used to stabilise land, should their failure to operate be considered? (If "Yes", please give circumstances).**

Summary of Results	Territorial Authority	Other Geotechnical Engineers	Engineering Geologists	Firm of Engineers & Geologists
No	0	2	1	0
Yes	15	14	6	5
TOTAL NO. of Replies	**15**	**16**	**7**	**5**

	Answer (Yes/No/ Sometimes)	Comment	Identification
Territorial Authorities	Yes	**Old age, failure to maintain, progressive blockage with fines**	TA
	Yes	**It is important that any mitigation measures continue to work and are regularly maintained**	TA
	Yes	**Land should still be stable to 1.2**	Waitakere City Council, Senior Subdivision Engineer - personal view
	Yes	**Access for clearing, cleaning etc. to be considered at outset. Access for inspection/repair to be considered**	TA
	Yes	**Our experience is that subsoil drains can become inoperative within 2 or 3 years. Once again would depend on all the above factors and the extent to which the drains were designed to remain operative**	Bruce Dobson Ruapehu District Council
	Yes	**Under the failure of bored drains, a factor of safety of 1.2 should still be maintained provided that the factor of safety of 1.5 is achieved with the installation of drains.**	TA
Geotechnical Engineers	Yes	**The implications should always be assessed. A low factor of safety under failure but at least 1.0 should be considered.**	Babbage Consultants Ltd - N.S. Luxford
	Yes	**But only if a factor of safety of 1.2 can be tolerated under these conditions**	Foundation Engineering Ltd - R. Melville-Smith

	Answer (Yes/No/ Sometimes)	Comment	Identification
	Yes	**Proper installation and maintenance of drains is necessary - does not happen though. If drainage required to raise FS from less than 1.3 to 1.5 failure of the drains is of greater concern**	Soil & Rock Consultants
	Yes	**Drainage works need to be designed to be fail safe in some way, or conservatively designed to ensure satisfactory performance in accordance with good engineering practice**	Other
	Yes	**Consider a compliance schedule (hydraulic structure) to ensure they remain operational through ABWF inspection. Refer to Whangarei District Council policy on dams**	Hawthorn Geddes Civil Structural - P Geddes
	Yes	**Of course. A major deficiency of many reports is a lack of direct long term measurement of piezometric pressures/water levels.**	D.K. Taylor
	Yes	**Bored horizontal drains are probably the least reliable on a long term basis, and hence their failure to operate should certainly be considered**	Laurie Wesley
	Yes	**A sensitivity study should be undertaken to assess reduction in F/S due to rise in g.w.l. following drainage failure, and this related to confidence in the maintenance procedures proposed to apply to the drainage system.**	Jackson Clapperton & Partners - A. Smith
	Yes	**This should be investigated and some degree of redundancy built into the works should be considered as part of the design**	C Newton
	Yes	**Otherwise what is the purpose of the drainage works? Perhaps one might say that having decided to install drainage works there is a maintenance responsibility on the owner**	Prof. M.J. Pender
	Yes	**Only to establish the extent of maintenance required**	Mark Mitchell
	Yes	**1. Long term 2. Risk of third party injury/damage 3. Variability/reliability of drain construction materials/ workmanship**	Bruce Horide
	Yes	**If consequences of failure are likely to be serious**	Fraser Thomas Ltd
Engineering Geologists	Yes	**Where their failure would affect .....**	Independent Self-employed Consultant
	Yes	**Drainage and other remedial works require regular checking and maintenance to ensure their proper function**	R. D. Beetham
	Yes	**If failure of drainage is critical to stability and consequences of slope failure is unacceptable**	Brian Patterson

	Answer (Yes/No/ Sometimes)	Comment	Identification
	Yes	**If stability is dependent on drainage this eventuality must be addressed**	Other
	Yes	**All drains should be accessible for later cleaning by flushing or pig. If this is done they can be viewed as acting indefinitely**	Geotech Consult Ltd - M Yetton
	Yes	**Long-term maintenance of drainage measures must be carried out in all situations involving slope stability. This raises the question as to who assumes responsibility once consents are issued, and these matters should be addressed at that stage.**	Dr D.H. Bell
Firms employing both geotechnical engineers & engineering geologists	Yes	**But would allow a much lower safety factor than 1.5**	Engineering Geology Ltd
	?	**Drainage works that have been engineered and constructed according to acceptable standards should have the normal performance expectations of other engineered works (e.g. retaining walls). Rather than imposing a failure criterion, it would be better to ensure that any works are designed adequately and have been reviewed as part of the normal checking process.**	Worley Consultants Ltd
	Yes	**Monitoring systems such as piezometers or drain discharges should be included in drainage works in situations where failure of the drains could cause instability problems** **Where appropriate other components of the remedial works should be designed to allow for drain failure, i.e. foundations, retaining structures. In this situation, a reduced factor may be acceptable.** **Note potential for shallow failures and creep in Q.8 above**	Tonkin & Taylor Ltd, (Auckland, Wellington, Christchurch, Dunedin)
	Yes	**There are concerns about acceptability of subsurface drainage in a domestic environment related to reliability of maintenance**	D.N. Jennings, Works Consultancy Services Ltd, (Hamilton)
	Yes	**The maintenance of remedial or stability improvement drainage is not adequately addressed.** **A formalised approach to maintenance should be required for a building or subdivision consent that includes stability drainage.**	Glyn R.W. East, Works Consultancy Services (Auckland)

QUESTION (10) What role does peer review of assessments play?

	Comment	Identification
Territorial Authorities	None	TA
	When the recommendations come up with requirements or approvals we do not consider are appropriate we ask for a peer review	Nelson City Council
	Building Consents are not generally reviewed by a Registered Engineer	Wanganui District Council - T Moran
	No formal review	TA
	1812 Overture	TA
	Peer review would be appropriate in high risk situations - i.e. where failure would likely cause loss of life or extensive physical damage	TA
	Auckland City Council carries out a peer review of all geotechnical investigation. If there is any disputes further independent review may be sought.	TA
	Important	TA
	None so far. Queries are redirected to the Soils Engineer for further consideration	Waitakere City Council Senior Subdivision Engineer - personal view
	Significant, especially when very specialised reports, calculations are received	TA
	Council does not employ Geotechnical staff and to date has not engaged consultants to review another Geotechnical specialist investigation/design	TA
	ALL: Peer review is important depending on the degree of complexity, cost, etc.	Bruce Dobson, Ruapehu District Council
Geotechnical Engineers	Internal peer review is used on a routine basis	Babbage Consultants Ltd - N.S. Luxford
	Helps ensure a more uniform (and higher) standard of stability reporting	Foundation Engineering Ltd, R Melville-Smith
	Maintains a standard of quality in an area where rigorous rules are difficult to establish. However, guidelines on the standards to be achieved is necessary to avoid confusion and argument.	Soil & Rock Consultants

	Comment	Identification
	For important, or difficult sites, peer review is appropriate	Other
	Important for difficult or high impact situations. Not required in all cases	Hawthorn Geddes, Civil Structural - P Geddes
	To ensure that basic principles are sound	Other
	Should be restricted to a check of procedures, content and method and NOT regarded as a second opinion in disguise.	D.K. Taylor
	Indemnity insurance spread i.e. risk spread	Other
	Most of our work is domestic; checking is carried out in-house under our QA plan	Royds Consulting Ltd - Paul Russell
	Provide quality assurance to ensure that all significant factors are taken into account in the assessment.	Fraser Thomas Ltd
	Appropriate to differences in opinion between assessor, and approving authority	Jackson Clapperton & Partners - A Smith
	All projects of this nature which has a degree of risk should be reviewed by another Geotechnical specialist	C Newton
	Valuable independent assessment	Prof. M.J. Pender
	Should only be required on major structure, or where the TA has some concerns with the design	Mark Mitchell
	Essential, but for bottom line recommendations only i.e. yes/no answers	Bruce Horide
Engineering Geologists	**Leads to consistency but also possible BIAS**	Independent Self-employed Consultant
	Essential for major works	R.D. Beetham
	Very important role of checking adequacy of the investigation and determination/judgement/ interpretation of data	Brian Patterson
	Can only be justified for very large projects/assessments	Riddolls & Grocott Ltd
	Maintains and improves standards by dissemination of ideas and methodology	Other
	Only required for large projects where the budget normally means reasonable testing and good data is available, plus the time for the inevitable discussions	Geotech Consult Ltd - M. Yetton

	Comment	Identification
	I would expect peer review in critical situations where there is concern for a particular site, or for the analysis that has been presented in support of a consent application. This poses real problems for smaller TAs who may not have sufficient expertise to recognise this concern. Peer review "between consultants" would rarely be required on residential slope stability issues.	Dr D.H. Bell
Firms employing both geotechnical engineers and engineering geologists	Useful for difficult or critical stability situations. Not generally required if experienced person has prepared report.	Engineering Geology Ltd
	Peer review should be mandatory for all geotechnical assessments involving public liability	Worley Consultants Ltd
	Peer review provides the client/local authority with assurance that an appropriate professional assessment has been undertaken (it usually involves identifying issues which may have been overlooked). It has a significant benefit in promoting professional standards	D.N. Jennings, Works Consultancy Services Ltd (Hamilton)
	• To ensure a satisfactory level of investigation/analysis/design is applied, appropriate to the slope stability hazards/risks and the implications to the development. • The review is then carried out by suitably qualified professionals and not the most available "qualified" professional within a TA. • A peer review may allow more reasonable and economic design. Particularly useful to endorse matters of judgement and to confirm acceptable levels of risks	Tonkin & Taylor Ltd, (Auckland, Wellington, Christchurch, Dunedin)
	A peer review provides a TA/Client without specialist geotechnical knowledge some assurance that all issues have been addressed. Perhaps whether a peer review is required is part of a qualified graduated hazard scale (4/ above). The peer review should not be a "nit-picking" exercise and ideally the project should be discussed with the reviewer throughout the investigation and assessment, i.e. the reviewer should be appointed at an early stage.	Glyn R.W. East, Works Consultancy Services (Auckland)

QUESTION (11) How is Section 36(2) of the Building Act applied?

	Comment	Identification
Territorial Authorities	Construction is registered against the title. The act says shall.	TA
	Consents refused for known problem sites	TA
	We require a certificate from a geotechnical specialist advising that 2(a) will apply before acting on Section 36(2) ourselves	Nelson City Council
	Generally only flooding is considered and then really only in respect to floor levels. Where a site has been considered a problem by say myself, the applicant can usually find a Registered Engineer to sign a letter etc. to say that the site is suitable for building on etc. This TA would rarely refuse to grant a building consent and under previous legislation in one case refused a building permit for similar reasons to S36, was challenged in court - lost, issued a permit, the site slipped causing damage, Council was successfully sued for damages!!	Wanganui District Council - T Moran
	This is handled by the Regulatory unit of council. We (Engineering Services) are seldom consulted at the Building Consent stage even if a lot has been "tagged".	TA
	According to law	TA
	In the knowledge of some potential failure but, as best as can be determined, that the proposed works will not exacerbate the situation. Only a few actions taken to date	TA
	Where building and site works such as driveways and services are suitably protected against land movement, but still there is a risk of land movement or subsidence. Council can approve a building consent under Section 36(2) of the Building Act and reference may be made to this clause when subdivision consent is given.	TA
	We have not had any cases since the Building Act has become law.	TA
	It is applied on: - known sites which have a past history of trouble - known areas which have a past history of trouble - doubtful proposals where developers are not prepared to take all reasonable steps to protect subsequent structures - situations where it is felt the development is unwise for a number of reasons. Therefore the developer may take the risk if they so choose but Council chooses not to partake in that risk. (Private Certifiers may overcome this last situation) All cases judged on an individual basis	TA
	Generally by applying judgements to each particular circumstance	Bruce Dobson, Ruapehu District Council

	Comment	Identification
Geotechnical Engineers	My interpretation is that Council cannot refuse to grant consent under Section 36(2) subject to its disclaimer. This is dangerous for the Geotechnical Engineer who carries the whole can. His disclaimer should be loud and clear in this case.	Foundation Engineering Ltd, - R Melville-Smith
	Have had little experience of the TA applying this Section of the Act	Soil & Rock Consultants
	If there is no hazard to life, and owner has been fully briefed of the risks, Sec. 36 is appropriate	Other
	Generally consistent within each TA but different TA's are applying it differently. Little change from 641(A) in the past	Hawthorn Geddes, Civil & Structural - P Geddes
	This is an extraordinary return to the "Caveat Emptor" philosophy which may lead to some interesting court decisions in future. The TA should apply it, not the consulting engineer	D.K. Taylor
	Has significant influence on resale value of existing properties if applied in recent times	Other
	NCC (Nelson City Council) have a hazard map. Any property with a notice recorded against it needs an opinion. If the Building Regulations (Clause B1) cannot be met, i.e. low probability, Section 36(2) of the Act must be resorted to. The TA accepts the opinion of the Geotech. Engineer with respect to the work not worsening, accelerating or resulting in any of the listed effects. The TA tags the title as required.	Royds Consulting Ltd - Paul Russell
	Appropriate to ancillary structures, physically separate from habitable buildings where the owner is fully aware of the condition and accepts the risk of probably uninsurable loss.	Jackson Clapperton & Partners - A Smith
Engineering Geologists	The council requires the consultant to address the risks of building on the site i.e.: • stability • flooding • inundation If subject to specific design they identify this on the title	Other
	With great caution in Christchurch. The CCC have not used it yet to my knowledge for hill property. They are concerned regards their liability for injury or death. I disagree with their interpretation of the Act and consider Section 36(2) a useful approach in a lot of cases	Geotech Consult Ltd - M Yetton
	I recommend s36(2) approvals in cases where I endorse house site construction on existing "dormant" landslides. This is based on the premise that the site is currently stable, and that drainage and other measures would not lessen the status quo. Implicit in s36(2) approvals is the acceptance of long-term risk by the property owner, who must be fully informed of these issues at the time. No quantitative analysis is carried out in such cases, as the costs would be prohibitive, and there is reliance of geological precedent.	Dr D.H. Bell

	Comment	Identification
Firms employing both geotechnical engineers and engineering geologists	**Very unevenly applied amongst TA's - seems to absolve them of all risk. Many TA's overcautious and use 36(2) to cover their uncertainties.**	Engineering Geology Ltd
	Cases assessed on their own merits and client would be advised of potential problems with site	Worley Consultants Ltd
	The geotechnical advisor should: **(1) Advise the client very clearly on the slope stability hazards and risks affecting the sites and the likely implications to the development.** **(2) Where appropriate, design remedial works to improve the FOS to meet the Building Code Requirements (modified to take foreseeable conditions into account)** **(3) Recommend against purchase/development in situations which do not comply with the Building code (under Registered Engineers)** **(4) Advise client of consequences of construction under 36(2). No acceptance of risk by Council, EQC and limited (defined) liability accepted by Designer, i.e. at client's own risk.** **(5) Recommend structures which are relocatable.**	Tonkin & Taylor Ltd, (Auckland, Wellington, Christchurch, Dunedin)
	No experience with this provision for slopes. This provision has been used for a site where there was potential for consolidation induced settlement	D.N. Jennings, Works Consultancy Services Ltd, Hamilton
	Section 36(2) is relatively clear providing the TA accepts the geotechnical stability assessment and the peer review. A client must accept the fact that geotechnical specialist cannot alter ground conditions to suit his requirements. It must be accepted that some hazard building sites such as some cliff tops and some North Auckland Onerahi Chaos sites must be "tagged". Some problems have been experienced in defining the "life of the development" in relationship to some natural progressive instability such as cliff top regression which should be qualified by TA's.	Glyn R.W. East, Works Consultancy Services (Auckland)

QUESTION (12) Please make any other comments

	Comment	Identification
Territorial Authorities	We will soon be requiring specific design of foundations plus geological reporting for all buildings on sloping sections	TA
	Normal procedure at subdivision approved stage is to impose a condition to the effect that "specific foundation design will be required at building consent time". Regulatory unit are advised of this requirement and expected to police it.	TA
	Because of a lack of projects which have required slope stability analysis, this Council has not developed a policy on the issue. It would welcome some guidance on the issues involved	TA
	We have not had any applications for land development on ground subject to slope stability problems. Our District is relatively "flat" and has not been a problem. However, I am interested in the outcome of this questionnaire so we can have a better "understanding" of the current and generally accepted practice.	TA
	Council tries to have a reasonable attitude towards all of geotechnical investigations.	TA
	Building Act Section 36(2) - presently a grey area. TA's are using 36(2) but I understand the DLR is not cooperating - Act doesn't spell out who pays for this part of the process. Until a case is tested in law we may be kidding ourselves thinking that all is in order. Applicants don't like 36(2) and yet it is reasonable from the TA viewpoint that they have an out when there is real doubt over certain sites. TA's under the Act accept considerable liability for work they don't do i.e. builder does the work but TA issues the CCC : TA can't be on site to observe everything yet once CCC issued becomes liable. Applicants must realise that some sites should be left alone - Abbotsford comes to mind - irrespective of their potential to make suitable home sites. Rarely does the TA gain the profit from the subdivision but it ends up carrying the mistakes and short cuts left by the developer.	TA
	This covers land development only. Comments do not apply to Building Consents	TA

	Comment	Identification
	This District mainly deals with smaller buildings i.e. houses and holiday homes. Larger buildings including all industrial developments are on essentially flat land.	TA
	Our level of engineering input into building permits has been low to date, but our experience of local conditions and experience is extensive mainly through roading construction. We have not had to give a "hell of a lot" of thought to Section 36(2)	Bruce Dobson, Ruapehu District Council
Geotechnical Engineers	A qualitative assessment should always be carried out as part of any overall slope stability assessment. The quantitative or analytical assessment forms only part of the overall assessment. However, the problem is that many Consultants and TAs become pre-occupied with the Factor of Safety of 1.5 which can result in certain cases of an unsound or conservative judgement being made	Soil & Rock Consultants
	While there is a general tendency towards specialisation and in many circumstances this is justified, the policy proposed needs to recognise the very wide range of circumstances and possible impacts of the sort of work which will be covered by it. The society (Geomechanics) does not want to be seen as attempting to "close the shop".	Hawthorn Geddes, Civil & Structural - P Geddes
	Some TA's want to avoid direct involvement of their staff in slope stability assessments and place "first-line" responsibility upon consultant's reports (usually engaged by "developers") perhaps with Peer Review. In that case the TA must show discretion as to the capability of the reporters - this can be more or less formal - <u>but</u> it is much more important that TA's promulgate Codes of Practice which set out fairly detailed requirements for the extent of investigations and the content of reports. This is essential in order to: (a) bring the reporters to a reasonable standard of capability (b) prevent reporters being pressurised by "developers" who do not want to face the real costs, but will pass off the responsibility to the reporters. Some SI reports, currently, are woefully deficient in fundamental aspects well observed by experienced practitioners.	D. K. Taylor

