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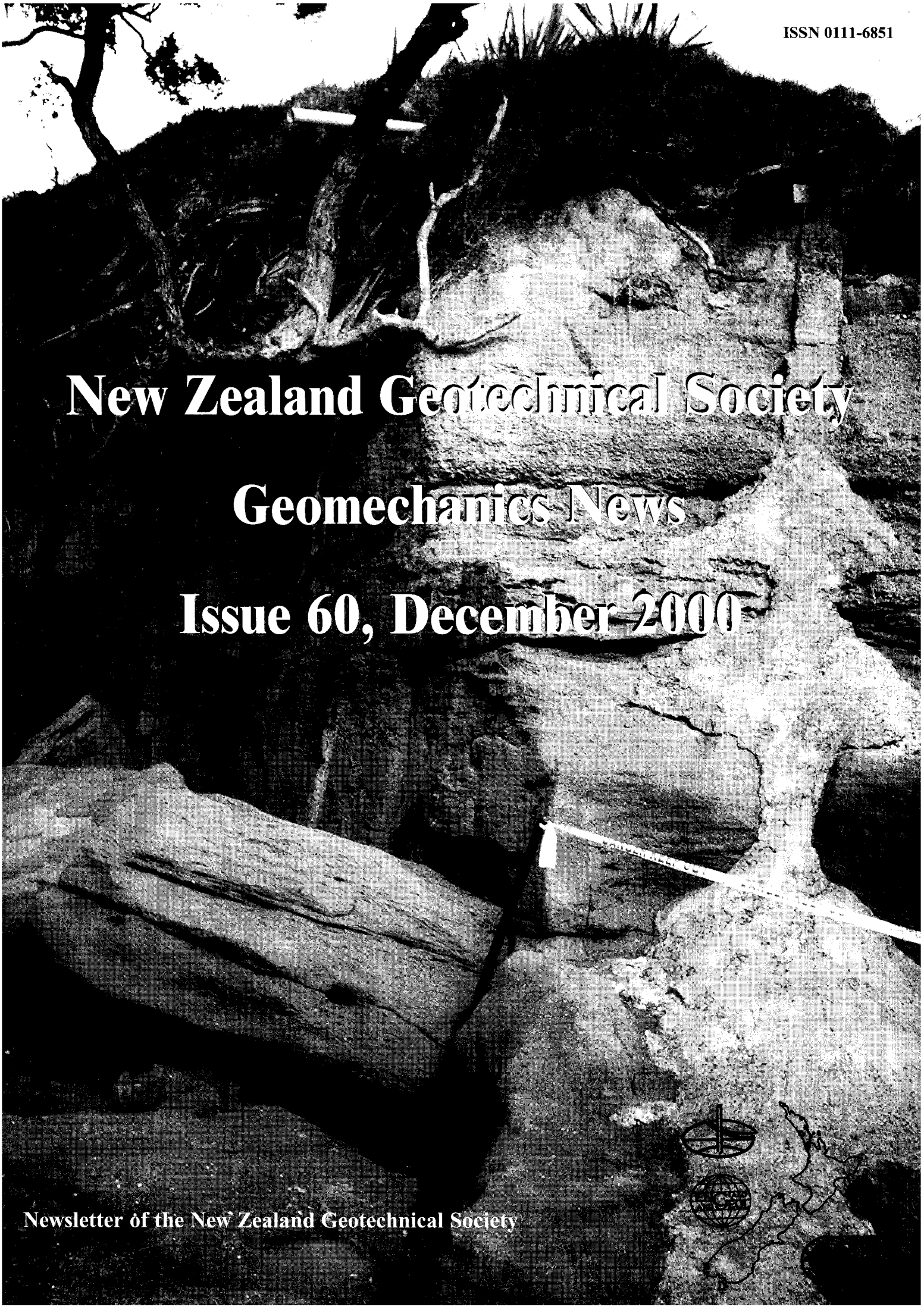


New Zealand Geotechnical Society

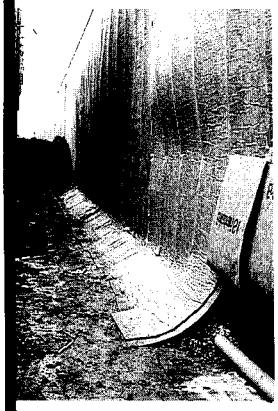
Geomechanics News

Issue 60, December 2000

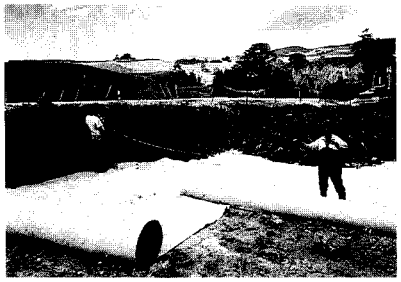
Newsletter of the New Zealand Geotechnical Society

INNOVATIVE SOLUTIONS

If it's related to retaining, erosion control, drainage, poor subgrades or pavement resealing — we have the solution for you.