	Comment	Identification
	I do not think the blanket requirement of a SF of 1.5 is a sensible approach to this issue. The approach should be that a competent person or organisation undertake the stability assessment, and their report should be the basis on which a decision is made as to whether or not the site is acceptable. The geotechnical issue with regard to numerical stability analysis is not simply one of measuring c′ and Ø′, and the seepage situation, it is whether the overall geology of the site has been properly identified and whether its implications for stability are appreciated. Thinking of stability assessment as simply an exercise in obtaining c′ and Ø′ values is "unhealthy".	Laurie Wesley
	Nelson City Council have kept hazard maps since circa 1975. Their procedure is consistent and established. They have also required geotechnical consultants including geologists to have prior approval of credentials for some time. The system works logically. We very rarely use B1/VM4 for stability work apart from retaining walls. We usually use the provisions of the Building Regulations. We have regularly considered work under Sect 36(2). The NCC have insisted on a formal statement for all stability reports and this must include both the NCC and the client as the named parties covered; we can exclude use by all other parties without consultation.	Royds Consulting Ltd - Paul Russell
	Variability of soils, and difficulty of ensuring that worst case soils are identified and tested, and in this case of "back analysis", the accuracy of assessing pre movement profile means that most assessments reflect the judgement of the assessor. i.e. Geotechnical Engineering more an "ART" than a "SCIENCE" A further complication lies in the recording of an assessment where care should be taken to avoid jargon which could make a report unintelligible to a layman and product user	Jackson Clapperton & Partners - A Smith
	The "minimum FS of 1.5" needs to be replaced with: "The minimum FS shall be in the range of 1.3 to 1.8 and the value adopted shall be justified by the Designer with adequate documentation.	Mark Mitchell

	Comment	Identification
	In a non analytical approach to site stability assessment, a qualified/experienced engineering geologist is probably the best qualified person to carry this out. In an ideal world the engineering geologist in association with a geotechnical engineer should combined to carry out assessment of important sites	Brian Patterson
Engineering Geologist	Presumably this questionnaire has been prompted by: 1. The inability of IPENZ to accept suitably qualified and experienced professional professionals into its ranks as professional engineers. (I am an example - 1st Degree BSc Geol Hons Masters in Engineering Rock Mechanics with 25 years of experience in rock and slope engineering, specialising in slope stability) 2. The tendency of geologists <u>not</u> to get involved in <u>analysis</u> Both 1. and 2. are ludicrous in today's technical world	Independent Self-employed Consultant
	Slope instability needs to be assessed on a site by site basis. Blanket requirements for a FS of 1.5 are ridiculous.	R. D. Beetham
	The primary problem facing engineering geologists working in the area of slope stability assessment for subdivision or individual dwellings is the requirement of some councils for those reports to be signed by a registered engineer. It seems however to matter little if that engineer has limited experience dealing with stability of slopes. In my own case I work in with a geotechnical engineer for foundation design to address problems identified on difficult sites. It would be an advantage to both engineers and engineering geologists if IPENZ looked seriously at addressing this registration problem (?) instead of hoping it will go away. We will see an increasing number of engineering geologists working in this area in the future.	Other
	I think the FS > 1.5 is one of those "good" ideas made by somebody after reading a few textbooks. Any one who has actually done slope analysis knows how sensitive the outcome is to input data, and generally how restricted the data is. The big problems are pore water pressures and cohesion. Cohesion, either apparent or true, holds up so many slopes in nature that it cannot be ignored in analysis. Yet there are few reliable methods to assess this properly. Back analysis is the best way, and an experienced engineering geologist actually does a visual "back analysis" of the site in his assessment.	Geotech Consult Ltd - M Yetton

	Comment	Identification
	1. From more than 15 years of experience with urban planning and development in New Zealand, the greatest hazard to homes comes from debris flows generated in steel catchments. Large-scale instability, such as at Abbotsford, is rare, and the precedent (i.e. qualitative) approach outlined earlier is suitable as a first stage of assessment of slope stability. Note also that under the Building Act and RMA a number of hazards have to be addressed, and I remain satisfied that the engineering geology approach which I have developed and used is entirely suitable for this initial appraisal and at least subdivision consent approval. If every slope on which a house has to be located requires a FS - 1.5, then there is something radically wrong. 2. Quantitative analysis requires a realistic engineering geological site model, and sufficient funds must be made available for these investigations. Very few clients have the money required, and I quite simply do not believe that such investigations are warranted in the great majority of cases. As a matter of interest the pre-quarrying factor of safety at Abbotsford was 1.07, whilst Pukaki Dam has FS - 1.3 under 0.3g - nowhere near 1.5 in either case, and I am reminded also that Brewery Creek went from 0.96 to 1.03 with the spending of $100M. I am afraid that the BIA approach is flawed fatally. 3. I have recommended approval of dwellings on existing landslides in Havelock North, the Marlborough Sounds and in Queenstown. The reasons for these approvals are that the property owners are prepared to accept the risks involved, and that the development will not worsen the situation. In each of these cases s36(2) allows this fact to be identified, and future owners of the land protected. Whilst in principle I do not recommend close residential subdivision of landslide areas, I am also sufficiently well aware from precedent that in many cases people can live safely on landslides.	Dr D.H. Bell

	Comment	Identification
Firms employing both geotechnical engineers & engineering geologists	The factor of safety approach alone to ground stability assessment is inappropriate. It has evolved into a practice in lieu of Risk Assessment (i.e. it is a defacto risk assessment). It takes no account of the probabilities of events occurring that may 'trigger' instability. At best it provides a 'safety' margin against unknowns that are used in stability analyses.	Worley Consultants Ltd
	Section 36(2) should only be applied in special circumstances where the client is already financially committed to the development (i.e. the existing property or landowner) and where it is not practical to improve the stability to meet the Building Code requirements, e.g. a single property owner on a large landslide which extends beyond the owner's property. All practical steps should be taken to mitigate the hazard, and the conditions on the site should not be worsened as a consequence of the development. FOS should be determined on basis of risk, hazard and confidence of ground conditions e.g. refer to Hong Kong slope design manual. A manual like this could be developed for NZ conditions.	Tonkin & Taylor Ltd, (Auckland, Wellington, Christchurch, Dunedin)
	With regards to items 4 and 8 it is important that a realistic range of strength and groundwater parameters are considered. It is not acceptable to assume average conditions (and FS = 1.5). Where calculated stability is assessed it is important that all potential mechanisms of instability are considered and that they are realistic. Probably the most important issue is that of "what information is the assessment based on?" Too often assessments are based on insufficient data to adequately define the site and its characteristics. Consequently questions of assumptions and analysis are premature.	D.N. Jennings, Works Consultancy Services Ltd, (Hamilton)
	(a) Slope stability assessment must apply both the engineering geology terrain evaluation and an analytical assessment based on a realistic model from the geological conditions. Little confidence can be given to an engineering geology approach if the mechanisms of failure are not understood and similarly little confidence can be given to analytical analyses without a realistic geological model.	Glyn R.W. East Works Consultancy Services (Auckland)

	Comment	Identification
	(b) What is the safety factor of safety? Apart from man made structures or perhaps embankment over weak soil any first order prediction analyses of a factor of safety must be viewed with suspicion within the normal bounds of field and laboratory testing and the precision of the geological model. The arbitrary unqualified safety factor of 1.5 probably allows for the unknowns but it would be more appropriate if there was a qualified graduated scale for (a) Level of knowledge; and (b) The hazard risk of failure. In some cases, this may require a SF >1.5. Whether a peer review is required could be part of this scale. (c) TA's (and the insurance industry) should give some consideration to the terrain evaluation/"back analyses" approach to stability assessments as an alternative approach to that of a safety factor from some arbitrary analytical analysis. In some terrain, notably partially weathered volcanics and greywacke, this is the only meaningful approach. With this approach, the stability conditions of an existing slope, or nearby slope in the same geological unit and structure, is physically and analytically assessed to obtain realist soil properties that are then used on the proposed slope. The safety factor results of the new slope can only be quantitatively qualified in relation to the existing slope (as a percentage of the existing slope). With this approach, TA's would need to accept the judgement of the geotechnical specialist that the slopes the assessment was based on, are stable and the proposed slope has a similar or better qualified safety factor. The approach allows for assessments to improve the stability by drainage, etc. (d) Consideration should be given to the need, or otherwise, of a Limit State Design approach to stability of slopes. Is the concept of a Safety Factor value out of date? A Limit State Design could allow more flexibility especially in relation to the "back analyses" approach.	

News Flash for Geotechnical Engineers.

1. Did you know that Welded Beams were useful as Retaining Wall Piles ?

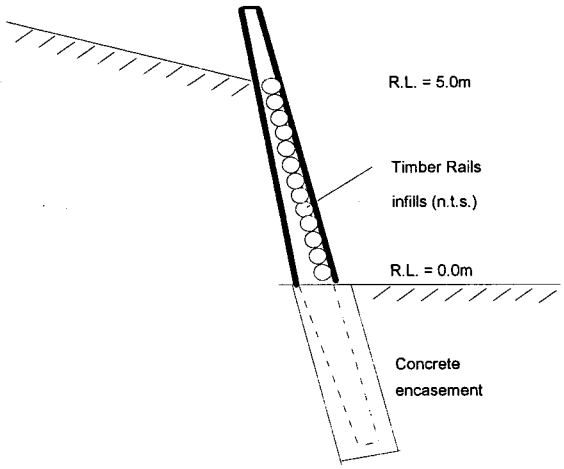

Figure 1. Westgate Retaining Piles.

A novel use of welded beams recently occurred on the Westgate Retail project in West Auckland. This was a 5m high, 250m long cantilever retaining wall with an additional surcharge loading, designed by Rodney Melville-Smith of Foundation Engineering Ltd. The retaining piles selected by Hauraki Piling were custom tapered welded steel sections reflecting the triangular loading pattern of the earth pressure, and complementing the overall appearance of the finished result.

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(ATTACHMENTS)
(supplied with responses to the questionnaire on slope stability assessments)

1. **Extract from the Hong Kong Manual for Slopes Pp 183,184 & 189 - supplied by D.N. Jennings, geotechnical engineer, Works Consultancy Services (Hamilton).**

2. **"Land Stability", Personal View - Senior Subdivisional Engineer, Waitakere City Council.**

3. **Riddolls & Grocott Ltd letter response which supplemented their questionnaire response.**

4. **David H. Bell's letter response which supplemented his questionnaire response.**

5. **Letter response and attached council requirements for geotechnical assessments, J. White, Development Engineer, Manukau Environment (Manukau City Council).**

6. **Copy of Section 36 of the Building Act 1991.**

7. **Extracts from the NZ Building Code 1993 Verification Method B1/VM4 relating to Site Investigations (2.0) and Slope Stability (3.2 - 1994).**

Table 5.1 - Recommended Factors of Safety for New Slopes for a Ten-year Return Period Rainfall

RISK TO LIFE / ECONOMIC RISK		Recommended Factor of Safety against Loss of Life for a Ten-year Return Period Rainfall		
		Negligible	Low	High
Recommended Factor of Safety against Economic Loss for a Ten-year Return Period Rainfall	Negligible	>1.0	1.2	1.4
	Low	1.2	1.2	1.4
	High	1.4	1.4	1.4

Note : (1) In addition to a factor of safety of 1.4 for a ten-year return period rainfall, a slope in the high risk-to-life category should have a factor of safety of 1.1 for the predicted worst groundwater conditions.
(2) The factors of safety given in this Table are recommended values. Higher or lower factors of safety might be warranted in particular situations in respect of economic loss.

EXTRACT: GEOTECHNICAL MANUAL FOR SLOPES,
Geotechnical Control Office,
Engineering Development Department,
Hong Kong

Table 5.2 - Typical Examples of Slope Failures in Each Risk-to-Life Category

Example	Risk to Life		
	Negligible	Low	High
(1) Failure affecting country parks and lightly used open-air recreation areas.	✓		
(2) Failures affecting roads with low traffic density.	✓		
(3) Failures affecting storage compounds (non-dangerous goods).	✓		
(4) Failures affecting densely used open spaces and recreational facilities (e.g. sitting-out areas, playgrounds, car parks).		✓	
(5) Failures affecting roads with high vehicular or pedestrian traffic density.		✓	
(6) Failures affecting public waiting areas (e.g. railway platforms, bus stops, petrol stations).		✓	
(7) Failures affecting occupied buildings (e.g. residential, educational, commercial, industrial).			✓
(8) Failures affecting buildings storing dangerous goods.			✓

Table 5.3 - Typical Examples of Slope Failures in Each Economic Risk Category

Example	Economic Risk		
	Negligible	Low	High
(1) Failures affecting country parks.	✓		
(2) Failures affecting rural (B), feeder, district distributor and local distributor roads which are not sole accesses.	✓		
(3) Failures affecting open-air car parks.	✓		
(4) Failures affecting rural (A) or primary distributor roads which are not sole accesses.		✓	
(5) Failures affecting essential services which could cause loss of that service for a temporary period (e.g. power, water and gas mains).		✓	
(6) Failures affecting rural or urban trunk roads or roads of strategic importance.			✓
(7) Failures affecting essential services, which could cause loss of that service for an extended period.			✓
(8) Failures affecting buildings, which could cause excessive structural damage.			✓

Note : These examples are for guidance only. The designer must decide for himself the degree of economic risk and must balance the potential economic risk in event of a failure against the increased construction costs required to achieve a higher factor of safety.

EXTRACT: GEOTECHNICAL MANUAL FOR SLOPES,
Geotechnical Control Office,
Engineering Development Department,
Hong Kong

Table 7.1 - Recommended Factors of Safety for Gravity Retaining Walls for a Ten-year Return Period Rainfall

Mode of Failure	Recommended Factor of Safety for a Ten-year Return Period Rainfall	
	New Walls	Remedial and Preventive Works to Existing Walls
Sliding	1.5	1.25
Rotation	2.0	1.5
	(For a masonry wall, the resultant force should lie within the middle-third)	
Bearing Capacity	3.0	Existing value to be maintained if below 3.0. For a wall with a toe slope, overall stability of the of the slope must be adequate
Slope Failure,	Refer to Section 5.2	

Note : (1) The factors of safety for remedial and preventive works are minimum values to be used only where rigorous structural, geological and geotechnical studies have been carried out, where the wall has been standing for a considerable time, and where the loading conditions, the groundwater regime and the basic form of the modified wall remain substantially the same as those of the existing wall.

(2) Should the back-analysis approach be adopted for the design of remedial or preventive works, it may be assumed that the existing wall had a minimum factor of safety of 1.0 for the worst known loading and groundwater conditions.

(3) For a failed or distressed wall, the causes of the failure or distress must be specifically identified and taken into account in the design of the remedial works.

EXTRACT: GEOTECHNICAL MANUAL FOR SLOPES
Geotechnical Control Office,
Engineering Development Department,
Hong Kong

SUBJECT: **LAND STABILITY**

DATE: 11 July 1994

BACKGROUND

Two recent reports from Geotechnical firms have discussed Factors of Safety less than 1.5 and recommended that Council accept "under extreme conditions" Factors between 1.22 and 1.27.

This brings up the question's of what is an acceptable factor of safety (F.O.S.), where is it derived from and what Council is obliged to ensure from legislation, engineering standards, the District Scheme and case law.

ACCEPTABLE F.O.S. AND ITS DERIVATION

Both Soils Reports refer to a F.O.S. of 1.5, one "the normally accepted value of 1.5 for subdivision developments" and the other by reference to the NZ Building Code, July 1992; B1/VM4, Section 3.2 "requires a minimum value of 1.5 for permanent slopes".

Manukau City Council in their Engineering Quality Standards Section 2.8.4 slope stability C4 discuss, the traditional "theoretical factor of safety of 1.5 achieved by analysis", the nature and level of investigation and risk assessment. They conclude that a factor of safety of 1.5 is customary to adopt.

COUNCILS OBLIGATIONS

R.M.A. Section 9 Restrictions on the Use of Land.

(1) No person may use any land in a manner that contravenes a rule in a district plan or proposed district plan unless the activity is

(a) Expressly allowed by a resource consent granted by the territorial authority responsible for the plan.

Sec 106. Subdivision consent not to be granted in certain circumstances

(1) A consent authority shall not grant a subdivision consent if it considers that either -

(a) Any land in respect of which a consent is sought, or any structure on that land, is or is likely to be subject to material damage by erosion, falling debris, subsidence, slippage, or inundation from any source; or

(b) Any subsequent use that is likely to be made of the land is likely to accelerate, worsen, or result in material damage to that land, other land, or structure, by erosion, falling debris, subsidence, slippage, or inundation from any source -

unless the consent authority is satisfied that sufficient provision has been made or will be made in accordance with subsection (2).

(2) A consent authority may grant a subdivision consent if it is satisfied that the effects described in subsection (1) will be avoided, remedied, or mitigated by one or more of the following:

(a) Rules in the district plan:

(b) Conditions of a resource consent, either generally or pursuant to section 220(1)(d);

(c) Other matters, including works.

Sec.9 and 106 in 2(a) refer to rules in the District Plan.

Sec 106(a) forbids the subdivision of land "that is or is likely to be ..." and (b) Requires consideration of whether the subsequent use is likely to reduce the factor of safety.

Sec 106 2(a) - (c) Allows consent provided that the effects are avoided, remedied and mitigated.

Sec.220 1(d) A condition that provision be made to the satisfaction of the territorial authority for the protection of the land or any part thereof, or of any land not forming part of the subdivision, against erosion, subsidence, slippage, or inundation from any source (being, in the case of land not forming part of the subdivision, subsidence, slippage, erosion, or inundation arising or likely to arise as a result of the subdividing of the land the subject of the subdivision consent):

This allows the Council to make a condition that "protects" the land or adjacent land.

406. Grounds of refusal of subdivision consent -

Notwithstanding anything to the contrary in Parts VI or X, a territorial authority -

(a) Shall not grant a subdivision consent if it considers that either -

(i) The land in respect of which the subdivision is proposed is not suitable; or

(ii) The proposed subdivision would not be in the public interest.

Sec 406 gives Council grounds to refuse the subdivision if the land is "not suitable" and in reverse if the land failed due to erosion, subsidence, slippage or inundation may be the clause that claims start at.

THE DISTRICT SCHEME

General Provisions 1.3.3 Circumstances under which Dispensations & Waivers may be granted.

1. General Circumstances

(a) (iii) Slope stability or other engineering circumstances or that it is unreasonable or impractical to enforce the provision in respect of that particular site.

Section 5.2 Uses and Development Permitted in all Zones.

5.3 **Suitability of Land and Drainage**

5.3.1 **Land to be suitable**

Notwithstanding any other provision of the Scheme, no building shall be erected or placed, and no use or development commenced or Scheme Plan of Subdivision approved on any land which is not physically suitable for the use proposed.

This Concept agrees with Sec.406 R.M.A.

ENGINEERING STANDARDS

N.Z.I.E. Code

2.3 Definitions: Stable Ground - Means ground existing in a state which is "unlikely" to settle slip etc.

2.5 Site Investigations:

2.5.2 Slope stability - general only

2.5.4 Specialist Services - Professional opinion as to the suitability of the land for development with details of any special conditions that should be imposed.

2.6.3(d) Slope Stability - Where necessary (to establish a F.O.S.) a specialist soils engineering investigation should be carried out by the Soils Engineer to determine acceptable limits of cut and fill slopes.

Appendix A Statement of Professional Opinion

The NZIE code does not define an acceptable F.O.S. but discusses a point where the land is unlikely to settle, slip etc and relies on the Professional Opinion of the Soils Engineer to interpret this.

CASE LAW

In McAlpine and Auckland City Council Decision No. A71/86. July 1986 (pre R.M.A.)

Without determining an existing F.O.S. on a site both parties agreed that part of the land in the subdivision (a 2 lot boundary adjustment) had marginal stability. Likewise it was agreed that building operations or earthworks on the steeper parts of the land could, indeed probably would, result in slippage or erosion.

The decision however went on to find the "land in the subdivision is not presently subject to erosion" and "The present stable situation will continue if the land remains undisturbed. In those circumstances it is not subject to slippage or erosion within the meaning of Section 274(1)(f).

Section 274(1)(f) is superseded by Sec 106 R.M.A. and the definition has changed from "is subject to" only, to "is or is likely" and in the second part the acceleration or worsening "due to subdivision" in 274(1)(f) has changed to "Any subsequent use that is likely to be made of the land is likely to accelerate ..."