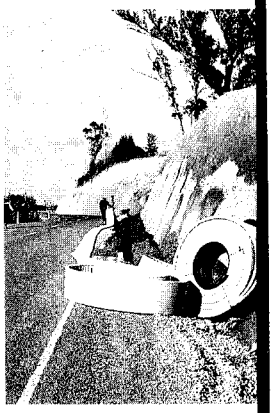


ENKADRAIN
walls, basements, roof gardens, turf drains



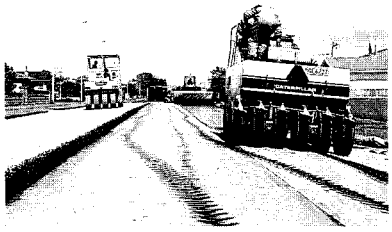
BIDIM
roading & sub-soil fabrics

GABIONS & RENO MATTRESSES
TERRAMAT
erosion & re-veg. biomat



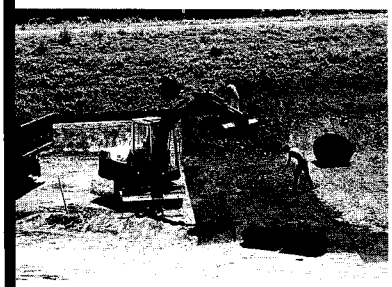
MEGAFLO
roadside, turf, subsoil drain

MACCAFERRI NEW ZEALAND



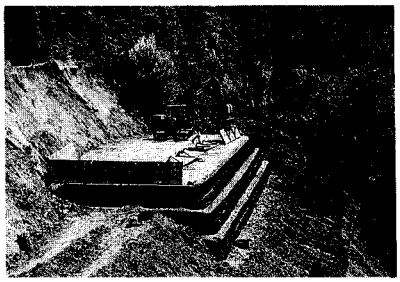
BIDIM Paving Reseal fabric & HATELIT grids for asphalt and slurry seals

STABILENKA
embankment stability and COLBONDDRAIN vertical wick drains for consolidation of soft soils



ENKAMAT
erosion mat for grass, floodways, spillways, channels, stopbanks

FORTRAC geogrid for subgrade/ sub base and slope reinforcement



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- ☐ SARMAC Protection of submerged pipelines
- ☐ GEOMAC Soil containment system
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- ☐ STABILENKA Embankment stability of soft soils
- ☐ COLBONDDRAIN Consolidation drainage
- ☐ HATELIT Grid for asphalt/slurry seal
- ☐ BARRIER FENCE Site safety

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Company:_____
Address:_____
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New Zealand Geomechanics News

Number 60 December 2000

Contents

Chairmans Corner 2

Editorial 3

Letters to the Editor 5

Report from the Secretary 6

International Society Reports 7

Laurie's Brain Teaser (No. 3) 8

NZGS Branch Activities
- Auckland Branch 9
- Wellington Branch 10
- Canterbury Branch 10
- Waikato/BOP Branch 10

New Zealand Geotechnical Society Information 12

Conference Reports 15

Standards, Law & Industry News
- AS/NZS Commentary 19
- ASFE/NZ? 20
- Dirt on the Net 22

Book Reviews
- Proceedings of the International Symposium on Slope Stability Engineering, *by Paul Finlay* 23
- Advances in Aggregate and Armourstone Evaluation, *by Doug Johnson* 25

Special Interests
- Numerical Analysis in Soil Mechanics, Part 2, *by Sergei Terzaghi* 27
- Tredbo Coroner's Report, *by Grant Murray* 30

The Bob Wallace Column 32

Project News
- Pavement Imaging Using GPR 33
- UV/Filter Piling - use of Hydraulic Hammers 35
- Polyrock by James Hardie 38

2001 Photo Competition 40

Technical Articles
- Coping with Subsurface Risks:Bidding Practices that Work, *by Richard Donnelly* 41
- Auckland Residual Soil - Compressibility Measurement 49
- Quantitative and Non-Quantitative Methods of Estimating Slope Stability, *by Laurie Wesley* 55
- Land Risk Management Concepts and Guidelines, *by Australian Geomechanics Society, Sub-Committee on Landslide Risk Management* 61

Chairman's Corner

LIMIT STATE DESIGN SEMINARS

It is very pleasing to be able to report that the two Geotechnical Society-sponsored seminars presented by Professor Michael Pender in Auckland and Christchurch during August proved extremely successful. Participants numbered approximately 80 in Christchurch and 160 in Auckland, and included members of both the Geotechnical and Structural Societies of IPENZ.

It was particularly pleasing to see the large number of out of town registrants, the cost to whom would have been very considerable to attend. Judging by the keen interest shown, it is clear there is a strong desire by our members to participate in similar professional development seminars in the future. The management committee would therefore welcome any comments on ideas for similar seminars.