The change of emphasis from "is" to "is or is likely" is central to the issue of a F.O.S. A part of the above decision was the applicant accepted that if the subdivision proceeded it was appropriate to impose conditions or controls to prevent disturbance (of the existing "stable" condition. In this case that would mean requiring Specific Design in the area of the lot affected and perhaps imposing maximum cut and fill requirements on the title of the property.

DISCUSSIONS

From these references it would seem that it is necessary for the Soils Engineer to present to the T.L.A. that it is unlikely that the land in question will suffer from subsidence, slippage or erosion.

Manukau City in their Engineering Quality Standards

2.3.4 Slope Stability C4

Discussion on Factors of Safety shows

Factor of Safety	Risk of Failure per annum
1.1	1:10
1.3	1:50
1.5	1:200
1.7	1:1000

This may be viewed in terms of Risk Quantification (IPENZ, Engineering Risk, Report of the Presidents Task Committee on Professional Practice & Risk 1983 page 19) where

A factor of safety of 1.5 equates to the failure each year of 1 slope in 200 slopes.

MANAGEMENT OF RISK

T.L.A.'s have open to them

1. Sec 106 (RMA) 1 (a) } refuse the application
   (b)

2. Sec 106 (RMA) 2 (a) } allow subject to conditions, other matters and/or works
   (b) as provided in 220(1)(d).

3. Sec 641(L.G.A.) - allows the T.L.A. to issue a permit for a building on land likely to be subject to damage arising from erosion, subsidence or slippage etc. (Now Sec 36 Building Act)

A question then is where does the line fall between likely and unlikely and can the risk be protected against sufficiently by works.

Another question is Sec 406 (ii) "The proposed subdivision would not be in the Public interest", when could this apply?

Perhaps subdivision of land is not in the Public Interest when it exposes Council and consequently the ratepayer to an unacceptable level of risk of incurring financial losses in the event of failure.

PRACTITIONERS POINT OF VIEW

Discussion with a Soils Engineer of a conservative firm centred on the requirement of some T.L.A.'s to examine the case of surface saturated soils when determining the benchmark F.O.S. In his opinion this was unlikely to occur in many slopes and especially in Auckland City clays, further he proposed that it was incumbent on the Soils Engineer to estimate the likely range of soil water table and report a normal to worst case regime for the slope in question. The normal case should be the one to be above the benchmark 1.5 with the sensitivity to saturation determining the drainage or other works proposed.

THE DEVELOPER'S POINT OF VIEW

Land within a subdivision will sell at an optimum size/price ratio, when this is decided the thrust is to produce as many lots as possible at this size. Lots left large to incorporate unstable land will generally not sell at a sufficient premium to offset the potential gain if they are earthworked and serviced.

When the land is sold the developer has the professional's opinion, to rely on and the T.L.A.'s acceptance of the lots and services within them gives Councils a vested interest.

SUMMARY

Decision-makers are required or entitled to make decisions. Engineers should not usurp that right, particularly if they wish the decision-maker to accept responsibility for the ultimate success or failure of the project. (Engineering Risk).

It would appear that a F.O.S. of 1.5 is an industry accepted standard with no legislative precedent, rather than alter this and by doing so take responsibility on, I recommend that 1.5 remain the basis of acceptance.

The only further point is under what circumstances should 1.5 apply?, I suggest this is best left to the judgement of the Engineer proposing the project. However in order that Council know it is exercising care in accepting the design the Engineer should be required to show the sensitivity of his decision to variations, of the parameters used.

POLICY

(i) The normally accepted F.O.S. of 1.5 is as high a risk as Council will accept for land deemed suitable for subdivision. This F.O.S. shall be as reported by the Soils Engineer however he shall show to the satisfaction of the Council the sensitivity of the slope, to variations of the parameters used.

(ii) This sensitivity shall direct the Soils Engineer to identify certain parameters, by physical testing, if they are critical. Where potential exists for the factor of safety to be reduced below 1.5, remedial or protection measures shall be put in place to ensure a minimum factor of safety of 1.5 is maintained.

(iii) For an extreme event such as when considering earthquake loads with the above stability considerations, or failure of stabilisation measures used to reduce ground water levels occurs, then a reduced factor of safety (say 1.2) against instability is satisfactory.

Where a F.O.S. of 1.5 is not available on any lot Council may allow a Consent Notice to be placed on the title that requires specific design of foundations of buildings, earthworks and stormwater control by a Registered Engineer experienced in Geomechanics. A foundation design for a typical lightweight flexibly clad dwelling shall be submitted to show that Building Construction on the lot is possible with a F.O.S. of 1.5 against failure by subsidence or slipping. of the building itself.

This discretion will only be allowed where Council is satisfied that the consequences of failure are acceptable e.g. no other property owner, Council services and access to properties are affected. A Building consent issued under these circumstances would be subject to Sec. 36 of the Building Act.

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Waitakere City Council

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Dear Mr

ENGINEERING GEOLOGISTS - LAND STABILITY REPORTS

As per our discussion today, Council's position regarding Engineering Geologists and Land Stability Reports is as follows.

1. Council has a duty to ensure that reasonable care is taken with any information received and/or accepted.

2. The role of the Engineering Geologist could be seen as ensuring that geological factors affecting the planning, design, construction, operation and maintenance of engineering structures and the development of groundwater resources are recognised, adequately interpreted and presented for use in engineering practice. Thus the Engineering Geologist presents information to be used as a basis for the subsequent design of or stability report for structures.

3. From discussions with Rodney County and Takapuna City it would appear that the Engineering Geologist is not accepted by them as a Geotechnical Engineer. In fact Engineering Geologists do not appear on the list of Geotechnical Engineers recognised by Rodney County as being capable of providing satisfactory geotechnical reports.

   On the other hand it is recognised that Engineering Geologists can produce satisfactory information on which these reports can be based.

4. The Waitakere position is similar to that of Rodney County. We will be prepared to receive Engineering Geological reports from Engineering Geologists but will not necessarily be prepared to accept them.

5. Any work in excess of NZS 3604 which will require specific design will be required to be signed by a Registered Engineer who certifies that he/she is experienced in the field of soils engineering.

6. Stability reports require certification on completion to be signed by a registered engineer experienced in soils engineering.

Yours faithfully

Riddolls & Grocott Ltd

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New Zealand • Tel/Fax: + 64 3 337 3226

Our Ref: 26 October 1994

Your Ref:

The Editor
NZ Geomechanics News
PO Box 4241
Auckland

Sir

In response to your invitation (NZ Geomechanics News, July 1994) to debate "a sensible approach to slope stability as it affects land development", one can be found in the Building Research Association of New Zealand (BRANZ) Report SR4 (Reference 1). This publication summarises the state of art for the geotechnical assessment of land development sites involving slope stability for New Zealand conditions, and is a must on the bookshelf and in the briefcase of every practioner and territorial authority subdivision officer. It builds on two other publications which provide general guidance on slope stability matters relevant to land development for New Zealand conditions (References 2 and 3).

BRANZ report SR4 provides sound commentary on acceptability of factor of safety for land development and the merits of an engineering geological versus a numerical approach to the assessment of slope stability:

1. Acceptability of Factor of Safety

There is always uncertainty attached to all numerical analysis of the stability of slopes, to the extent that it is doubtful whether such techniques truly represent the actual stability of a slope. This uncertainty arises for a variety reasons including:

i) different methods of analysis yield different factors of safety for the same input data

(ii) paucity and validity of input data including choice of relevant shear strength criteria and groundwater conditions, and

(iii) inability to take into account the time factor with respect to possible changes in slope conditions particularly groundwater effects.

All practioners have suffered the problem of deriving safety factors of less than unity for slopes which observation tells us are stable. Conversely, some have suffered the embarrassment of failures of slopes which according to analysis should have been stable. All of this leads to the possibility for manipulation of the input data to derive the "right" result for subdivision compliance purposes.

The use of a minimum safety factor such as 1.5 to comply with Building Code requirements is therefore strongly to be discouraged and in fact has little relevance other than as a guide as to the stability of the slope at the time the analysis was carried out.

Of much greater relevance is the use of numerical techniques to evaluate the sensitivity of a slope to changing ground conditions including groundwater and earthworks effects, and external loads. Local authority officers involved with subdivision permits would be doing themselves and geotechnical practioners a much greater service by requesting that the sensitivity of a slope to say changing groundwater effects be considered. For example, commonly it is short duration high intensity rainfall events resulting in short term but rapid piezometric response which are the controls on slope stability. Such sensitivity studies are a particularly useful tool for identifying critical slope variables and in the design of remediation measures. They therefore have much greater relevance than compliance with any minimum factor of safety.

2. Engineering Geological Versus Analytical Techniques

The engineering geological (or observational) approach seeks to develop a geological and groundwater model for the slope from the nature and distribution of the materials as established by site investigation. Further, the behaviour and characterisation of local slopes adjacent to the site under investigation is used to evaluate potential failure mechanisms and to assess the hazard or consequences of any failure. The outcome of such an approach is commonly a relative risk assessment where two or more hierarchies of risk of slope instability are recognised.

The engineering geological approach is therefore based on the use of observation, judgement and experience in defining the most likely form of instability and assessing the likelihood of that instability occurring. The engineering geological approach is unquestionably valid as a method of slope stability assessment, but the conclusions reached are only qualitative in their evaluation of risk. However, for many sites this approach is the only valid technique that can be applied, as hydrological and geological conditions are too complex for meaningful numerical analysis.

Analytical techniques on the other hand merely allow the relative stability of a slope to be expressed quantitatively.

We do not view the two techniques as opposing or competitive, but rather complementary, whereby the relative stability of a slope may be expressed in different ways. We sincerely hope that there are no suggestions that only one of either of the two methods should be exclusively used for slope stability assessments.

Guy Grocott
Riddolls & Grocott Ltd.

Bruce Riddolls

References

1. Worley Consultants Ltd., 1987. Slope stability assessment at building sites. Building Research Association of New Zealand. BRANZ Study Report SR4.

2. Taylor, D.K., Hawley, J.G., Riddolls, B.W., 1977. Slope stability in urban development (DSIR Information Series No. 122), DSIR, Wellington, 71p.

3. New Zealand Geomechanics Society, 1980. Stability of house sites and foundations: Advice to prospective house and section owners. Published for The Earthquake and War Damage Commission.

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25 November 1994

Mr Geoff Farquhar
Editor Geomechanics News
c/- Worley Consultants Ltd
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AUCKLAND

Fax: (09) 379 1210

Dear Geoff

re: SLOPE STABILITY QUESTIONNAIRE

Attached please find the completed questionnaire, together with the following additional comments:-

1. Defining Slope Stability: Instability may include falls, slides or flows, or combinations of these ground failure mechanisms. To apply a single factor of safety approach to all of these situations is in my view totally inappropriate, as it is really only slides that lend themselves to this kind of quantitative analysis. As noted in my reply, debris flows accompanying storm events are in my view a most serious stability hazard because of their speed and mobility. Similarly rockfalls are potentially very dangerous for the same reasons, and in either case I must question whether these are readily amenable to analysis of the sort envisaged by the BIA. My own experience is that the majority of building sites on natural slopes can be realistically assessed without quantitative analysis, although where significant excavation or filling takes place a very different set of parameters applies and here close control of earthworks is indicated.

2. Approach to Residential Development: I have long advocated an engineering geological approach as the first stage in evaluating a development proposal, and have published extensively on this subject as well as working as a consultant in the field. The method involves hazard identification, evaluation of foundation conditions, and testing of materials if or where appropriate. Considerable reliance is placed on geological analysis of slope or site evolution, and extensive use made of air photos to obtain data to supplement field information. In such evaluations slope stability is only one of the hazards considered, and I find your questionnaire a little too focused on this one aspect because all are important in assessing site suitability. One of the prime aims of the engineering geology assessment is to identify those parts of the development where specific design is needed, those where no houses should be allowed, and those that can be safely built on using precedent and the various earthworks codes. I find that many South Island councils accept this (or a similar) approach, and in fact a number specify geological (rather than engineering) assessment, at least as a first step.

3. Professional Recognition: One of the concerns in the past has been the fact that engineering geologists cannot be registered under the Engineers Registration Act, and there has often been a tendency to specify reporting by a registered engineer. My experience suggests, however, that many professional engineers who are not geotechnical specialists cannot deal adequately with slope instability problems without additional assistance, and this is frequently provided by engineering geologists. The current proposal whereby engineering geologists can become full members of IPENZ should in my view eliminate much of this concern, as reliance on a code of ethics that demands professional competence will achieve as much (if not more) than registration.

4. FS = 1.5 Approach: I am quite amazed at the BIA approach which baldly specifies a factor of safety of 1.5. Apart from the fact that many slopes in this country probably have FS values much less than 1.5 (eg the Lake Dunstan landslide stabilisation project), there are significant differences in the input parameters for soil and rock stability analysis and any requirement to meet such a conservative stability factor presupposes an unlimited budget for investigations and remedial measures. It is clearly necessary to specify whether the worst case scenario should be assumed in this analysis, or some lesser set of parameters. My own view is that the worst case conditions obtaining within a reasonable design life (say, 100 years) should be considered, but only with a requirement for FS > 1.0 in such a situation. On the other hand I would not object to engineered structures, such as retaining walls, being designed to around 1.3 although whether this should be with design earthquake loads is another issue. Given that Pukaki High Dam has FS = 1.3 under 0.3g, and the MCE is likley to produce around 0.5g, there is something very wrong if all slopes in urban residential development have to be designed to FS > 1.5. This is especially so if development can be permitted under the Building Act on land subject to hazards, and I have in the past and will in the future make such recommendations in terms of s36(2) for certain landslide areas.

There are many more matters that I could raise on this subject, but I will leave them to see what comes from the next isue of Geomechanics News.

Yours sincerely

DAVID H BELL
Senior Lecturer in Engineering Geology

Fax - Manukau Environment: 262-5163
Taiao O Manukau

8 November 1994

New Zealand Chairmechanics Society
The Editor - Geomechanics News
Mr G Farquhar
C/- Worley Consultants Ltd
PO Box 4241
AUCKLAND

Dear Sir

SLOPE STABILITY IN LAND DEVELOPMENT

I am in receipt of your letter dated 2 November 1994 regarding our approach to the requirement for geotechnical assessments for land development, Section 36 (2) of the Building Act 1991 etc. Firstly in response to the direct questions raised in your questionnaire I will comment as follows:

1. Manukau City's longstanding policy is to require a report which has been prepared under the supervision and signed by a registered engineer experienced in geomechanics. While the content of the report in relation to the physical ground conditions is assessed as to its appropriateness the Council relies on the professional judgement of the engineer undertaking the works as to their ability in terms of the scope of the particular site. Council officers base their judgement on the content of the report, the methodology used and the conclusions arrived at in terms of the topography nature and history of the site under investigation. Where we feel a registered engineer has strayed beyond his professional capability we will generally recommend he seek advice from others in the profession to assist him.

2. The geotechnical assessment varies considerably depending on the nature of the site, the scope of the work and the type of development. For example in rural areas the Council still requires that a geotechnical assessment of the suitability of the land be made (acting under its obligations in terms of Section 106 of the Resource Management Act in relation to subdivision). In the case of rural subdivisions covering a large area of land the Council favours a broad brush geotechnical assessment approach to identify those parts of the site which are suitable for normal house construction not requiring specific design, those parts of the site in which due to the steepness of the topography for the presence of surface features or historic information will require specific investigation and design at time of building consent and those parts of the site which would not be suitable for building in terms of stability or inundation.

.../2

L/10
New Zealand Chairmechanics Society
8 November 1994

3. From our experience numerical stability analysis has rarely been used to identify suitable building sites or assess the risk of instability. In most instances the engineer assessing has drawn on likely soil strengths from similar materials and undertaking an assessment of the risk of failure. When assessing particularly cliff top beach frontage properties stability of the immediate slope is only one factor to be considered, other adjustments as to the likely rate of regression of the shore line and any works that may need to be carried out in order to mitigate the effects form a major part of the geotechnical assessment.

4. In critical cases in which an analysis is required the Council accepts an approach of analysing the existing slope under its existing ground water and soil conditions using average values. Under these conditions a safety factor of 1.5 should be established to satisfy the Council in terms of Section 106 of the Resource Management Act and Section 36 of the Building Act.

5. As previously stated it is general accepted practice for the Council to accept indicative soil strengths from similar soils rather than specifically test soils on site. Should the analysis demonstrate that this particular slope may be critical then it is anticipated that the investigation may need to go a second stage and undertake specific soil sampling, strength tests and a rigorous analysis to satisfy the Council.

6 & 7 The Council has generally not required assessment of slopes under earthquake conditions.

8. Under extreme conditions of elevated groundwater a safety factor less than 1.5 is acceptable. It is generally required that the engineer undertake a worst case analysis to satisfy himself that the slope will still remain stable with a safety factor in excess of 1, generally around 1.2. As a note it should be remembered that a safety factor of 1.5 does not guarantee that the slope has 1½ times protection against failure.

9. Generally the Council does not rely on the provision of drainage works as the sole means of ensuring slope stability. Often drainage works are put in but in most cases as a protective measure to improve the current situation rather than a primary means of providing protection. In all cases provision is made to identify an protect the drainage system and its outfall, this would now be done in terms of consent notices for the case of subdivisions.

10. In the cases where Council officers hold a contrary view to that of the geotechnical engineer supplying the report, and the issues cannot be resolved, the Council with the permission of the applicant will obtain a peer review from an independent geotechnical engineer.

11. Section 36 of the Building Act is currently applied within the Manukau area under the following basis. If the land on which the building is sited, that is the footprint of the building, is likely to be subject to inundation or slippage the Council will require the issuing of a certificate in terms of Section 36 of the Act. If works are undertaken to make the land free of inundation and safe from slippage then Section 36 certificate would not apply. In all cases the other provisions of Section 36 regarding effects to other property etc and the protection of the house from flooding would have to have been satisfied.

12. In reviewing the Manukau City Council's engineering standards a joint working party of Council officers and practitioners within the Manukau City area was formed to prepare the geotechnical requirements of the new engineering quality standards. I attach a copy of that for your information and use. You will note that great emphasis is placed on the methodology and the logic of assessing the geotechnical aspects of a site with considerable reliance placed on the experience and judgement of the engineer providing the assessment.

13. The territorial authority is the Manukau City Council and this letter has been prepared by the Council's Development Engineer, Mr John White.

If there are any other matters that I may assist the Society on I would be pleased to be involved, and I enclose my card which has various phone and fax numbers at which I may be contacted.

Yours faithfully

J White
Development Engineer
MANUKAU ENVIRONMENT

Encl

JW:lt/133

2. GEOTECHNICAL

2A MINIMUM REQUIREMENTS

2.A.1 Every allotment shall contain a building site suitable for building types appropriate to the zoning of the land.

2.A.2 All resource consents for subdivision and multiple household units, shall be accompanied by a geotechnical investigation report relevant to the site.

2.A.3 An allotment which has specific geotechnical requirements may be approved subject to restrictions which shall be registered on the title to the land by a consent notice, security documents if required by the RMA, and the Council Property Information Register as appropriate.

2.A.4 For every subdivision for which a Resource Consent has been granted a geotechnical completion report shall be submitted in respect of every allotment/ building site whether affected by work or not.

2.A.5 All geotechnical investigation & completion reports shall be prepared by a Professional Engineer who is experienced in the practice of geotechnical Engineering and registered under the Engineers Registration Act 1924 and who has professional indemnity insurance cover.

2B. Means of Compliance

2.B.1. Introduction

2.B.1.1 This document provides a means of compliance with the Manukau City Council Engineering Quality Standards, Section 2, GEOTECHNICAL

2.B.1.2 The document is presented in a manner which outlines an acceptable methodology for achieving compliance with Council's standards. It is presented with matching commentary where considered appropriate. The commentary is provided so as to provide a means of clarification for practitioners, and for Council officers who are required to interpret Council's standard requirements, and the data submitted to Council in compliance with those standards.

2.B.1.3 This document has been developed on the basis of consensus between a Working Party of Council officers and Practitioners.

C1

One of the problems which has affected the orderly assessment of the suitability of land for development and building foundations has been a lack of uniformity of understanding between practitioners in their approach to the issues, and Council officers in their interpretation of these approaches.

It is recognised and accepted that no two engineers will approach any one problem in exactly the same way, nor will they reach entirely the same conclusion, nor present their data and interpretations in exactly the same way.

Often the comparative lack of specialised knowledge or experience acts to hinder the rationalisation of these issues, usually at considerable cost to all parties involved, including the practitioner and the Council.

This Means of Compliance document provides a quality management means of overcoming these issues of communication and understanding within the framework of a generalised set of expectations.

2.B.2. Phases of a Site Investigation

2.B.2.1 A rational approach to site investigations for urban land subdivision is given in NZS 4404:1981, New Zealand Standard Code of Practice for Urban Land Subdivision, and the IPENZ (Auckland Branch) Code of Practice for Urban Land Development (c.1979), from which NZS 4404 was developed.

2.B.2.2 Drawing G1 presents the relationship of site investigation to all forms of civil engineering works, and particularly to large scale land development. The principle is as follows:

C2

In terms of a quality management approach to the satisfaction of engineering criteria it is necessary for a geotechnical engineer to consider his approach to an investigation with respect to:-

(i) The end use of the site.

(ii) The liability consequences in the event of a wrong appreciation to:-

(a) The client and subsequent owners

(b) The engineer

(c) The Local Authority

The reality is that the backstop in terms of liability is:-

(a) The engineer's own professional indemnity insurance.

(b) The Local Authority's insurance.

Neither the insurers, the client, nor the Local Authority will thank you for making a wrong appreciation.

(i) Preliminary Exploration

A preliminary exploration is necessary to gain an initial appreciation of a site. In the case of land development, this is usually simply a visual appraisal. In some cases a visual appraisal is all that is necessary.

(ii) Field Investigation

A preliminary exploration will in most land development projects, be followed by a specific and detailed field investigation , with in situ and laboratory tests as appropriate.

This investigation will yield the basis of the detailed geotechnical appreciation of the site, and will provide guidelines for the development of the site, including limitations on that development. Often as not, detailed design is likely to be progressing concurrent with reporting and must be accommodated.

(iii) Construction Observation

Observation during construction is essential to verify the appreciation obtained from (i) and (ii). It is not unusual for the appreciation to be modified as site development proceeds, resulting in further investigation and changes in design and/or construction concepts.

(iv) Performance Observation

Performance observation may be necessary where the earthworks design predicts post construction performance, and which is critical to the ultimate land usage - for example the monitoring of the settlement of deep fills and fills underlain by highly compressible organic soils.

2.B.2.3 In summary, investigation in one form or another should be continued throughout construction, and may extend beyond the construction phase.

A quality management approach then says that the process of site investigation is not complete until the construction aspects of the project are complete.

An in-complete or less than thorough approach to a site investigation is a waste of effort, inconclusive and fraught with risk.

The principles relating to geotechnical investigation for land development/subdivision purposes are the same, irrespective of whether an intensive urban subdivision or a rural subdivision is being considered. What differs is the extent of the process that needs to be followed. Often the methodology can be abbreviated if site conditions are appropriate. The important element is the provision for hazard assessment and risk mitigation.

2.B.3. Site Investigation Logic

2.B.3.1 General

C3

All sites will not require the application of the Means of Compliance to the same degree. For example, a flat site with no obvious risks or identifiable hazards may simply require a short letter stating that the site has been inspected by a competent person and an appropriate judgment appreciation made. This assessment should be reported in letter form to ensure continuity of record of the Quality Assurance audit trail.

The objectives of a site investigation are to:-

(i) Assess the suitability of a site for its proposed use.
(ii) Foresee construction difficulties.
(iii) Collect enough information for satisfactory design

A site investigation should address the following factors:

(a) Ownership

(b) Geology)
) Nature and
(c) Groundwater) extent of soils
) and rocks, etc.
(d) Subsoil conditions, foundation)
conditions and stability)

(e)Services

(f)Access

A review of the existing information is essential, and which may include:-

1.Geological maps and reports.

2.Data from adjacent sites - i.e. previous investigations.

3.Aerial photographs.

4.Performance of related and/or adjacent developments.

The logic of site investigations for land subdivision and development projects is summarised in Drawing G2

2.B.3.2 Geotechnical Investigation Report

Geotechnical investigation reports should generally cover, but not necessarily be restricted to, the following:

(a) Purpose

To investigate, examine and report on the general geology of the area and its influences on foundation conditions and comment on the likelihood of any problems with the stability of slopes in cut, natural or filled ground.

(b) Soils

Investigate and report on the soil characteristics of the site with regard to foundation and construction conditions.

(c) Foundation Requirements

Consider the types of buildings likely and their load requirements, and evaluate the foundations for each allotment. Consider the type of road and evaluate the foundation at subgrade level.

(d) Effluent Disposal

In areas where sewage disposal is by means of septic tanks, the report should also comment on the suitability of the site to accept septic tank effluent disposal and its influence on land stability.

(e) Non Engineered Fills

Identify the existence of previous filling activities on the site, and comment on the quality and suitability of such fills for development purposes.

(f) Slope Stability

Where appropriate, carry out a slope stability appraisal to determine whether the development will provide stable and accessible building sites.

(g) Earthworks Development and Control

Discuss earthworks aspects of the site and provide a specification for earthworks control and the installation of services.

(h) Conclusions and Recommendations

Set out the findings of the investigation and provide recommendations for:-

(i) Restriction on use of the land if all or part of the land is unsuitable for some uses.

(ii) Suggested changes to a subdivisional layout to achieve better use of the site, and/or minimise construction difficulties.

(iii) Control during construction.

(iv) Further investigation where required.

(v) Regulation and control, or future action necessary to maintain suitability.

A check list for reporting is given in Drawing G3

2.B.4. SLOPE STABILITY

2.B.4.1. A primary objective of land development engineering is to provide stable and accessible building sites. These considerations are important, particularly in the case of residential land development, which is proceeding more and more into areas of marginal stability, previously avoided in past years.

2.B.4.2. A principal factor of a site investigation is to identify indications of past instability and to determine and recommend mitigating measures to provide an acceptable level of risk against future slope failure.

C4

Most landslips are caused by an increase in the slope angle of the land surface and/or a decrease in the shear strength of the slope materials. A large number of interrelated factors do, however apply.

Flowing water from rainfall is constantly changing the shape of the earth's surface. The water cuts out a channel and the side slopes of the resulting channel or valley are left oversteep and subject to landslip (i.e. are marginally stable).

These side slopes gradually become less steep as the topography matures, and erosion and landslipping becomes less frequent. Steeper parts of the valley system may, however, remain in a state of only marginal stability and some incident, such as excavation or exceptionally heavy rain, can cause further landslipping to occur.

It follows then that many slopes are potentially unstable in their natural state, and must therefore be considered unsuitable for residential development unless improved by properly designed engineering works.

The filling of gullies and flattening of hill slopes in subdivisional development converts the young and potentially unstable slopes found in many parts of New Zealand to a more mature and stable land form, thus achieving in a short term what nature would otherwise

2.B.4.3. An experienced person can recognise previous landslip areas by stereoscope examination of pairs of aerial photographs. This is a fast and economical means for assessing the general slope stability characteristics of a large area.

2.B.4.4. Visual signs of ground instability include cracked or hummocky surfaces, crescent shaped depressions, crooked fences, leaning trees or power poles, swamps or wet ground in elevated positions, and water-seepage.

2.B.4.5. The evaluation of slope stability by the measurement of soil strengths and groundwater conditions, and the calculation of theoretical factors of safety, is a difficult task and which requires the exercise of a large measure of skill and judgment. The problem is exacerbated by the need to consider:-

(1) The range of parameters assumed to be applicable, given the present state of stability of a slope.

(2) Present and future groundwater levels.

(3) The consequences of and limitations on future site development.

2.B.4.6. Preliminary reconnaissance using site inspection, air photo interpretation and available geological and geotechnical records provides an initially low cost sound basis for the conceptual planning of a land development project. This preliminary appraisal should identify:-

(a) Areas where previous slope failures have been positively identified.

(b) Areas where it is suspected that slope failures may have occurred many years ago (i.e. historic features).

(c) Areas of surface soil creep.

(d) Springs, swamps, or other areas of either poor drainage or high groundwater conditions.

achieve through landslips and erosion in many thousands of years.
In the situation where the concept of land development is to minimise earthworks and to leave the land in its largely undisturbed natural state, many problems arise and are experienced between the practitioner and local authorities in determining acceptable slope stability criteria, levels of risk and reporting.

Traditionally, if a theoretical factor of safety of 1.5 can be achieved by analysis, then the slope is deemed to be stable. The problem arises in determining the correct parameters to use and the influence of subsurface conditions on the form of analysis, and which is consequently dependent on the nature and level of investigation.

Cumulating experience suggests that the proper selection of a theoretical factor of safety for slope stability purposes is dependent upon a proper assessment of the level of risk.

Brand (1982) on analysis and design in residual soils, reports the established practice in Hong Kong (Geotechnical Control Office, 1979) in which the design factor of safety is related to the risk category. The risk category for a particular slope is assessed in terms of the likelihood of loss of life should the slope fail. Typical of high risk slopes are high cut slopes immediately adjacent to schools and occupied apartment blocks. An example of a low risk slope is one which threatens only a secondary road.

Typical values of acceptable factors of safety in residual soils are given in Table C1. The design "standard" of slope safety (i.e. the probability of failure) is (logically) governed largely by the consequences of failure in terms of loss of life, damage to property, and disruption of communications and services.

TABLE C1: ACCEPTABLE FACTORS OFSAFETY FOR SLOPE IN RESIDUAL SOILS (Based on Brand, 1982)

Risk Category	*Minimum Factor of Safety for Transient Condition (e.g. 1:10 Year Storm)*
Low	*1.2*
Significant	*1.3*
High	*1.4*

It is noted that factors of safety adopted by engineers in geotechnical design have been developed to cover uncertainties in:-

** the geometric accuracy (e.g. of the slope or retaining wall being designed);*

** the soils strength (which is likely to vary from point to point even in the same soil "layer");*

** the method of analysis adopted (which is usually two dimensional and have simplifications that may not accurately reflect the actual situation);*

2.B.4.7. Subsequent specific investigations should provide data on subsurface conditions and establish specific design criteria for such factors as maximum slopes, subsoil drainage, retention or establishment of vegetation, soakage to dispose of septic tank effluent, and the like.

2.B.4.8. An acceptable concept for the investigation and assessment of the suitability of land developments in which natural slopes are intended to be left undisturbed provides for the delineation of building limits as follows:

(i) A **Building Line Restriction** which represents the closest proximity to a slope for any building development.

(ii) A zone, defined as the **Specific Design Zone**, extending from the Building Line Restriction to the Building Line Limitation , within which building development requires specific design by a registered engineer either experienced in soil mechanics, or with the assistance of an engineer experienced in soil mechanics, and in particular, slope stability.

(iii) A zone (the **Non Specific Design Zone**), deline ated by the Building Line Limitation, extending beyond the Specific Design Zone in which build ing development can be carried out in accord ance with the appropriate Codes of Practice (e.g. NZS 3604:1990, New Zealand Standard Code of Practice for Light Timber Frame Buildings not requiring specific design) without risk from slope instability.

2.B.4.9 The determination of the zones should be derived from an assessment of potential risk under varying site conditions. The start of the Non Specific Design zone, defined by the Building Line Limitation, would typically commence 3 m to 5 m (or possibly more) beyond the Building Line Restriction, and would assume a factor of safety against slope regression of 1.5 or greater. The respective zones are to be defined by suitable diagram which can be used to establish relevant ground control points.

** the validity of assumptions made (e.g. depth to groundwater level, depth to rock or hard layer, etc.).*

For these reasons, it is customary to adopt a factor of safety value of 1.5 for subdivisions or housing development. This factor of safety does not in every case assure safety from instability or slope movement. Based on references (1 to 3) noted below, the average risk of failure for different factors of safety adopted are:-

Factor of Safety	*Risk of Failure Per Annum*
1.1	*1:10*
1.3	*1:50*
1.5	*1:200*
1.7	*1:1000*

References:

1 *.Meyerhof, G., Canadian Geotechniques. Vol.7, No.4 (11/70)*

2. *Wu, T.H., et al, ASCE, SM2 (3/70)*

3. *Semple, R.M., Ground Engineering (9/81)*

C5

In almost all circumstances, it will probably prove to be prudent for the developer/engineer to at least obtain an initial appraisal by a geotechnical engineer to provide input to the conceptual design.

In many circumstances (e.g. where the size of the development is sufficiently small), a visual appraisal will suffice. In most circumstances, however, some form of investigation will be required.

2.B.5. Earthworks

2.B.5.1 Earthworks can be loosely defined as any alteration to the ground by means of excavation and/or backfilling.

2.B.5.2. The engineer/investigator should, when initially considering a land development project, address at least the following, to provide a preliminary assessment of potential difficulties or the need for specialist advice:-

(a) The present topography and any surface features such as hummocky ground, irregular land forms, rushes, and obvious geological features which might infer past or present instability.

(b) Any exposures of soil types, which might indicate potential difficulties of construction, i.e. sands, clay, rock.

(c) Existing drainage conditions, and their relation ship to the proposed development.

(d) The performance of similar engineering works (cut and fills) in adjacent areas.

2.B.5.3. In summary, the proper appreciation of the earthworks concepts for land development will identify:-

(i) The suitability of the site for the concept, including the appraisal of aerial photo for larger sites and records of previous filling.

(ii) Particular engineering measures that will need to be incorporated in the engineer ing design (e.g. tomos).

(iii) The influence of the earthworks concepts on slope stability and mitigating design measures.

(iv) Special measures that might be required for settlement considerations, depending on fill depths, etc. (i.e. settlement monitoring, delays on building construction, etc.).

(v) Control measures for earthworks.

2.B.6. Earthworks Quality Assurance and control

2.B.6.1 The quality control of earthworks is an essential phase of land development, and is aimed at providing a uniform construction in terms of engineering performance. Earthworks should be certified as to the way in which they have been carried out and their suitability for their end use. The form of quality control will evolve from the earthworks appreciation and will generally be developed about:-

(i) Adequate strength.

(ii) Limited volume change.

2.B.6.2 The engineering performance of soils depends on their condition at the time of compaction and cannot be adequately reflected in a single parameter.

2.B.6.3. A considerable amount of judgment is involved in the determination of quality control criteria.

2.B.6.4 Quality control should be undertaken either by, or under the direct supervision of an experienced geotechnical engineer and should involve:-

(a) Visual inspection.

(b) Quantitative testing.

C6

It is recommended that a full quality assurance system be developed to ensure that the end product, i.e. the completed house lot is suitable for its end purpose.

In addition to following guidelines, set out in national publications (such as NZS 4431:1989, New Zealand Standard Code of Practice for Earthfills for Residential Subdivisions), it is recommended that progressive testing be carried out to avoid rework and to avoid unsuspected poor quality fill.

The fill needs to have sufficient checks (quality assurance procedure) at progressive stages of the works, i.e:-

** on completion of clearing and removal of unsuitable soils:*
** on completion of compaction of each fill layer until completion of the whole fill;*
** all with clearly dated and surveyed test points.*

Corrective measures need to be specified and carried out where the target quality is not met.

The final fill control certificate provides a record of work done and tests carried out (quality control record).

Visual inspections should be made:-

(a) After stripping and prior to filling.

(b) During installation of drains.

(c) Suffiently often to check that:-

(i) Fill is not placed over soft or organic ground, unless provided for by design.

(ii) Seepages and potential seepage areas are provided with drains.

(iii) Unsuitable materials are not used as fill.

(iv) Compaction operations are systematic and uniform.

(v) Conditions encountered are in keeping with those anticipated from the initial site investigation.

Quantitative testing should be related to the control criteria determined by the soils engineer and should have a higher frequency (say about 500 to 1000 m3 intervals) at the initial stages of earthworks to sort the operations out, with a lesser frequency as the fill progresses, the compaction criteria are being achieved, and visual appraisal indicates the overall operations are satisfactory.

2.B.7. SETTLEMENT

2.B.7.1 Settlement of soils (consolidation) is a complicated natural phenomenon, which is influenced by a number of factors, including the nature and mineralogy of the soil, the soil particle arrangement, whether the soil is undisturbed or remoulded, its past stress history, the drainage conditions affecting the particular circumstances, etc.

C7

Because of the variables in the theoretical appreciation of the likely magnitudes of settlement, monitoring of the actual settlement performance of earth fills can often expedite the release of the development.

The absolute magnitude of settlement, except when it is large, is often of lesser concern for most forms of construction than the magnitude of differential settlement and thus angular distortion. It is often overlooked, that conventional analytical processes generally imply settlements of the order of approximately 25 mm.

2.B.7.2. For land development works, the predevelopment soils investigations should identify areas of risk, such as organic soils, swampy areas, etc., and the likely performance of the foundation under earth fills.

It is also often overlooked that seasonal moisture variations, and the associated swelling and shrinking of the soil, will occur with the associated likelihood of seasonal swell/shrink movements of the order of at least 10 mm. These natural effects may cause distortion of building frames, and may cause doors and windows to jam. In many instances, and as more difficult country is developed, earth filling of narrow gullies is occurring with considerable variations in fill thickness over short distances, and thus with the potential for differential settlement. .

2.B.7.3. Settlement will also occur within earth fills due to the self weight of the fill.

Consequently, the limits of earth filling and the variations in the depths of fills should be clearly identified on "as built" drawings.

2.B.7.4. The consolidation settlement and elastic compression of fill are a function of time, albeit of long or short duration, thus in some cases it may be necessary to allow a period of time to elapse from the placement of fill to the commencement of building construction.

The expansive nature of many of the soils within the greater Manukau area needs to be recognised also, as generally provided for in NZS 3604:1990, New Zealand Standard Code of Practice for Light Timber Frame Buildings not requiring specific design.

2.B.8. Subgrade Design

2.B.8.1. The general principles of earthworks investigation, construction and control as outlined in this Means of Compliance also apply to road subgrades.

C8

The geotechnical report should address any specific design criteria perceived necessary for subgrade preparation (e.g. stabilisation, recompaction, geotextile, drainage, etc.).

2.B.8.2. For subgrades in fill materials, it is usual to attempt to achieve a higher strength fill within the upper 500 mm to subgrade formation level.

2.B.8.3. Two principal methods of pavement design are currently used:-

(i) CBR method.

(ii) Deflection method.

2.B.8.4. The CBR method is most commonly used and is based on laboratory or in situ tests and is dependent upon approval of the subgrade for uniformity and standard of construction. Uniformity may be assessed by proof rolling or in situ tests such as the dynamic cone (Scala) penetrometer.

2.B.8.5. The deflection method is based on end of service empirical performance criteria and requires the in situ testing of the subgrade by Benkelman beam prior to placing the metal courses.

2.B.8.6. In general, cut ground may require more treatment for acceptance to road subgrade standards than a controlled earth fill. Undercutting and recompaction is often a prudent course to follow on cut ground.