NZGS 2001 SYMPOSIUM

It is also pleasing to report that work is well under way with planning for our 2001 Symposium in Christchurch, the title of which is "Engineering and development in hazardous terrain". As the title states, the Symposium is intended to encapsulate the challenges of investigation design and construction of all types of development projects encompassing the range of hazards experienced in New Zealand. This includes slope instability and erosion, flooding, seismic shaking and faulting, volcanism, and man made hazards arising from contamination of the environment.

The organising committee led by Kevin McManus of Canterbury University has identified a number of theme presenters for each of the main themes, and it is intended that the 2001 Geomechanics lecture will be presented at the Symposium. An invitation has been issued to the Australian Geomechanics Society for its members to participate, and it is very much hoped that our trans Tasman colleagues will be with us.

The Symposium will include trade exhibitions and field trips to visit development projects on the spectacular SH73 Arthur's Pass highway including the recently completed viaduct project and Candy's Bend reconstruction, and major debris flow protection structures at the Mount Cook tourist village. Planning has also started (yet to be confirmed) for a joint Australia-NZ International Society of Engineering Geology (IAEG) workshop preceding the Symposium.

BUILDING ACT SUB COMMITTEE

Under the guidance of Paddy Luxford of Babbage Consultants Ltd in Auckland, a sub committee has been convened to review Section 36(2) of the Building Act. As most practitioners are aware, Section 36(2) allows for a territorial authority to instruct the land registrar to "tag" a property title to the affect that the land is subject to a particular hazard such as land instability.

Such a tag relieves the territorial authority and its officers of professional liability in the event of the hazard being realised, but has significant implications for the property owner with respect to insurance and possible re-sale value. Most of us are aware of the inconsistencies being applied by territorial authorities surrounding this section of the Building Act and it is timely that the Society formulate a policy statement on this topic.

The sub committee has raised the possibility of a private member's bill being put before parliament once an agreed policy statement has been formulated. The Office of the Parliamentary Commissioner for the Environment is also presently reviewing the policy framework of the Resource Management Act 1991 and the Building Act 1991 in relation to building on or close to active faults. This has arisen out of recent public concern with the manner in which Section 36(2) of the Building Act has been applied in respect to a development proposal crossing an active fault hazard.

If any our members is interested in participating with the section 36(2) sub committee, in the first instance contact either the Chairman or the Secretary.

GEOENG2000

As your Chairman, I have been invited to chair one of the technical sessions at GeoEng2000 in Melbourne. It is extremely pleasing to see three of our members, John Berrill, Mick Pender and Trevor Matuschka, each of whom are presenting theme talks at this major southern hemisphere geotechnical forum, while Bruce Riddolls is also chairing a session. I look forward to meeting other NZGS members who will be attending what will be without doubt an exciting event.

SUBSCRIPTION COSTS

Unfortunately we are facing rising costs from a number of fronts, and it is my unpleasant duty to inform you that I will be seeking consent from members at the AGM next year to raise our subscriptions.

Whilst subs were last raised only two years ago, cost increases are arising due to the weakness of the NZ dollar and the impact this has on affiliation fees to international societies. We also face additional costs to support a new Vice President for the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE).

Our subs still compare very favourably when compared with other similar organisations, and given the number of activities carried out on behalf of members, I am of the firm opinion that our subs are very good value.

INCORPORATION

It is with a lot of frustration that I can report we are making slow progress towards the management committee's intention for the Society to become an incorporated organisation. Despite two mail outs and personal calls by the management committee to a number of silent members, we are still short of the required number of votes to achieve the required majority.

I wish to strongly urge all members who have not yet voted to please do so by returning your voting forms to the Secretary or alternatively, contact the Secretary to obtain a voting form if you do not have one.

I wish you all an enjoyable and relaxing Christmas break, and a prosperous New Year.

Guy Grocott

Editorial

I was very pleased to see such a flood of letters coming into my post bag following the first issue of our new look NEWSLETTER. I was very politely, but firmly, reminded that we can't call the GNews a journal until we have the papers refereed. It's hard enough getting articles out of members anyway without the threat of a referee's sharp eye for technical flaws or inconsistencies to worry about.

It would be like having your work Peer Reviewed and apparently nobody likes that.

In this issue you will find the usual items – Laurie's Brain Teaser, Book Reviews and Society News. It is great to see the enthusiasm and support the society has through the various branches. The co-ordinators should be given every encouragement and the best way to do this is to make every effort to attend a Branch Meeting.

Other interesting items to look at include an article presented by the ASFE. I am looking forward to the reaction of the membership and the market place to their proposal. Presumably the NZGS Committee, IPENZ and ACENZ will be similarly interested.

In the Special Interest section we have the second instalment from Sergei Terzaghi on advanced analytical techniques and a discussion item I prepared based on my reading of the Thredbo Coroner's Report. Hopefully that serves as an appetiser for the main course in the Technical Articles. It is the stated intention of GN to appeal to as wide an audience as possible so if you have a special Geo-science issue that you want to discuss please contact me by phone or email. I would be particularly keen to extend our fields of interest into Geo-environmental and Groundwater / Hydrogeology.