2.B.9. Geotechnical Completion Report

C9

Not all soils will require the application of the Means of Compliance to the same degree. For example, a site with no obvious risks or identifiable hazards, and which has not been subject to earthworks or other engineering works such as the installation of services, may simply require a short letter from a competent person reaffirming the suitability of the site for its intended purpose. This assessment should be reported in letter form to ensure continuity of record of the Quality Assurance audit trail.

2.B.9.1 A geotechnical completion report should cover, but not necessarily be restricted to, the following:

(a) Purpose

To report on the development after completion of the works with a view to recording construction information and extending the site investigation report if appropriate, and expressing an opinion on the suitability of all lots within the subdivision for their intended use.

(b) Scope

The report should describe all activities from the preliminary site investigation, to the completion of the physical works, including earthworks compaction control testing.

2.B.9.2. The report should cover, and re-evaluate where necessary, the requirements of the Geotechnical Investigation Report, the requirements of NZS 4431:1989, New Zealand Standard Code of Practice for Earthfills for Residential Subdivisions, where appropriate, and requirements set out in the conditions of subdivision consent.

2.B.9.3. The description should include sufficient test results, site inspection data, and other information to enable an independent assessment to be made as to the suitability of the development.

2.B.9.4. The emphasis of the report should be on stating what happened during construction, supported by detailed field notes, test results, and construction reports to provide an accurate detailed "as built" record.

A suitable check list for reporting is given in Drawing G2.

2.B.10. Summary

2.B.10.1. A good guide to the various aspects that should be addressed as a Means of Compliance with the Manukau City Council Engineering Quality Standards, Section 1, Geotechnical Assessment, is provided in the IPENZ (Auckland Branch), Code of Practice for Urban Land Development and NZS 4404:1981, New Zealand Standard Code of Practice for Urban Land Subdivision. These documents set out the areas in which the rational processes of engineering appraisal and design (site investigation) are necessary, and which requires the involvement of a geotechnical (soils) engineer.

2.B.10.2. In summary, Engineering appraisal and design are required:

(i) Prior to detailed planning, which usually involves some form of subsurface investigation.

(ii) During the review of and advice on design concepts.

(iii) During construction to ensure the adequacy of the bulk filling and the execution of the earthworks design.

(iv) After construction, to provide certification and/ or define limitations of the works.

C10

Site investigations for land development projects require the exercise of considerable skill, experience, and judgment, depending on the nature and complexity of the project.

With most of such projects, the site investigation aspects of a project extend throughout the project and may require considerable flexibility during the development phase, particularly if complex and varying conditions not anticipated from the formal phases of investigation are encountered.

The processes that have been outlined provide the means of achieving the objectives of sound foundation investigation for subdivisional and land development purposes, viz:-

(i) Identifying any constraints on land use.

(ii) Mitigating any risks.

(iii) Reducing the potential liability of all parties involved in the processes,

i.e. The owner
The engineer
The local authority

These processes define the state of the practice for subdivisional and land development investigation, which being based largely on a common sense pragmatic approach to hazard identification and risk mitigation, in conjunction with a good measure of practical down to earth experience, will ensure:-

(a) A low risk outcome for all parties involved.

(b) A product which is readily understood by most local authority officers, and which, through a logical process, should address their concerns.

(c) As a consequence of (b), a relatively easy progression through the approvals processes, which has a nett benefit to a client in terms of minimal delays.

(d) For a well engineered subdivision or land development project, a product which is readily marketed, as a consequence of identified and mitigated risks.

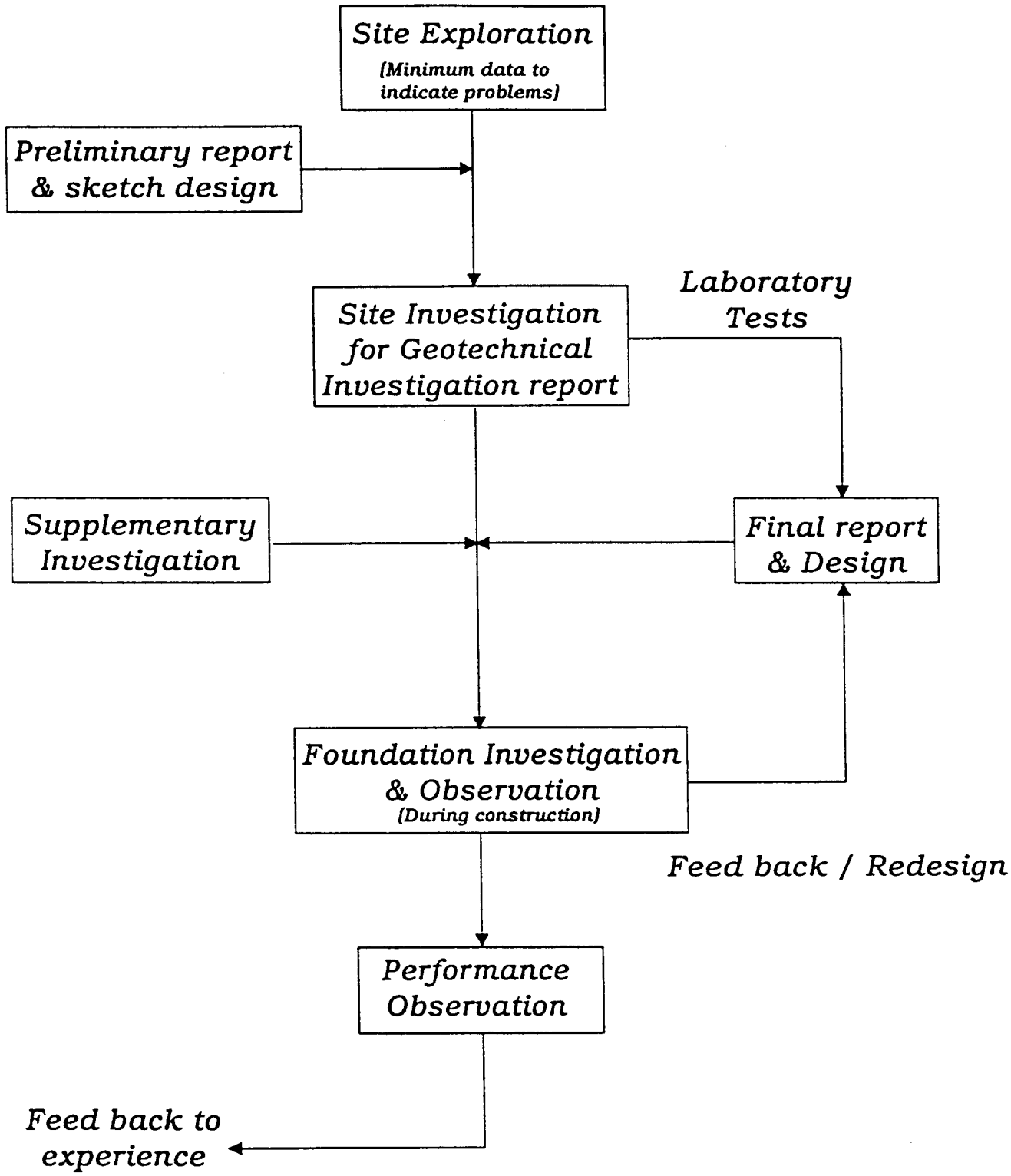
-i.e. minimum uncertainties.

The object and development of these processes has the underlying objective of communicating a technical appreciation of a site in a manner which addresses all the relevant issues, and in a manner which can be readily understood by all parties involved, including the lot purchaser/home owner.

The process may be subject to variation, depending on specific site circumstances. However, if followed rationally, the outcome should always be the same - a quality managed engineering product based on hazard identification and risk mitigation, and which can vary from a one page letter through to a sizable report, depending on the specific site circumstances.

It should be noted that any aspects submitted to Council in compliance with the Geotechanical Assessment requirements of Council's Engineering Quality Standards are the copyright of the writer, and belongs to the person for whom the report was prepared.

Updated: 1 April 1993



MANUKAU
City Council

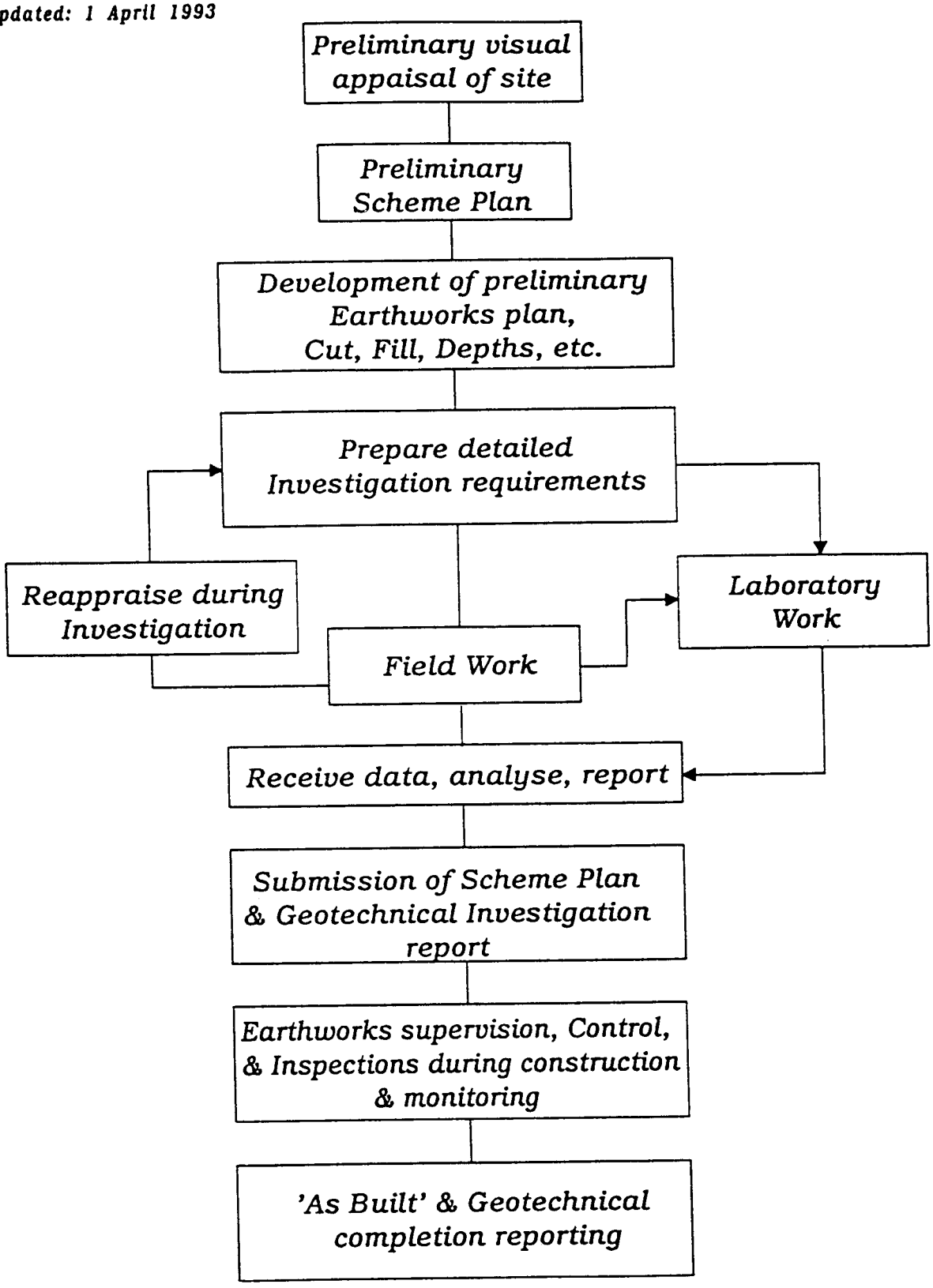
PHASES OF INVESTIGATION

Land Development Control

MANUKAU CITY COUNCIL

No. G1

Updated: 1 April 1993



MANUKAU
City Council

LOGIC FOR SITE INVESTIGATION OF SUBDIVISIONS

MANUKAU CITY COUNCIL

Land Development Control

No. G2

Updated 1 April 1993

1.INTRODUCTION

- *site history, aerial photos*

2.TOPOGRAPHY

- *past, present, filling*

3.GEOLOGY

4.FIELD AND LABORATORY INVESTIGATION (SCOPE)

- *borehole locations (drawing)*

5.SUBSURFACE CONDITIONS

- *Soil (fill?)*
- *Rock (drawing?)*
- *Groundwater (drawing?)*
- *Topsoil & Soil Strengths*
- *Organic (drawing?)*

7.DISCUSSION OF SUBSURFACE FEATURES AS THEY AFFECT THE PROPOSED DEVELOPMENT

8.GROUNDWATER & SURFACE WATER CONSIDERATIONS

- *Subsurface drainage (seasonal effects)*
- *Surface water drainage*
- *Post construction groundwater levels*

9.GROUNDWATER FLUCTUATIONS & SOIL MOISTURE CHANGES

10.INFLUENCE OF ORGANIC SOILS

11.SETTLEMENT CONSIDERATIONS
(Settlement vs fill height, time)

- *Bulk filling*
- *Buildings*

12.ALLOWABLE FOUNDATION BEARING PRESSURES

- *Undrained soil strengths*

13.SLOPE STABILITY

- *Cut slopes*
- *Fill slopes*
- *Strength parameters*
- *Form of analysis*
- *Conditions considered in analysis*
- *Results and comments*

14.EFFLUENT DISPOSAL

- *Suitable areas*
- *Soakage characteristics*
- *Limitations*
- *Design recommendations*

15.STORMWATER DISPOSAL

- *Site suitability for ground soakage*

16.EARTHWORKS CONSIDERATIONS

- *Proposed fill areas (drawing)*
- *Site preparation*
- *Fill materials*
- *Site preparation*
- *Compaction criteria*
- *Bulking considerations*
- *Road subgrades*

17. EXISTING FILLS

- *Engineered or non engineered*
- *Inferred extent*
- *Nature of fill*
- *Quality*
- *Suitability*

18.CONCLUSIONS AND RECOMMENDATIONS

TABLES

- **Atterberg Limit Test*
- **Triaxial Test Results*
- **Consolidation Test Results*
- **Other Test Results*
- **Distribution of Organic Compressible Soils*
- **Summary of Rock Depths*
- **Measured Groundwater Levels*
- **Measured Topsoil Depths*
- **Range of Undrained Shear Strengths*
- **Allowable Foundation Bearing Pressures*
- **General Stratigraphic Description of Soils*
- **Summary of Slope Stability Analysis Results*
- **Soakage Test Results*

FIGURES

- **Summary of Atterberg Limit Test Results*
- **Settlement vs Fill Height*
- **Soakage Disposal Field Trenches*

APPENDICES

- *1.Field and Laboratory Test Results*
- *2.Fill Specification*

DRAWINGS

- **Site Plan & Borehole Locations*
- **Cut & Fill Areas*
- **Cross Sections (limitations statement)*
- **Groundwater Considerations*
- **Drain Details*
- **Contours of Rock Surface*
- **Stability Analysis & Results*

MANUKAU
City Council

CHECKLIST FOR REPORTING

Land Development Control

MANUKAU CITY COUNCIL

No. G3

Updated 1 April 1993

1.INTRODUCTION

- *purpose of reporting*
- *related construction drawings, reports, etc*

2.SITE INVESTIGATION

- *General- Investigation Report references*
- *Dates*
- *Topography - Pre-site development*
- *Geology} As summarised*
- *Subsurface Conditions] from Site Investigations*
- *Fill materials} Report*

3.CHRONOLOGY OF CONSTRUCTION EVENTS

- *Dates of principal activities*
- *Contractors involved*
- *Stripping of topsoil*
- *Removal of unsuitables*
- *Occurrences of wet weather*
- *Influence of continuity of earthworks*
- *Identification of unusual features*
- *Failures*
- *Additional investigations*
- *Silt traps, location and treatment*
- *Underfill drainage*
- *Holiday periods*
- *Springs, special drainage*
- *Service trenches*
- *Road subgrades*
- *Completion of bulk earthworks*
- *Respreading of topsoil*
- *Earthworks completion*

4.EARTHWORKS EQUIPMENT USED ON SITE

5.CONSTRUCTION SUPERVISION AND CONTROL

- *Frequency of supervision/site visits*
- *Summary of principal activities and engineering observation during construction*
- *Photographs*
- *Control testing*
- *Location and elevation*
- *History*
- *Frequency of testing - reasons*
- *Summary of control criteria vs specification*
- *Class A fill*
- *Road subgrade*
- *Reserves*
- *Special engineering measures*
- *Springs, underfill drains, conduits, etc*
- *Opinion*

6.ROADING

- *Cut or fill*
- *Comments, etc as for "Construction Supervision and Control"*

7.UNCERTIFIED FILLS

- *Location*
- *Extent*
- *Reason*
- *Effect*

8.SERVICE TRENCHES

- *Depth*
- *Backfilling*
- *Special Limitations*

9.ORGANIC DEPOSITS

- *Removal or other treatment, if any*
- *Location*
- *Influence*
- *Constraints*

10.OPINION

- *Natural Ground and Cut and Fill Areas*
- *Limitations - organic, founding depths, etc*
- *Constraints on development timings*
- *Suitability for NZS 3604 construction*
- *Conclusions and Recommendations from Site Investigation Report as appropriate*

TABLES

**Compaction Control Test Locations and Results*

FIGURES

**Summary of Compaction Control Test Results*

APPENDICES

**Field and Laboratory Test Results*
**Fill Specification*
**Compaction Control Test Results*
**Photographs*

DRAWINGS

**Site Plan showing borehole locations*
- as built fill contours
- special engineering features (e.g. depth to rock or soft subsoils)
- test locations

MANUKAU
City Council

CHECKLIST FOR GEOTECHNICAL COMPLETION REPORTING

Land Development Control

MANUKAU CITY COUNCIL

No. G4

Limitations and Restrictions on Building Consents

36. Building on land subject to erosion, etc.-

(1) Except as provided for in subsection (2) of this section, *a territorial authority shall refuse to grant a building consent* involving construction of a building or major alterations to a building *if-*

(a) *The land on which the building work is to take place is subject* to, or is likely to be subject to, *erosion*, avulsion, alluvion, falling debris, *subsidence, inundation, or slippage;* or

(b) *The building work itself is likely to accelerate, worsen, or result* in erosion, avulsion, alluvion, falling debris, subsidence, inundation, or slippage of that land or any other property

unless the territorial authority is satisfied that adequate provision has been or will be made to -

(c) *Protect the land or building work or that other property* concerned from erosion, avulsion, alluvion, falling debris, subsidence, inundation, or slippage; or

(d) Restore any damage to the land or that other property concerned as a result of the building work.

(2) *Where a building consent is applied for and the territorial authority considers that -*

(a) *The building work itself will not accelerate,* worsen, or result in erosion, avulsion, alluvion, falling debris, subsidence, inundation, or slippage of that land or any other property; but

(b) *The land on which the building work is to take place is subject* to, or is likely to be subject to, erosion, avulsion, alluvion, falling debris, *subsidence*, inundation, or *slippage*; and

(c) The building work which is to take place is in all other respects such that the requirements of section 34 of this Act have been met-

the *territorial authority shall, if it is satisfied that the applicant is the owner in terms of this section, grant the building consent, and shall include as a condition of that consent* that the territorial authority shall, forthwith upon the issue of that consent, notify the *District Land Registrar* of the land registration district in which the land to which the consent relates is situated; and the District Land Registrar *shall make an entry on the certificate of title* to the land that a building consent has been issued in respect of a building on land that is described in subsection (1)(a) of this section.