A couple of the technical articles presented in this issue have been published previously elsewhere but I felt they were valuable items that should be brought to the attention of the NZGS membership. Richard Donnelly's articles will be presented in two parts and will continue a Dam Theme that is likely to become a common thread over the next two or three issues. I didn't have room for Robin Fell's EH Davis paper but it is a landmark reference on Dams and should be on every practitioner's shelf and therefore is something to look forward to in the next issue.

The lessons learned from the Thredbo Disaster will not be soon forgotten and the AGS is demonstrating a serious commitment to spreading and improving the knowledge base on slope hazard management. I am very pleased to include their guidelines published earlier this year and look forward to hearing how you think they may apply in New Zealand. As an introduction we have an interesting little discussion paper from Laurie Wesley on the merits of Quantitative Risk Assessment for slopes.

To complete the technical articles for this issue, one of our regular stalwarts and a fine supporter of the GN presents the University of Auckland's most recent findings from the FORST research programme.

Finally, why not dust of those old project Scrapbooks and photo albums that clutter up your offices and archives to find an entry for this year's photo competition. There is some good money to be won and I know there have been some classic failures so don't be shy. And just for clarification – a geotechnical failure does not include that Year 4 tutorial question from John Berrill that you totally flunked.

Grant Murray
Editor.

Email *GMurray@skm.co.nz*

Editorial Policy

NZ Geomechanics News is a newsletter issued to members of the NZ Geotechnical Society. It is designed to keep members in touch with recent developments within the Geo-Professions both locally and internationally.

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

The editor's team is happy to receive submissions of any sort for future editions of *NZ Geomechanics News*. The following comments are offered to assist potential contributors:

Technical contributions can include any of the following:

- technical papers which may, but need not necessarily be, of a standard which would be required by the international journals and conferences
- technical notes
- comments on papers published in *NZ Geomechanics News*
- descriptions of geotechnical projects of special interest.

General articles for publication may include:

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

Submission of text material in camera-ready format is not necessary. However, typed copy is encouraged particularly via e-mail to the editor or on floppy disk. We can receive and handle file types of almost any format. Contact Grant if you have a query about format or content.

Diagrams and tables should be of a size and quality appropriate for direct reproduction. Photographs should be good contrast black and white gloss prints and of a suitable size for mounting to magazine format. *NZ Geomechanics News* is a newsletter for Society members and papers are not necessarily refereed. Authors and other contributors must be responsible for the integrity of their material and for permission to publish.

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Letters to the Editor

Dear Editor,
Marvellous new format. Well done! I especially liked the new paper and the standard text applied throughout – it makes it look that little bit more professional. We could do with some more news items on projects. Perhaps some new projects that are just starting up, or in the pipeline, and project descriptions that have finished and gone well (or badly!). In our academic world we miss out on all that fun. I'm looking forward to the next issue.

Prof Martin Penhaligon, University of Aotearoa

Dear Editor,
So, the Geotechnical News is starting to look a bit flash. I preferred the old style. It had a more earthy, practical feel and we weren't full of our own self-importance. Why do we always have to try and improve things? It rarely works. Anyway, I reckon I could do just as good a job with my old John Bull Printing Set and it would cost a lot less I can tell you. Perhaps the membership wants to think about how much of their good money is being wasted on the newsletter.

And another point – Who is this chap Bob Wallace? Is he even a member? I don't know him so why does he think he is qualified to talk about ethics and peer reviews?

Glen Westfield, Optimistic Consultants International

Dear Editor,
Not a bad issue for your first effort but I think you will find it hard to maintain that standard. I predict that you will struggle to get regular contributions from interesting or informed geo-professionals. I expect you will end up pestering the same people time and again to produce items that eventually become boring and repetitive.

Fortunately, I am very busy on a lot of very important and very large projects. In fact, I am so busy all of the time that I can barely spare five minutes to flick through the journal anyway. Perhaps you could put more pictures in it so I wouldn't have to read so many words.

Derek Connor, Bettar Connor & Haslotsov Feez

Dear Editor,
I must write to take exception with that clever clogs Sergei Terzaghi. I do not think the Geomechanics News should be wasting it's time printing articles about that Finite Element and Constitutive Model nonsense. What use is that ever going to be in New Zealand? From my perspective it is totally irrelevant.

If I suggested to any of my clients that we needed to spend loads of money on that computer rubbish I would not have many clients left. They pay me to be a consultant not to do any of that complex analysis that they could never begin to understand anyway.

The Geomechanic News should stick to basic, solid traditional articles that we can all use and understand. That other Terzaghi fellow wrote some interesting stuff. Why can't we get him to put something in?

Paul Mitchell, Soldier & Sailors

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Report from the Secretary

Society membership is currently flourishing with a total of 460 members. There have been a large number of new members that have taken up the offer from the Ultimate Limit State Design Seminar. We welcome you all.

New Members

It is a pleasure to welcome the following new members into the Society since the last issue of Geomechanics News: -

C Krumdieck
C Manktelow
C Thelin
D Oakley
D Reid
E Ladley
E Lust
G Kell
G Littler
G Thompson
G Twose
H Avdjiev
H Wick
I Stuart
J Boersen
J Dale
J Fulton
J Ko
J Wood
K O'Rouke
M Hedley
M O'Brien
M Smith
M Taylor
N Speight
N Taylor
R Bailey
R Dick
R Gibson
R King
R Knowles
R Mutton
R Osborne
R Perry
R Sullivan
S Anwar
S Moon
T Simpson
W Syme

Resignations

E Hudson-Smith, C Simpson, J Rutledge, R Seyb, A McMenamin, G Shaw, S Stewart and H Turnbull have tendered their resignations from the Society.

Subscriptions

Invoices will be sent in late October/early November for the 2000/2001 financial year subscriptions. Subs have not been increased however it is an item to be discussed at the forthcoming AGM.

Society Incorporation

Gaining enough votes to represent the majority of the membership to approve the motion to become incorporated is going very slowly. We are still well short of the required number. If you have not yet voted please do so.

Debbie Fellows
Management Secretary

International Society Reports

ISRM

INTRODUCTION

This report covers ISRM business for the period April 2000 to October 2000. The majority of the Board's activity has been by email. The next Board meeting will be held in Melbourne at GeoEng2000 on 18 November, 2000. A combined Board meeting with ISSMGE and IAEG will also be held on this day. The next Council meeting will be held in Melbourne on 19th November.

COMMISSIONS

At this time I have not received any information regarding possible new Commissions.

INTEREST GROUPS

At this time I have not received any information on firm proposals for forming new interest groups. Please contact me if there is any interest in forming an Interest Group.

2001 ISRM SYMPOSIUM

2nd Asian Rock Mechanics Symposium will be held in Beijing, China on September 11-14,2001. Date for submission of abstracts is now past. The address and contact are as follows.

Address:
Institute of Geology and Geophysics,
Chinese Academy of Sciences,
P.O.Box 9825
Qijiahuoji,
Beijing 100029 CHINA
Fax +86 10 62040574
E-mail egml@igcas.igcas.ac.cn
Web. Address : http://isrm2001.homepage.com
Contact: Prof. Wang Sijing, Chairman of Organizing Committee, Prof. Fu Bingjun, Secretary General

ROCHA MEDAL

The selection of the next Rocha medallist will take place at the next Board Meeting at GeoEng2000 in Melbourne. No nominations were received from New Zealand or Australia.

ISRM NEWS JOURNAL

I would greatly appreciate receiving articles for the ISRM News Journal.

XI ISRM CONGRESS, 2007

Letters of intent to hold this congress have been received by the Italian Geotechnical Society (Torino) and Portuguese National group (Lisbon).

Associate Professor Chris Haberfield
ISRM Vice-President for Australasia
Department of Civil Engineering
Monash University
Clayton, 3800

ISSMGE

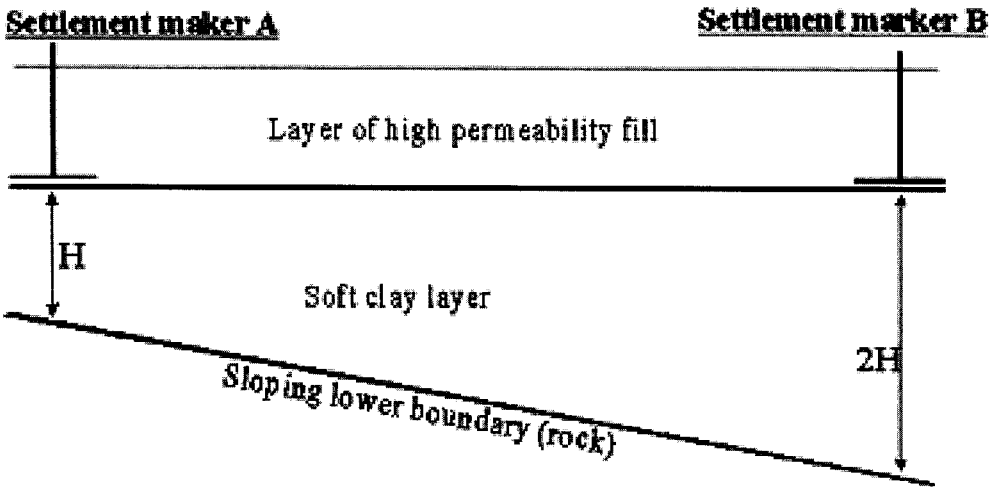
No report received.