(3) Where the territorial authority determines that the entry referred to in subsection (2) of this section is no longer required it shall send notice of the determination to the District Land Registrar who shall amend his or her records accordingly.

(4) *Where* -

(a) Any building consent has been issued under *subsection (2)* of this section; and

(b) The territorial authority *has notified the District Land Registrar in* accordance with subsection *(2)* of this section that it has issued the consent; and

(c) The territorial authority has *not notified* the District Land Registrar under *subsection (3)* of this section that it has determined that the entry made on the certificate of title of the land is no longer required; and

(d) *The building* to which the building consent relates later *suffers damage* arising directly or indirectly from erosion, subsidence, avulsion, alluvion, falling debris, inundation, or slippage or from inundation arising from such erosion, subsidence, avulsion, alluvion, falling debris, or slippage -

the *territorial authority* and every member, employee, or agent of the territorial authority *shall not be under any civil liability* to any person having an interest in that building on the grounds that it issued a building consent for the building in the knowledge that the building for which the consent was issued or the land on which the building was situated was, or was likely to be, subject to damage arising, directly or indirectly, from erosion, subsidence, avulsion, alluvion, falling debris, inundation, or slippage or from inundation arising from such erosion, subsidence, avulsion, alluvion, falling debris, or slippage.

(5) Where an application made by or on behalf of the Crown is such that, if the applicant were not the Crown, subsections (2) and (4) of this section would otherwise apply, the territorial authority, in approving any such application, shall notify the appropriate Minister and the Chief Surveyor for the land district in which the land is situated, and include with that notification a copy of the project information memorandum issued in respect of the building consent; and such notification shall be deemed to meet the requirements of this section.

(6) Where an application made by or on behalf of the owners of Maori land is such that, if the application were not in respect of Maori land, subsection (2) of this section would otherwise apply, the territorial authority, in approving any such application, shall notify the Registrar of the Maori Land Court, and include with that notification of a copy of the project information memorandum issued in respect of the building consent; and such notification shall be deemed to meet the requirements of this section.

(7) Where any notification is given pursuant to subsection (5) or subsection (6) of this section, the Chief Surveyor or the Registrar of the Maori Land Court, as the case may be, shall enter in his or her records the particulars of the notification together with a copy of the project information memorandum included with that notification.

(8) For the purposes of subsection (2) of this section, the term "owner" means the person having ownership of the fee simple of the land on which the building work is or has taken place.

VERIFICATION METHOD B1/VM4 FOUNDATIONS

2.0 SITE INVESTIGATION

2.0.1 The characteristics of the ground shall be assessed before the start of *construction* or any earthworks on a *building* site.

2.1 Preliminary investigation

2.1.1 The preliminary site assessment must address:

a) General land form, geology and any conditions likely to facilitate landslip, soil creep, shrinkage and expansion, or subsidence.

b) Available records of previous constructions, excavations, fillings, *drains* and concealed works, on and adjacent to the site.

c) History and behaviour of neighbouring *buildings*, and details of their foundation types, depths and loadings.

d) Potential for flooding (see also NZBC E1), and seasonal changes of soil characteristics.

e) Seasonal, tidal or other natural ground water changes.

f) Presence of corrosive soil and ground water, and effluents (see also F1/VM1).

2.2 Detailed investigation

2.2.1 The ground conditions at the *building* site shall be investigated to the extent necessary by test bores and excavations, inspection, laboratory and field testing of soil and rock samples, or by seeking the advice of a person with appropriate expertise and experience. The investigations shall provide sufficient data for the design of the *building*.

2.2.2 The description of the foundation material shall be recorded.

Comment:

A suitable method for describing soil and rock is contained in "Guide-lines for the field description of soils and rocks in engineering use" published by the New Zealand Geomechanics Society.

2.2.3 The site investigation record shall include a site plan showing the locations of the test bores and excavations.

3.2 Slope stability

3.2.1 Slope stability shall be analysed using unfactored loads. Slopes include unsupported earth faces, banks and vertical ground profiles.

Comment:

Work is underway on developing limit state designs for slope stability but is yet to be finalised.

Amd 2
Aug '94

3.2.2 Permanent slopes shall have a *factor of safety* against instability of no less than 1.5.

3.2.3 The *factor of safety* for temporary slopes shall be evaluated for each specific case, having regard to confidence in the soil and rock data and the consequence of failure.

News Flash for Geotechnical Engineers.

1. Did you know that Welded Beams were useful as Retaining Wall Piles ?

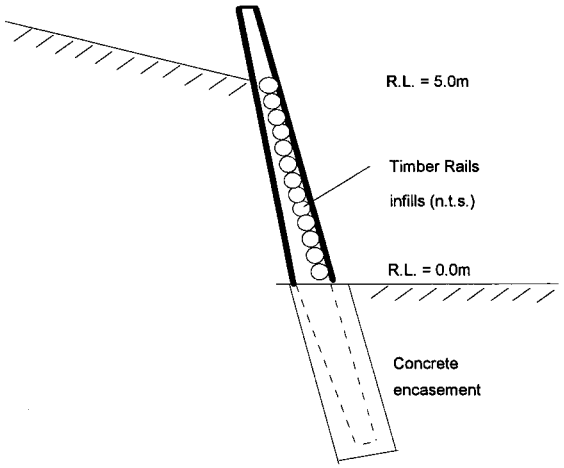


Figure 1. Westgate Retaining Piles.

A novel use of welded beams recently occurred on the Westgate Retail project in West Auckland. This was a 5m high, 250m long cantilever retaining wall with an additional surcharge loading, designed by Rodney Melville-Smith of Foundation Engineering Ltd. The retaining piles selected by Hauraki Piling were custom tapered welded steel sections reflecting the triangular loading pattern of the earth pressure, and complementing the overall appearance of the finished result.

2. Did you know that there are Welded Beam 'H' Piles now available ex-stock ?

We are pleased to advise that there are now a range of welded 'H' piles and Bearing piles held in stock. The piles currently held are in 7.95m lengths and are available for immediate delivery. As stock levels are constantly varying you should contact your local structural steel merchant for details on specific availabilities and price

Phone 09-267-3875 Fax 09-266-3080

STELTECH STRUCTURAL LTD

BHP New Zealand Steel

Limit State Design of Retaining Walls and Foundations for Geotechnical and Structural Engineers

This seminar was advertised in the last issue of Geomechanics News and took place in Christchurch, Wellington, Auckland and Taupo in early November. In excess of 200 people attended the four seminars each of which lasted for one day. The seminar was a joint activity between the Geomechanics Society and the Structural Engineering Society. The presenters representing the NZGS were M J Pender and T Matuschka For SESOC P Smith was the main presenter.

SESOC has a well established format for such seminars, some explanatory material is given but the bulk of the time is spent on the explanation of worked design calculations. The basis of the seminar was the proposals made by the NZGS and SESOC group that met last year in Auckland to consider the provisions in NZS4230:1992 for foundation design. This came up because the Loadings Standard assumes that designs will be done in a limit state format. There are some suggestions in the code as to how this might be done and the joint committee developed things from there. Recommendations and background information were published in the SESOC Journal for December 1993 and also repeated in the last issue of Geomechanics News (also reproduced in this issue because some pages were inadvertently omitted from the last issue).

The seminars were well received and provided some very useful feedback on the proposals made by the committee. There were many thought provoking contributions from members of the audience and the consensus at the end of each day was that the seminars had been very worthwhile. The main areas of discussion were about the need to use a factor of 0.9 on actions that contribute to stability when estimating the ideal resistance and the manner in which thrusts generated from earth pressures should be factored.

Those who presented the seminars and prepared the worked examples are now developing some revisions of the provisions and also the extending them to cover pile foundations and foundations for capacity designed structures. These updated suggestions will be published in a future issue of the SESOC Journal along with a set of worked examples.

Apples, oranges and factors of safety

M.J. Pender

We have reports in this issue of Geomechanics News of the Geotechnical Limit State Design seminar and also some discussion of appropriate values for factors of safety in relation to slope stability applications. These two topics have caused me to ponder the meaning of factors of safety and to wonder what the numerical values actually tell us.

If we start with the traditional wisdom and look up typical recommendations for factors of safety we will find a list something like this: bearing capacity 3, sliding failure of a gravity retaining wall 2, and slope stability 1.5. These suggestions will usually be hedged with warnings about the need to exercise judgement in choosing a design factor of safety because of such things as the consequences of failure and our confidence in the values of the soil properties etc. I have to admit to never, until recently, having thought in detail about the significance of the various values for the factor of safety. I had always assumed that the above sequence of values is reasonable as a bearing capacity failure of a foundation would usually be more serious than the outward sliding of a gravity retaining wall which in turn might be more serious than a slope failure.

The recent discussion of limit state ideas has led me to the realisation that the definition of the factor of safety used for bearing capacity calculations is quite different from that used in slope stability situations. The conclusion I have reached is that the apparent differences in the relative risk of failure are more apparent than real so that to get a valid appreciation of what the various factor of safety values mean we need to compare apples with apples.

Firstly we need to review definitions of factors of safety. The slope stability definition is based on how much of the shear strength of the soil is mobilised:

$$F_{\text{slope stability}} = \frac{\text{average shear strength along the failure surface}}{\text{average shear stress along the failure surface}}$$

For a soil with both frictional and cohesive strength components we assume that the same proportion of c' and tanϕ' are mobilised, thus at some point on a failure surface with a factor of safety F we have:

$$\text{Shear stress} = \frac{\text{Shear strength}}{F} = \frac{c'}{F} + \frac{\sigma_n' \tan\phi'}{F}$$

On the other hand the definition for bearing capacity is based on the ratio of the net ultimate bearing pressure to the net applied bearing pressure:

$$F_{\text{bearing capacity}} = \frac{\text{net ultimate bearing pressure}}{\text{net applied bearing pressure}}$$

The ultimate bearing pressure is calculated with bearing capacity factors which are quite complex functions of the friction angle of soil and do not simply involve tanϕ'. This means that the FoS definition for bearing capacity is quite different from that used in slope stability calculations.

The significance of the differences between these definitions will be illustrated by considering the results of a simple bearing capacity calculation as set out in the table below.

Bearing capacity of a strip foundation, 2 m wide, founded 1 m beneath the surface (horizontal) of a layer of cohesive soil with c' = 20 kPa, ϕ' = 25^0 and γ = 18 kN/m^3.	
Net ultimate bearing pressure for the given soil properties: (ie $F_{\text{bearing capacity}} = 1.0$)	845 kPa
Net applied bearing pressure for $F_{\text{bearing capacity}} = 3.0$:	845/3 = 282 kPa
Now we ask what proportionate reduction in tanϕ' and c' will give an ultimate bearing pressure of 282 kPa? The answer is 0.6. This means that a cohesion of 12kPa (20x0.6) and a friction angle of 15.7° (0.6xtan25° = tan 15.7°) are mobilised for the strip foundation detailed in the heading when the bearing capacity factor of safety is 3.0. Having reached this conclusion we can calculate the "Slope stability" factor of safety corresponding to $F_{\text{bearing capacity}} = 3.0$:	$F_{\text{slope stability}}$ = tan25°/tan15.7° = 1.67 = 20/12 = 1.67

The results of the calculations for the particular situation specified in the heading to the table show that $F_{\text{bearing capacity}} = 3$ is approximately equivalent to $F_{\text{slope stability}} = 1.7$. The apparent large difference in the reserve of strength between the bearing capacity situation, with a traditional value of 3, and slope stability applications, with a factor of 1.5, is then merely a consequence of different definitions of the factor of safety and a consequent erroneous comparison of the bearing capacity value (orange) with a slope stability value (apple). If one accepts the hierarchy of failure scenarios mentioned above (bearing failure more serious than sliding of a retaining wall etc) then a slope stability factor of safety of 1.5 possibly seems too conservative in relation to the observation that the appropriate equivalent bearing capacity value, for comparison purposes, is 1.7.

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Suggestions for Geotechnical Ultimate Limit State Design in NZ

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Introduction

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

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

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

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This meeting had several papers proposing various limit state methods as well as papers reporting on the experience of various countries with the use of limit state methods. One of these papers was from Canada and explained how it is planned to reissue the Canadian Foundation Code in 1995 with limit state and conventional formats in parallel; at a later stage it is expected that the code will be entirely in a limit state format. Also in Canada, the Ontario Bridge Code has been in a limit state format since 1983, the third edition was published in 1991. In 1991 a report was published under the auspices of the National Cooperative Highway Research Programme of the United States Transportation Research Board. This dealt with the design of bridge foundations and proposed a limit state approach. Also in 1991 a document on the design of retaining structures was released for comment by the Geotechnical Office of the Hong Kong Government, this proposes limit state methods. Access to much of this international information has provided important background for the committee preparing the proposal presented in this paper.

Methods for Ultimate Limit State Design

There are three distinct approaches to ultimate limit state design in geotechnical engineering. These are:

Total Factor of Safety: $R/FoS = \Sigma F_i$

Partial Factor of Safety: $R_f \geq \Sigma \alpha_i F_i$

Load and Resistance Factored: $\Phi R \geq \Sigma \alpha_i F_i$

where:

F_i	are the applied loads
R	is the ideal[1] resistance (or strength)
FoS	is the total factor of safety
α_i	are load factors
Φ	is the strength reduction factor[2]
R_f	is the resistance calculated using the design strength parameters obtained by reducing the characteristic strength values (ie $(c')_d = c'/P_{fc'}$ and $(\tan\phi')_d = \tan\phi'/P_{f\phi'}$, or $(s_u)_d = s_u/P_{fsu}$).
$P_{fc'}$	etc are the partial factors of safety. (Note that there is more than one partial factor of safety.)

Herein the load and resistance factored design method will be referred to with the abbreviation: LRFD.

It must be appreciated that the total factor of safety method is associated with a range of factors of safety. In the case of the bearing capacity of foundations values in the region of 3 are expected, a factor of safety between 1.5 and 2 is usual for retaining structures, and for slopes values of about 1.5 are typical. These differences reflect the consequences of failure in the various applications.

Although quite distinct there are very strong similarities between the three approaches. Thus one way of evaluating a new proposal is to check how the designs reached compare

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[1]This resistance is commonly referred to with several terms. Among them are *theoretical* resistance, *nominal* resistance as well as *ideal* resistance.

[2]Referred to as a Performance Factor or Resistance Factor in some documents.

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

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

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

Developments elsewhere

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

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

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

under evaluation for inclusion in the foundation design section of the National Building Code of Canada. The strength reduction factors proposed for use in Canada are very similar to those proposed herein.

In the United States the National Cooperative Highway Research Programme funded research into the design of bridge foundations. The report of this work, Barker et al (1991), presents a series of manuals covering ultimate and serviceability limit state design for shallow foundations, retaining structures, and deep foundations. All of these are presented in an LRFD format. Currently the methods given in these manuals are offered by AASHTO as an alternative to working stress methods for the design of bridge foundations. In addition the Barker et al manuals are the basis for current work on the development of a more complete foundation design manual for AASHTO. An interesting feature of the manuals is the recommendation of a range of values for the strength reduction factors depending on the method used for estimation of soil strength values. For example the Φ factor for resistance estimates based on the Standard Penetration Test (SPT) is more severe than that associated with strength estimates based on the Cone Penetration Test (CPT). A table of these suggested strength reduction factors for shallow foundations is reproduced in Appendix I (other tables are given in the report for pile and drilled shaft foundations). The load factor section of the Barker et al report proposes a special factor for loads derived from active earth pressure.

Several ultimate limit state methods are compared in Appendix II.

NZS 4203: 1984

This code had a live load factor of 1.7 and a factor of 1.7 for loads derived from earth pressures.

Taking into consideration the factoring of the loads the code states that 1.8 is a satisfactory factor of safety for shallow foundations (clause C3.3.7.1). The justification of this, given by Taylor (1976), recalled that the traditional value for the bearing capacity factor of safety is 3 and noted that the product of 1.7 times 1.8 is about 3. It was pointed out by Professor Taylor that the factor of 1.8 is closely related to a strength reduction factor. If instead of using a factor of safety of 1.8 one used a strength reduction factor of $1/1.8 = 0.56$, essentially the same design would result. Thus the 1984 version of NZS 4203 is closely related to the LRFD approach proposed herein.

NZS 4203:1992

This edition of the Loadings Standard has no specific load factor for loads derived from earth pressures.

Clause 6.6.3 states that the strength reduction factor for soil is to be taken as 0.6. In the commentary, clause C6.6.3, it is implied that this value is somewhat tentative pending a geotechnical standard. The following difficulties arise from the value of 0.6 and the way it is used in the worked example in the commentary:

(a) Clause 6.6.3 refers to a strength reduction factor but in the example in clause C6.6.3 this factor is applied to the soil strength parameters in the manner of a partial factor of safety. Thus there is confusion in terminology.
(b) In the example in the Commentary the 0.6 is applied as a partial factor of safety.

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

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

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

Proposal

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

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

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

(i) Load Factors and Load Combinations

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

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

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

the unit weight of the soil involved in any bearing capacity calculations is factored by 0.9. Application of these load factors is illustrated in the calculations for the three examples discussed below and given in Appendices III and IV.

(ii) **Strength Reduction Factors for Shallow Foundations**

Two strength reduction factors are proposed as follows:

Φ_{bc} = 0.5, for bearing capacity and resistance derived from passive earth pressure.

Φ_{sl} = 0.8, for sliding resistance.

These values are comparable with those presently under consideration in Canada, Becker et al (1993). It was explained above that NCHRP Report 343, Barker et al (1991), proposes a range of values for strength reduction factors depending on how the strength parameters were determined. It is apparent from the table in Appendix I that the above values are representative of the range in the NCHRP report.

A legitimate question is why are two Φ values proposed. Firstly this is consistent with other LRFD methods which in turn reflects the fact that the sliding and bearing capacity mechanisms are quite different. Sliding failure is not usually as significant as bearing failure and is mobilised at very small displacements without rotation. Sliding resistance depends directly on the tangent of the friction angle. In the case of passive earth pressures and bearing capacity the resistance is not controlled directly by the tangent of the friction angle but by complex functions which change very rapidly once the friction angle is beyond the mid 30's. In addition the displacements required to mobilise passive resistance and bearing capacity are large.

(iii) **Strength Reduction Factors for Deep Foundations**

In this paper the primary applications intended for the ultimate limit state method are retaining walls and shallow foundations. However, values of Φ values similar to those for shallow foundations are given by Barker et al (1991) for deep foundations. In addition Becker (1993) gives strength reduction factors for deep foundations. Thus the LRFD approach can be expected to be applicable to deep foundation design.

(iv) **Strength Reduction Factors for Slope Stability Analysis**

Eurocode 7 includes a chapter on slope stability. The justification for this is that often slopes have foundations and retaining walls attached to them, thus a limit state approach would be in the interests of consistency. The proposals set out above for shallow foundations could be applied to slope stability calculations. If one assumed that a slope failure was an example of sliding and that a dead load factor of 1.4 was appropriate, then the resulting factor of safety is 1.75. This can be compared with the common "target" value of 1.5 for routine slope stability work. Values less than this are accepted when there is good control on material properties, such as in the construction of an earth dam, and greater values are used when the material properties were highly variable, such as for a natural slope in volcanic soil. Within the limit state format these application differences could be taken into account when assigning the design strength.