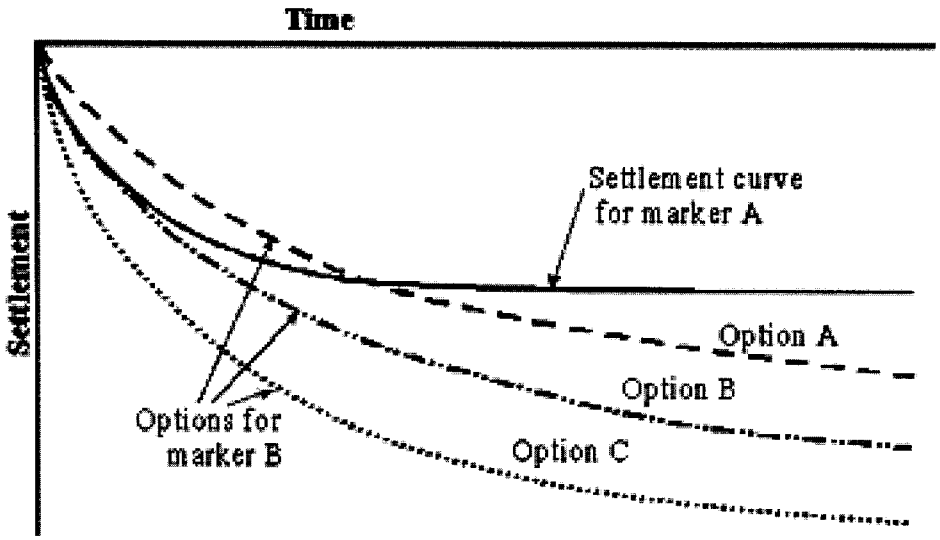
IAEG

No report received.

Laurie's Brain Teaser (No. 3)

The diagram below shows a soft clay site, over which a layer of fill of constant thickness is to be placed. The soft clay layer varies in thickness across the site, being twice as thick at one end as the other. A settlement marker (A) placed at the shallow end yields the record shown in the figure. Which of the three other curves shown would you expect to obtain from settlement marker B, at the deep end of the site?





Answer to Brain Teaser No 2 (June, 2000)

I am not sure about the answer to this question. The theoretical answer is easy - the rate of flow through the double layer is over twice that through the single layer. According to conventional calculations, the flow rates are as follows:

Single layer:	1.2×10^{-8} m³/sec/m²
Double layer:	2.67×10^{-8} m³/sec/m²

This seems surprising; the explanation is that by placing the additional 1m thick layer below (or above) the 0.5m layer, the head driving seepage through the layers is more than doubled, while the resistance to flow is increased by a much smaller amount.

Whether this theoretical reality should dictate the choice of the lining design is open to debate. My understanding of leakage through lining layers is that it is often associated with defects in the layers, and the thinner the layer the greater the likelihood that such defects will control the performance of the layer. With thicker layers, formed by compaction in a series of lifts, the likelihood of defects controlling performance is greatly reduced. Hence a case can certainly be made for using the double layer in preference to the single layer. Readers might like to comment on this issue.

NZGS Branch Activities

Auckland Branch Activity Report

The middle part of the year has been filled with a number of excellent presentations to the Society in Auckland. All of these have been very well attended showing a pleasing support for the program of talks by the society. The topics are summarised briefly below for those who missed the meetings.

Section 36(2) **5 July**

Five presenters provided brief talks on this particular clause in the legislation that has come to prominence after the results of recent court cases. These speakers provided an excellent range of viewpoints on this contentious topic. The general consensus from these discussions was the need for further work to assist with the application of the legislation.

A more detailed summary of this meeting by Debbie Fellows is presented later in this report.

Project Manukau **26 July**

Presenter	
Phillip Clayton	Beca Carter Hollings & Ferner

Phillip provided an excellent overview of the expansion of the Mangere Wastewater Plant. He went on to discuss some of the specific geotechnical aspects of the project and the solutions that were arrived at for some of the problems encountered.

Of interest was the wide range of conditions encountered, from soft sediments to volcanic soils to basalt rock. A wide range of geotechnical problems were faced and resolved including groundwater inflow in basalt rock and foundations for settlement sensitive structures. Some time was spent on the detail of the solution for the clarifier tanks, where settlement/rebound was a significant issue. This was resolved using a sophisticated monitoring system to confirm modelling predictions.

Planning Strategies **August**

Presenter's	
Noel Reardon	Auckland Regional Council
Bruce Harland	Manukau City Council

These two presenters talked to the society on planning issues facing Auckland. Noel Reardon spoke about the regional growth forum and the results of the work involved with this. He outlined the prediction for future growth of Auckland and the proposed strategies to minimise worsening transport problems and to confine the ongoing urban sprawl.

Bruce Harland spoke about the more detailed planning problem of development in the East Tamaki area. He provided some very interesting insights into the thinking behind the planning process and the drive toward more high density housing in this area.

Natural Hazards **30 August**

Presenter's	
Tam Larkin	University of Auckland
DV Toan	Beca Carter Hollings & Ferner
Phillip Shane	University of Auckland
Michele Daly	Auckland Regional Council

These four presenters gave presentations on the recent natural hazard mapping study undertaken by the Auckland Regional Council and others over the last few years. Michele Daly of the ARC gave an overview of the study and the origins of the work. Particular details of the study were highlighted by the other speakers with Tam Larkin on the seismic hazard mapping, DV Toan on liquefaction and instability, and Phil Shane on volcanic hazards.

This talk highlighted the depth of information available documenting risk areas in the Auckland region.

Pisa **20 August**

Presenter	
Laurie Wesley	University of Auckland

Laurie Wesley recently visited Pisa and toured the works currently underway to stabilise the towers ongoing movement.

Laurie used this background to provide a detailed and enlightening talk on the history of the building and development of the problems that have led to the closing of the tower and the start of the remediation program. Of particular interest was the historical trends showing the influence of various works on the building, along with settlement of adjacent buildings.

Laurie also described the precision soil removal that is currently being used to stabilise the tower which is showing dramatic results in reversing the trend of increasing rotation.

Future Talks

We have several presentations of exceptional quality to round out the year. These will include a presentation from Grant Murray (Sinclair Knight Merz) on "The New Zealand Landslide Safety Net" as well as two major speakers from the GeoEng2000 conference who will be travelling here in November to give their GeoEng2000 presentations to the Society. Bob Schuster will present his Plenary Session lecture on "Dams on Pre-existing Landslides", while Professor Alan McGowan will present his GeoEng2000 Mercer Lecture on "A Reassessment of Geosynthetic Reinforcements for Soil Structures"

If you have a potential topic or wish to present something to the Society in Auckland please do not hesitate to contact me.

Chris Bauld
Tonkin and Taylor Limited (09) 355 6000

Waikato/Bay of Plenty Branch Activity Report

On 23rd August, Grant Murray presented his paper on "The New Zealand Landslide Safety Net" to the Waikato/BOP branch of the Geotechnical Society. About 12 people attended (allegedly a good result) with the room provided free by the University of Waikato and drinks/food provided by Geotechnics Limited - Tauranga.

The talk was very informative for all that attended. It was also great to hear a quality presentation without having to get your calculator out and the talk was easily understood by all who attended (which also included some students). I am sure that many people gained a useful insight into how the EQC works in the case of natural disasters.

If any other members from around the country are in Hamilton or Tauranga and could spare an evening to give a small presentation to the local branch or has any suggestions for future topics, please feel free to contact me.

Paul Burton
Geotechnics Limited
(07) 544 4910

Wellington Branch Activity Report

The Wellington Branch has had a busy year. Talks since June have included:-

- Bruce Symmans from Tonkin & Taylor on 13 June presenting his talk on slip remedial works in the Aro Valley, Wellington.
- Peter Foster from OPUS on 18 July talking about the Taiwan earthquake. This was held as a joint talk with the local NZSOLD branch. Despite this the turnout was very disappointing.
- Graham Hancox from IGNS on 22 August talking about earthquake induced landslides and the Mt Adams landslide in South Westland.
- A discussion about anchors primarily in weathered greywacke on 26 September. A series of wall charts was used to present various aspects of anchor design and to promote discussions. Twenty people turned out for this talk which is a record for Wellington so I will try to arrange a similar discussion type meetings in the future.

In addition we have two or three more talks planned before the end of the year.

If any one has suggestions on future topics, particularly on the discussion type meetings, please let me know want you want and I will try to organise it.

Ian McPherson
Connell Wagner Limited
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Otago Branch Activity Report

On 15 November at 5.30pm the Otago Branch will hear a presentation on the geology and engineering of the Fairfield By-pass by Phil Glassey of IGNS and Graham Salt of Tonkin and Taylor Limited. This will be followed by a visit to the site.

For Further Information Please Contact:
John Henderson
City Works
(03) 477 6363

Canterbury Branch Activity Report

On 21 October Canterbury were thoroughly beaten by Wellington in the NPC final. This has obviously caused much grief and suffering in the region. I am sure I can speak for all of the regions when I extend my heartfelt sympathies to the Canterbury Branch during this time of great loss. We are all looking forward to receiving their next branch report.

Building Act Section 36(2)

In July the Auckland Branch of the Geotechnical Society held a panel discussion on the Building Act and particularly Section 36. This meeting was prompted by a recent court decision (Auckland City Council v. G Logan) which highlighted the application of Section 36 notices on land titles. The meeting was well attended by a vocal group of members with numbers exceeding 100.