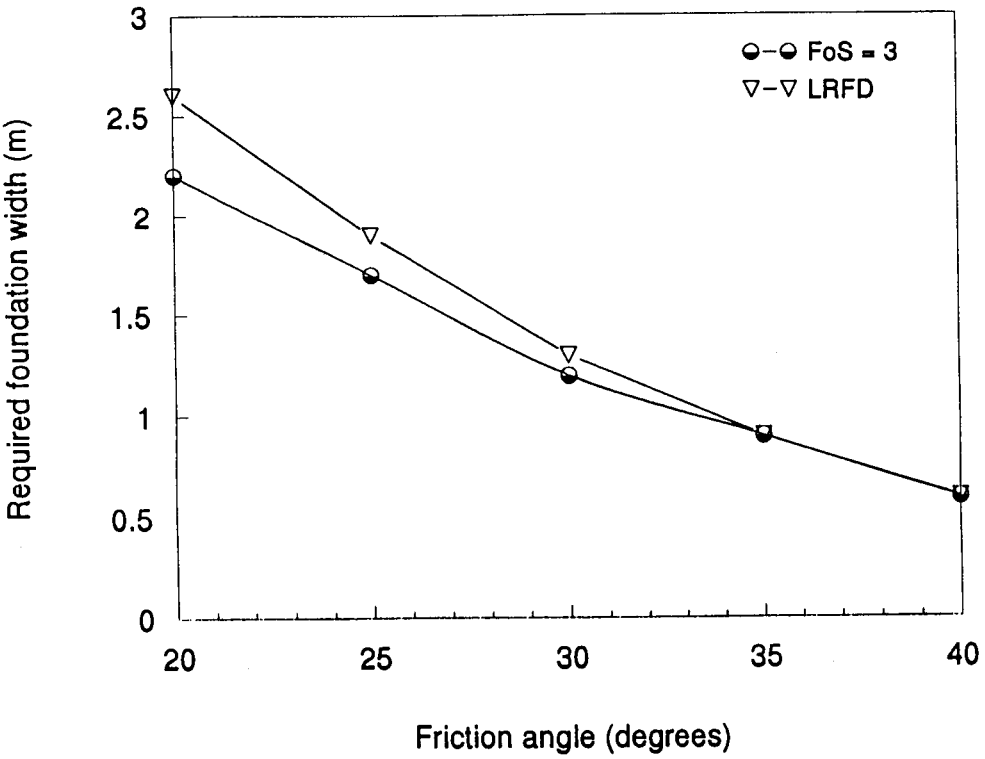

Figure 1 *Required strip foundation width for foundations in dry sand. Comparison of the widths required using the proposed LRFD and total factor of safety methods.*

Examples

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

(a) Width of a strip footing in dry sand

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

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

This example covers calculation of the maximum ideal stem moment for a simple cantilever retaining wall. The intention of the calculations is to compare the maximum ideal stem moment given by a number of approaches. Active earth pressures on the retaining wall were derived from the Coulomb theory for a dry backfill with a unit weight of 16 kN/m^3. The ideal section strength was derived from the limit state or ultimate strength modified by the strength reduction factor consistent with the design and materials code combinations being considered. On this basis of the ideal maximum stem moments for each code can be compared. The calculation of the stem moment for the proposed method is as given in the example set out in Appendix IV under the heading Third Limit State (page 4 of the worked solution). The bending moments calculated using various material codes

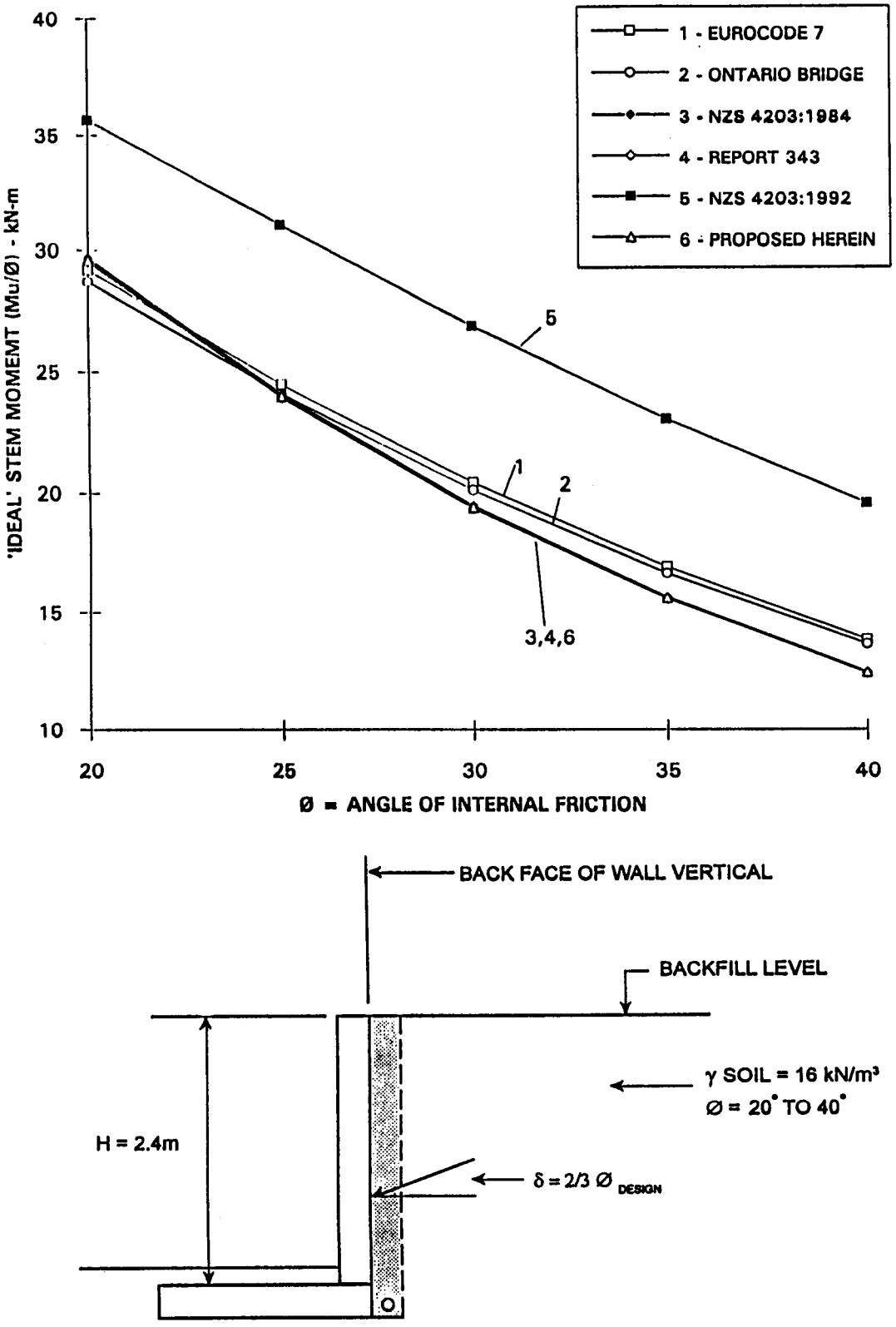

Figure 2 *Comparison of the maximum ideal stem bending moment for a cantilever retaining wall given by various methods.*

are compared in Figure 2. It is clear that the maximum moment at the base of the stem calculated using the proposal of this paper is almost exactly the same as that given by the provisions of NZS 4203:1984 and several other methods of design. In comparison NZS4203:1992 gives values which are 20-40% higher for the range of backfill friction angles considered.

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

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

Comparison

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

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

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

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

Material parameters

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

expected.

Conclusions

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

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

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

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

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

Note:

1 ϕ = friction angle of sand

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

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

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

Notes:

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

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

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

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

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

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

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

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

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

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

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

Vertically loaded foundation in dry sand.

Depth of foundation: Df = βD.

ORIGIN = 1

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

$RadDeg = DegRad^{-1}$

VD = 50. Dead load (kN)

VL = 100. Live load (kN)

β = 0.5 Df/B

αD1 = 1.4 Load factor for dead load

αD2 = 0.9 Dead load factor for surcharge

αL = 1.6 Load factor for live load

Φbc = 0.5 Strength reduction factor

F = 3.0 Bearing capacity factor of safety

gamma = 18 Unit weight of soil

i = 1..5

$\phi_i = 20 + (i - 1)\cdot 5$ Array of friction angles

$\phi_i = \phi_i \cdot DegRad$

Bearing capacity factors:

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

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

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

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

dg = 1

Alter Nc, Nq, and Nq:

$Nq\lambda_i = Nq_i \cdot dq_i$

$Ng\lambda_i = Ng_i \cdot dg$

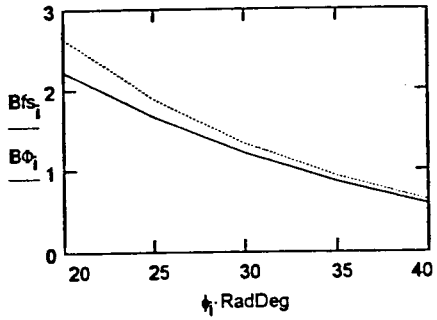
Now evaluate required width of foundation using LRFD approach:

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

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

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

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



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

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

Calculations by:
M J Pender,
Department of Civil & Resource Engineering,
University of Auckland.
ph: (09) 3737 599 ext. 7919 fax: (09) 3737 462

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

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

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

Dry sand backfill and clay foundation.

Short term analysis of bearing capacity and sliding of foundation.

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

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

Five possible ultimate limit states:

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

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

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

(iv) overall short term slope stability failure,

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

MathCAD housekeeping:

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

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

WALL AND BACKFILL PROPERTIES:

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

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

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

ACTIVE PRESSURE COEFFICIENT:

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

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

ACTIVE THRUST:

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

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

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

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

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

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

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

$xcg = \frac{Mom}{Vert}$ xcg = 0.656 Distance from heel to Vert.

$xreaction = xcg + \frac{Pah \cdot Hw}{3 \cdot Vert}$ Where reaction crosses base.

$Bd1 = 2 \cdot (Bw - xreaction)$ Bd1 = 0.819

$Bd2 = 2 \cdot (xreaction)$ Bd2 = 2.381

$Bd = if(Bd1 > Bd2, Bd2, Bd1)$ Bd = 0.819

$\sigma v = \frac{Vert}{Bd}$ σv = 168.455

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

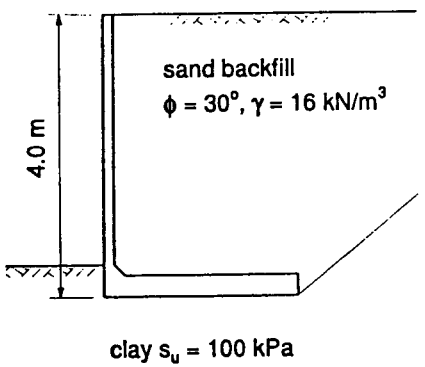
$ic = 0.5 \cdot \left(1 + \sqrt{1 - \frac{Pah}{su \cdot Bd}}\right)$ ic = 0.785

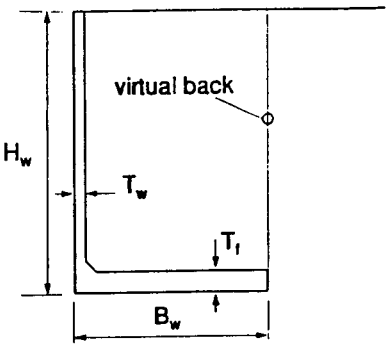
Nc = 5.14

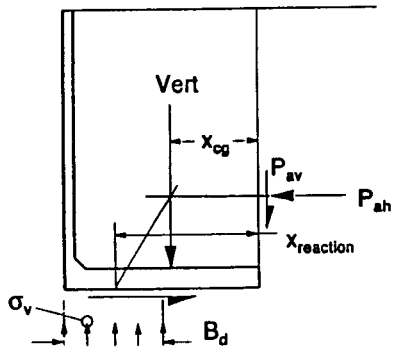
$qu = su \cdot Nc \cdot ic$ qu = 403.534 $quΦ = qu \cdot Φbc$

σv = 168.455 quΦ = 201.767 If quΦ > σv, then OK

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







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

SECOND ULTIMATE LIMIT STATE (sliding):

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

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

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

$Rsd = su \cdot Bd$ $\quad RsdΦ := Rsd \cdot Φsl$

Pah = 55.296 $\quad RsdΦ = 65.545$ $\quad$ If RsdΦ > Pah, then OK

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

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

$$BM := Ka \cdot gammabf \cdot LfP \cdot \frac{(Hw - Tf)^3}{6}$$ Maximum bending moment in wall stem (kNm/metre run of wall)

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

$$BMcap_{ideal} := \frac{BM}{Φbm} \qquad BMcap_{ideal} = 63.233 \quad \text{(kNm/metre run of wall)}$$

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

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

FIFTH ULTIMATE LIMIT STATE (Check on long term effects)

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

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

CONCLUSION:

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

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

THE SECOND AUSTRALIA-NEW ZEALAND YOUNG GEOTECHNICAL PROFESSIONALS CONFERENCE

NOTICE

22nd to 25th November 1995

Aims

To bring together young geotechnical engineers and engineering geologists so they may become more aware of the work of others (similarly and more experienced) in their field. 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 their affairs. To continue the success of the first conference of this type in the Australia-NZ region.

Location & time

From Wednesday evening 22nd to Saturday afternoon 25th November 1995 at the University of Auckland, Auckland, N.Z. It is hoped that organisations from both Australia and the Pacific will sponsor registrants to this conference. It is expected that the third conference will be held in 1998.

Target audience

One or two fully funded registrants from each of the sponsoring organisations. It is expected that each organisation will cover the full conference cost for their registrants, this will include: accommodation, meals, conference and travel. Registrants are expected to contribute their time from leave entitlements and should be aged 35 years or younger. The number of registrants will be limited to about 30 to ensure a relatively relaxed atmosphere. If too many applications 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 Australasian geotechnical community. Attendees of the first conference reported that the experience was of great value and a chance to exchange views with peers from around NZ and Australia.

Format

The conference format will be:

* The conference will be residential even for those registrants who reside in Auckland, registrants arriving on Wednesday November 22nd.

* Each day there will be a single paper presented by an experienced geotechnical professional on: professional practice, interesting case studies, or new topic areas (e.g. environmental geomechanics, soft rocks, ground improvements etc.). These speakers will be carefully selected, invited and will act as mentor for the day, along with the organising committee. The audience will therefore be, say, four or five senior professionals and the young registrants.

NOTICE

THE SECOND AUSTRALIA-NEW ZEALAND YOUNG GEOTECHNICAL PROFESSIONALS CONFERENCE

* Each registrant will have to submit to the conference (for prior inclusion in a limited distribution proceedings) a 3 to 5 page "paper" on a project on which they had been involved (research or work related). The registrants will then present a 15 to 30 minute discussion of the project to the conference. This is not meant to be onerous but will provide valuable experience in preparation and presentation of papers and enable the ready transfer of experience between registrants. This activity is a requirement of all registrants.

* Two half day site visits to local projects of interest led by experienced geotechnical professionals and visits to local university geotechnical engineering departments.

* Evenings will be informal social gatherings; the programme is yet to be finalised but may include: first, registration and drinks; second, barbecue; third, meal at local restaurant. University college breakfasts and lunches.

No single commercial organisation will dominate the conference and access by senior staff will be limited so each organisation need not be concerned that the conference will become a recruiting forum for the opposition. Hopefully each sponsoring organisation will be confident of their ability to keep their own staff.

Cost

Registration cost will be kept to an absolute minimum and will include:

* College accommodation, breakfasts and lunches.
* Three evening functions.
* Proceedings.

Registration cost should be between $300 and $400, subject to more careful costing.

****

Are you interested?

There will be no general distribution of bulletins nor call for papers; this notice will be the main request for expressions of interest. If you or your organisation are interested in participating/sponsoring please write to or fax:

Prof. M.J.Pender
Chairman, Organising Committee,
Second ANZ Young Geotechnical Professionals Conference
C/- The School of Engineering, Dept. of Civil & Resource Engineering,
The University of Auckland,
Private Bag 92019, Auckland, New Zealand.

Fax: +64 9 3737 462 Int'l fax: +64 9 3737 462

and further particulars will be sent to you in due course. In the first instance please provide the following details: 1. Organisation name, 2. Post address, 3. Fax number, 4. Contact name & phone number, 5. Possible nominee registrant (if known, leave out if selection pending) and 6. Likelihood of sponsoring one or more nominees.

NOTICE

NZ GEOMECHANICS SOCIETY STUDENT PRIZE

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 David Bell of the Geology Department, University of Canterbury, Private Bag, Christchurch.

TAKEN FROM NORMAL ACCIDENTS

Living with High-Risk Technologies
Basic Books 1984

Charles Perrow

Losing a Lake

... Our final accident in this chapter provides comic relief from the grim world of mining and an example of a human-made modification of a natural system that produced a genuine system accident. It occurred in 1980 in Louisiana when the state lost an oil rig, a salt mine, and an entire lake. I will draw entirely on the very entertaining account of Michael Gold[1] in the journal *Science 81*.

It is an example of the proximity problem that briefly created an interactive and tightly coupled system out of two independent linear systems. We do not generally expect a Louisiana lake, an oil rig, and a salt mine to suddenly be subsystems of a larger system.

You may have guessed the interaction. Texaco was drilling for oil in Lake Peignur in southern Louisiana. The drill was down 1,250 feet when it got stuck, and when pulled loose, the drill inexplicably jumped up and down in five- and ten-foot leaps. This is unusual for a device that weights 40 tons. One hour later the men noticed that the rig was listing badly, and they abandoned it. Watching from the shore they were surprised to see it sink from sight in a part of the lake that was only 3 to 6 feet deep. Meanwhile, men working a salt mine, part of which extended under the lake about 1,300 feet below it, noticed that their area of the mine was flooding, and they sounded the alarms. All fifty-one of them managed to escape as water rushed in to caverns that were 80-feet high and as wide as a four-lane highway. Meanwhile, on the surface, a whirlpool threatened some early morning fishermen, and eventually pulled in some barges and a tug associated with the drilling. The whirlpool grew in size until it pulled in 65 acres of the Rip Van Winkle Live Oak Gardens, a tourist attraction. A river (more like a canal) to the Gulf of Mexico, flowing out of the lake, reversed its course, eventually creating a 150-foot waterfall. An underground gas well ruptured and sent bubbles of natural gas to the surface, where they burned. After seven hours, the entire lake, once about one mile by two miles, had drained into the salt mine.

[1] Michael Gold. "Who pulled the plug on Lake Peignur" Science 81, (Nov. 1981)

Diamond Crystal, the salt company, sued Texaco for ruining its very lucrative mine; Texaco sued the salt company, claiming they had not notified the company where their mine was wandering. The drilling company sued Texaco for the loss of equipment. The Rip Van Winkle Gardens sued both Diamond and Texaco for the loss of buildings and plants, including 30,000 poinsettias ready to bloom for Christmas. Eventually, seven lawsuits were filed. The damage ran into the hundreds of millions of dollars.

Everyone in danger, it turned out, was lucky. The fifty-one miners got out by the skin of their teeth, driving their trucks on the soggy salt under ground highways. The two fishermen, noticing the lake dropping and the catfish jumping and swimming for deeper water, headed into the lake only to confront the whirlpool. They were blocked from behind by mud too deep to stand in, which would eventually go into the hole, and in front on one side by a row of barges cabled together, with the whirlpool to the other side. But two barges broke loose and were slurped up by the whirlpool, and this left room for the fishermen to motor at full speed for a bank, and to scramble out. The river, or canal, that began to flow backwards created a 20-knot current near the top of the waterfall, and boats moored along it were pulled in. A tug tried to lay a long barge across the river to stop the shrimp boats from being pulled in, but both the barge and the tug went over the waterfall, just after the crew jumped into the muck on one side of the canal. The barge and tug went into the whirlpool and down into the salt mine, where they still are. Seven other barges later popped up after the salt mine filled.

As the water poured into the mine, the air vented out through a ventilation shaft, keeping the emergency elevator bouncing around at its top. Then a 400-foot geyser erupted from the shaft, lasted for twenty minutes, and died. The next day the lake started to refill, after the mine had consumed an estimate 3.5 billion gallons of water. A few days later the owner of the Gardens had to hire a team of scuba divers to try and recover his wine cellar.

The salt mine was a big one, dug into the top of a salt dome that was about one mile across. Several shafts radiated out from the central shaft, the critical one about three-quarters of a mile out. Oil is often found near salt domes. When Texaco started drilling, they didn't bother to notify the Diamond Salt Company. Diamond learned of the drilling when they were asked by the US Army Corps of Engineers if they had any objections to a dredging permit sought by Texaco. Texaco had charts showing where the mine was, but

the charts conflicted, one put the mine under the drill hole, the others did not. Texaco never checked with its neighbour. Diamond never checked with Texaco. Texaco claimed that only Diamond knew where both the rig and the mine tunnels would be. Diamond claimed they had no information on how deep the rig would drill. A typical bureaucratic tale, one might say - lack of communication. But more to the point, it involved an unexpected interaction of two systems that were more proximate than anyone thought to notice.

Had Texaco tried to hit a mine tunnel from available charts, even one as wide as a four-lane highway, one suspects they would have failed. System accidents are infrequent even in complex, tightly coupled systems. They are exceedingly rare, and truly freakish, when two large systems such as this, apparently independent, briefly interact. That is why no one bothered to check. Who would have thought that....?

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GEOTECHNICAL ENGINEERING OF EMBANKMENT DAMS

by Robin Fell, David Stapleton and Patrick MacGregor

The aim of this book is to present a 'state-of-practice', particularly highlighting common problems, errors and omissions. *Topics:* Embankment dams, their zoning and selection; Weathering processes and profiles in valleys; Geotechnical questions associated with various geological environments; Planning, conducting and reporting of geotechnical investigations; Site investigation techniques; Laboratory testing techniques and their limitations; Design, specification and construction of filters; Clay mineralogy soil properties, dispersive soils and piping failure; Dams on highly permeable soil foundations; Stability analysis; Foundation preparation and cleanup; Foundation grouting; Embankment details; Specification and quality control of earthfill and rockfill; Design of dams to withstand earthquakes; Concrete face rockfill dams; Mine and industrial tailing dams; Monitoring and surveillance of embankment dams; References. 90 5410 128 8, July 1992, 680 pp., Hfl.165/$95.00.

GEOSYNTHETICS CASE HISTORIES

Edited by G.P. Raymond and J-P. Giroud

The book was prepared under the auspices of the International Society for Soil Mechanics and Foundation Engineering's Technical Committee TC9 on Geotextiles and Geosynthetics.

The book contains 127 case histories involving geosynthetic materials used in geotechnical engineering applications reported from 24 countries. The case histories are organized into 11 chapters based on type of application. These cover most of the applications of geosynthetics in geotechnical engineering.

The chapters are as follows:

1. Dams (12 case histories)
2. Containment ponds, irrigation and reservoirs. (14 case histories)
3. Scour erosion control (10 case histories)
4. Slope, bank and pipeline erosion control (15 case histories)
5. Tunnel and hydroelectric applications (4 case histories)
6. Railways and asphalt pavements (13 case histories)
7. Deep foundation improvement (11 case histories)
8. Pavement subgrade and base reinforcement (16 case histories)
9. Fill and embankment reinforcement (12 case histories)
10. Reinforced soil slopes (9 case histories)
11. Reinforced soil walls (11 case histories)

The materials used in the projects presented in the case histories encompass all types of geosynthetic materials including geotextiles, geomembranes, geomats, geonets, geogrids, geocomposites, and related products (some of them made from natural fibres, but included into the geosynthetic category according to the prevailing usage). Also, a number of case histories deal with geosynthetics system; soil-yard and soil-mesh mixtures, geotextile-concrete erosion control mattresses, and concrete blocks or panels associated with geosynthetic reinforcements, to name a few.

All case histories are presented in a consistent manner. The left-right two-page format gives a quick access to all the information provided for each case history. All case histories contain the same type of information presented, as much as possible, in the same order.

The 127 case histories presented in this book demonstrate the feasibility of using geosynthetics in a wide spectrum of geotechnical applications. Many of the projects described have been in use for ten or more years and their excellent performance since construction provides a valuable indication of the durability of geosynthetics.

Readers can purchase the book by writing to:

BiTech Publishers Ltd.
173 - 11860 Hammersmith Way
Richmond, British Columbia, V7A 5G1
Canada

The book contains 277 pages and is printed in hard cover. The cost of the book is $(US)107.00 or $(Cdn)131.00 plus GST if applicable. The cost includes shipping and postage.

The above reviews have been reproduced from the recent Australian Geomechanics Society News Journal, November 1994

1995

FEBRUARY 5-7, 1995
Newcastle, NSW, Australia
ENGINEERING GEOLOGY OF THE NEWCASTLE-GOSFORD REGION
Themes: Regional Engineering Geology and Engineering Activities (Hydrogeology tunnelling; Mine subsidence; Slope stability; Reactive soils; Coastal and marine engineering; Construction material; Earthquake engineering).

APRIL 2-7, 1995
St Louis, Missouri
THIRD INTERNATIONAL CONFERENCE ON RECENT ADVANCES IN GEOTECHNICAL EARTHQUAKE ENGINEERING AND SOIL DYNAMICS
Topics: Deformation of soils and rocks under monotonic and cyclic loads; Liquefaction; Practice in geotechnical earthquake engineering; Seismic design of piles; Lab and field testing; Dams and landfills under seismic conditions.

APRIL 11-13, 1995
Istanbul, Turkey
INTERNATIONAL SYMPOSIUM ON GEOTECHNICS - 70 YEARS OF SOIL MECHANICS
Topics: Terzaghi classics - tunnels, dams, slope stability, retaining structures, open excavations; New methods in laboratory and in-situ testing techniques - ground characterisation; Environmental geotechniques.
A number of invited papers related to the theme of the symposium will be published prior to the symposium and will be made available to each registered participant. Other contributions and proceedings of the symposium will be published at a later date.
Language: English

APRIL 10-14, 1995
Vienna, Austria
MECHANICS OF JOINTED AND FAULTED ROCK
Topics: Geology; Faulting; Applied Fracture Mechanics; Dynamics of Jointed Rock; Physical Modelling; Numerical Studies; Seismicity and Tectonics; Testing; Instrumentation; Hydraulics; Mining and Quarrying; Tunnelling and Underground Construction; Environmental Technology; Special Problems.
Language: English.

MAY 10-12, 1995
Hiroshima, Japan
INTERNATIONAL SYMPOSIUM ON COMPRESSION AND CONSOLIDATION OF CLAYEY SOILS
Themes: Measurement of compression and consolidation of clayey soils; Consolidation theory; Case histories and their implications.
Language: English,

MAY 17-19, 1995
Sherbrooke, Quebec, Canada
FOURTH INTERNATIONAL SYMPOSIUM ON PRESSUREMETERS
Topics: Technological developments in pressuremeters; Laboratory and field pressuremeter testing; Applications in the design of geotechnical structures.

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 FO 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 fo 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 30 - NOVEMBER 4
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, 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, 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.

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 November 1995 and final papers by February 1996.

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.

1997

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

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
Back dated issues of Geomechanics News	$ 0.50	$ 0.50	N/A

GEOMECHANICS SOCIETY PUBLICATIONS ORDER FORM

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IPENZ
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WELLINGTON NORTH

PAPERS PUBLISHED BY SOCIETY MEMBERS

The Society regularly updates a list of papers published by Society Members. The updated list is to be published in Geomechanics News once per year and used to assist in selecting a paper for the Geomechanics Award.

To assist in updating this list, if you have recently published a paper please complete the following form and post it to:

The Publications Officer
Stuart Palmer
C/- Beca Carter Hollings & Ferner Ltd
P.O. Box 3942
WELLINGTON

GEOMECHANICS SOCIETY LIST OF PAPERS PUBLISHED BY MEMBERS

AUTHOR(S) _____

TITLE _____

_____

WHERE PUBLISHED _____

WHEN PUBLISHED _____

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.

ANNUAL SUBSCRIPTION

Annual subscriptions, which include the newsletter are (for 1994)

Members	*(IPENZ members)*	*$26.00*
	(others)	*$36.00*
Students	*(IPENZ members)*	*$13.00*
	(others)	*$18.00*

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

International Society for Soil Mechanics and Foundation Engineering (ISSMFE)		*$16.00*
International Society for Rock Mechanics (ISRM)		*$16.00*
International Association of Engineering Geology	*(IAEG)*	*$10.00*
	(with bulletin)	*$32.00*

N.B. Students may affiliate to one International Society for no cost.
All subscriptions/fees are GST inclusive.

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

NZ Geomechanics Society
P O Box 12 241
WELLINGTON

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

NEW ZEALAND GEOMECHANICS SOCIETY

APPLICATION FOR MEMBERSHIP

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

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

POSTAL ADDRESS: . . . . . .

. . . . . .

Phone No:. . . . . Fax No.: . . . . . .EMAIL: . . . . . .

ACADEMIC QUALIFICATIONS: . . . . . .

PROFESSIONAL MEMBERSHIPS: . . . . . Year Elected. . . . .

PRESENT EMPLOYER: . . . . . .

OCCUPATION: . . . . . .

EXPERIENCE IN GEOMECHANICS: . . . . . .

. . . . . .

. . . . . .

STUDENT MEMBERS:

TERTIARY INSTITUTION: . . . . . **SUPERVISOR:** . . . . . .

SUPERVISORS SIGNATURE: . . . . . .

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

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

I wish to affiliate to:

International Society for Soil Mechanics and Foundation Engineering	(ISSMFE)	Yes/No
International Society for Rock Mechanics	(ISRM)	Yes/No
International Association of Engineering Geology	(IAEG)	Yes/No

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

Signed . . . . . .

Date . . . . . .

ANNUAL 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

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(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	Two Issues
Back Cover	$250*2	$400*2
Inside Cover (Front or Back)	$200*2	320*2
Full Page Internal	$180	$290
Half Page Internal	$125	$200
*Note: 1 All rates are excluding GST 2 Subject to availability		

The deadline for advertising copy for the next issue is 15 May 1995. Arranging artwork for your adverts can be carried out at a reasonable additional cost if requested. However, advance notice is required for this additional service.

If you are interested in advertising in the next issue of Geomechanics News please contact:

The Publications Officer
Stuart Palmer
c/o Beca Carter Hollings & Ferner Ltd
P.O. Box 3942
WELLINGTON

Tel: (04) 473 7551
Fax: (04) 473 7911

The Assistant Editor
Stephen Crawford
c/o Tonkin & Taylor Ltd
P.O. Box 5271
AUCKLAND

Tel: (09) 377 1865
Fax: (09) 307 0265

NEW ZEALAND GEOMECHANICS SOCIETY

NAME CHANGE QUESTIONNAIRE

Should the name of the Society be changed to reflect modern terminology?
Are the Society's activities and membership wider than the term "geomechanics" reflects?
This is your chance to indicate your preferences. Read on.

Background

The Society formed in 1958 and later became the first Technical Group of IPENZ (then NZIE). The name "NZ Geomechanics Society" was adopted in 1972. The term "geomechanics" was regularly used in those days but is hardly used today. Would someone outside the engineering profession recognise it today? "Geotechnical Engineering" is more popular these days and is understood by a wider group of people.

The objects of the Society include "the advancement of soil mechanics, rock mechanics and engineering geology", and members are required to affiliate to one or more of the following International Societies :

International Society for Soil Mechanics and Foundation Engineering
International Society for Rock Mechanics
International Association of Engineering Geology

Examples of allied groups overseas and publications are :

Australian Geomechanics Society
British Geotechnical Society
Canadian Geotechnical Journal
ASCE Journal of Geotechnical Engineering

Our last symposium held earlier this year in Wellington was on "Geotechnical Aspects of Waste Management" and attracted many participants from outside the Society and the field of engineering. Is the society covering wider interests than 20 years ago, e.g. environmental geosciences?

Should the Society have a name to reflect these changes, a name more readily recognised by the public? The Management Committee wants to hear from you.

COMPLETE THE QUESTIONNAIRE BELOW

Please indicate your membership status and preferred name from the following choices :

☐ Current member of Society ☐ Non-member

☐ Retain current name, i.e. NZ Geomechanics Society ☐ NZ Dirt Society

☐ NZ Geotechnical & Geological Engineering Society ☐ NZ Geotechnical Society

☐ NZ Ground Engineering Society ☐ NZ Geosciences Society

Other suggestions .....

The artists among you are invited to submit a logo for the Society on a separate sheet.

Please return this sheet quickly to :

Mr G B Farquhar
The Editor - NZ Geomechanics News
PO Box 4241
AUCKLAND

Post Card

Secretariat for the
International Workshop on Rock Foundation

c/o Prof. Tadashi Yamabe
Department of Civil and Environment Engineering,
Saitama University
Shimo-Ohkubo 255, Urawa,
Saitama 338,
Japan

Stamp

AIR MAIL

INTRODUCTION

It is our great pleasure to invite you to Tokyo attending both the ISRM 8th International Congress and the International Workshop on Rock Foundation.

The ISRM 8th International Congress will take place from 25 to 29 September 1995, and will be followed on 30 September by the International Workshop on Rock Foundation.

This Workshop is a one-day workshop and sponsored by the Organizing Committee for the 8th International Congress on Rock Mechanics, the Japanese Society of Soil Mechanics and Foundation Engineering and the Japan Institute of Systems Research.

Large and important structures in the field of transportation, infrastructure and energy are constructed on the rock masses. The stability and safety of their rock foundation are great concern because of their importance to the society.

This Workshop aims to provide an updated discussion on the rock foundation engineering and to have intensive direction and prospect of this matter.

We invite researchers, consultants, contractors and other engineers concerned with the rock mechanics and rock engineering. They are requested to participate in the Workshop in order to present papers, exchange experiences and meet colleagues from all over the world.

We look forward to seeing you in Tokyo in 1995.

INFORMATION

LANGUAGE
Written contributions will be accepted in English only. Oral presentation will be in English and Japanese. Simultaneous translation English-Japanese will be available.

WORKSHOP PROCEEDINGS
The individual papers comprising the Workshop will be published in a Volume of Proceedings before the Workshop starts.

TIME SCHEDULE

·Abstract Submission	1 August 1994
·Paper Acceptance	1 September 1994
·Full Paper Submission	1 March 1995

ADVISORY COMMITTEE
- ·Kawamoto, T., Emer. Prof., Nagoya University
- ·Miki, K., Dr., Vice President, Kawasaki Geological Engineering Co. Ltd.
- ·Sakurai, S., Prof., Kobe University
- ·Tatsuoka, F., Prof., Institute of Industrial Science,University of Tokyo
- ·Yamagata, M., Senior Technical Advisor, OECF

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SECRETARY
Prof. T. Yamabe / Ms. H. Saito
Department of Civil and Environment Engineering, Saitama Univ.
Shimo-Ohkubo 255, Urawa, Saitama 338, Japan
Phone +81-48-858-3544 / 3540
Fax. +81-48-855-9361

PROGRAMME

The Workshop will focus on the rock foundation of the large-scaled structure, for example, the long span suspension bridge, nuclear power station, dam, large marine structure and high steel tower.

WORKSHOP THEMES
1. Investigation and Testing of Rock Masses
2. Evaluation of Deformability and Strength of Rock Masses
3. Design and Analysis of Rock Foundation
4. Construction and Measuring of Rock Masses

The following topics are suggested for papers and discussions.
- – Bearing capacity of rock foundation
- – Deformation or settlement of rock foundation
- – Dynamic or seismic behavior of rock foundation
- – In-situ test or laboratory test of rock masses and rock specimens
- – Rock mass classification and scale effect
- – Discontinuity and anisotropy of rock masses
- – Stability or deformation analysis of rock foundation
- – Construction method and measuring of rock foundation including back analysis
- – Improving and reinforcing treatment of rock masses

PLACE & DATE
Place: Nippon Convention Center (Makuhari Messe)
2-1, Nakase, Chiba 260, Japan
Date : Saturday, 30 September 1995 9.00-17.00

CALL FOR PAPERS
Authors are invited to submit original papers giving high level contributions to the scientific and technical aspects of the Workshop.
Prospective Authors are requested to announce their intention and to submit a 200 word abstract, which will constitute the basis for preliminary acceptance by the Organizing Committee. Some selected papers are given time for oral presentation at the Workshop.
Notification of acceptance and instructions to Authors will be issued in due course.

SPECIAL LECTURES
Special lectures on the worldwide topics of large-scaled structures by guest lecturers are given at this Workshop. Their papers will be included in the proceedings.
1. Foundation Engineering of Honshu-Shikoku Connecting Bridges
2. Rock Foundation of Nuclear Power Stations on Soft Rock and Discontinuous Rock
3. Foundation Engineering of Large Offshore Structure in North Sea
4. Rock Engineering of Dam Foundation

ANNOUNCEMENT AND CALL FOR PAPERS

International Workshop on

ROCK FOUNDATION

of Large-Scaled Structures

30 September 1995, Tokyo

ISRM 8th International Congress on Rock Mechanics
25-30 September 1995, Tokyo

Sponsored by
The Japanese Society of Soil Mechanics and Foundation Engineering

Organizing Committee for the
8th International Congress on Rock Mechanics

Co-Sponsored by
The Japan Institute of Systems Research

Preliminary Registration

Those who are interested in submitting a paper are kindly requested to fill out and return the Preliminary Registration Card to the Secretariat as soon as possible.
A 200 word abstract of your paper should be sent to the Secretariat by 1 August 1994.

International Workshop on Rock Foundation
30 September 1995, Tokyo
Preliminary Registration

(Please type or print in block letters and check the appropriate boxes.)

☐ I intend to participate in the Workshop.
☐ I intend to submit a paper. Proposed title of the paper is :

Name and Address :
☐Prof. / ☐Dr. / ☐Mr. / ☐Ms.
Name :
Organization (Institution) :
Department :
Mailing Address :
Country
Phone : Fax. :
Message to Secretary :

GEOTECHNICAL ENGINEERING SOFTWARE AND EQUIPMENT

SLOPE - *Slope Stability Analysis Program*

A program for analysing the stability of circular and no circular slip surfaces on natural and man made slopes.

Slope models up to 9 different soil layers with multiple water tables or piezometric surfaces, submerged slopes, tension cracks, surcharges and anchor loads. Earthquake forces can also be modelled as equivalent static forces. An optional extension to the program offers Reinforced Earth analysis and Design.

WALLAP - *Retaining Wall Analysis Program*

Wallap analyses the stability of cantilevered and propped retaining walls including sheet pile and diaphragm walls. It also has a single pile option.

Factors of safety are computed according to standard codes of practice while all displacements and bending moments are calculated by finite element analysis which models the actual sequence of construction of the wall. Earth pressures are calculated automatically from basic soil properties.

The program models up to 20 different soil layers with hydrostatic or non linear water pressure distribution, surcharges, struts and anchors.

AUGERS AND PENETROMETERS

The penetrometer has a 1.5m probe, case hardened threads and complies with NZS4402.

The augers' come in different configurations and have either 55mm or 80mm heads.

There is a comprehensive range of options for the augers including Shear vane kit.

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If you would like demonstration software, brochures or further information on these programmes or on the equipment please contact Martin Hewitt at:

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