The Panel included 2 lawyers, 2 local body representatives and 2 consultants. They were:-

Mike McQuillam	Auckland City Council
Padraig McNamara	Simpson Grierson
Dick Cobb	Rodney District Council
Peter Leman	Phillips Fox
D V Toan	Beca Carter Hollings and Ferner
Tim Sinclair.	Tonkin and Taylor

The decision in the High Court in March this year was with regard to a property in Auckland. The owner applied to the council for a consent to put in 2 dwellings and 3 carports on the site. It is a flat site with an overland flow path for stormwater. The application was refused and the owner appealed to the BIA who overturned the ruling. The council had the BIA ruling quashed in the court and the case went to the Court of Appeal. This case has already been discussed in issue No 59 of the Geomechanics News issued in June this year and will not be repeated here.

The panel presented some of the implications of the ruling, which are as follows:-

- Section 36(2) does not depend on the Council granting a waiver or modification of the building code.
- Section 36(1) and 36(2) should be read sequentially and the "or" actually means "and". So as a practitioner it is necessary to apply Section 36(1) and if this does not apply use section 36(2)
- Section 36(1) is to protect the land as well as the building work.
- A common sense approach should be used as to whether land is subject to or likely to be subject to hazard. For instance, in this case the application of the terminology meant that the whole site was flooded and the application of Section 36(1) was invoked.

The panellists felt that there were a few questions unanswered. These included:-

- Use of the terminology "land on which building work takes place." Does this mean the "whole site" for a large 100 acre property or a portion of it? How to apply this for large properties compared with suburban properties.
- What is meant by "adequate provision" to protect the land and building work? The High Court considers this a question of fact for the Council or the BIA. The protection of land may involve the elimination or containment of the hazard.
- What is meant by "intimately in contact"? Is it the amenity, ingress or egress.

Another interesting planning type question was raised regarding the building restriction line put onto a property on a cliff top. Does the restriction line get moved if the cliff top moves? Of course, no answer was immediately available for this question.

A lawyer on the panel quoted that the assessment of stability now and in the future is a "mine field for engineers". He felt that that the engineer is generally better served if 36(2) is put on the title. Such an action, however, can halve the value of a property. It also causes insurance problems as the property becomes "uninsurable" and insurers have a poor understanding of the ramifications and the possibility of no cover in the event of section 36(2). All panellists felt that the common sense approach is important.

Where to from here?

A number of Territorial Authorities are preparing their own guidelines or practice notes.

The enthusiastic input from Society members at this branch meeting suggested that the Society should also have a role and that perhaps we should also have a guideline. As a result a working group of Society members has been set up to consider the preparation of as guideline and documentation suitable for presentation to parliamentary bodies to lobby for change in the Building Act. Paddy Luxford of Babbage Consultants in Auckland currently chairs this group.

Congratulations to the Auckland Branch Co-ordinator for organising a lively and well attended meeting.

Debbie Fellows
URS (NZ) Limited
Formerly Woodward Clyde (NZ) Limited

New Zealand Geotechnical Society

OBJECTS

(a) To advance the study and application of soil mechanics, rock mechanics and engineering geology among engineers and scientists.
(b) To advance the practice and application of these disciplines in engineering.
(c) To implement the statutes of the respective international societies in so far as they are applicable in New Zealand.

MEMBERSHIP

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

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

ANNUAL SUBSCRIPTION

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

Basic membership subscriptions, which include the magazine, are:

Members ***$67.50***
[IPENZ members receive a $15 rebate on their IPENZ subscription for belonging to the society]

Students ***$28.10***
[IPENZ student members receive a $7.50 rebate on their IPENZ subscription for belonging to the Society]

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

International Society for Soil Mechanics and Geotechnical Engineering	***(ISSMGE)***	*$22.00*
International Society for Rock Mechanics	***(ISRM)***	*$28.50*
International Association of Engineering Geology & the Environment	***(IAEG)***	*$21.00*
	(with bulletin)	*$70.00*

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

NZ Geotechnical Society, P O Box 12 241, WELLINGTON

Note:
Members of IPENZ now receive their discount on society fees $15 for members, $7.50 for students) directly on their IPENZ subscription.

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

NEW ZEALAND GEOTECHNICAL 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.:** ..... **E-MAIL:** .....

DATE OF BIRTH: .....

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

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

Signed.....Date...../...../.....

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

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

Signed.....Date...../...../.....

(for office use only)

Received by the Society .....

Recommended by the Management Committee of the Society .....

Approved by the Council of the Institution .....

New Zealand Geotechnical Society Management Committee Address List 2000

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Conference Reports

Ultimate Limit State Design of Foundations Seminar
Professor M.J. Pender, University of Auckland
18 and 19 August 2000

Reporter: Simon Woodward, Geotek Services Limited

It is not the intention of this review to recycle the detail of the course, because we were supplied with comprehensive notes, well laid out although perhaps a little surprisingly, not in the same order as the presentation. For detail, attendees should refer back to the course notes. Those that didn't attend, should date, marry or rob an attendee.

The fact that this was expected to be a significant State of the Art Review was confirmed by the full turn-out, from the wide-eyed novice to the calc-encrusted old-timer. An initial polling indicated that there was a ratio of approximately 2 to 1, of Structural to Geotechnical Engineers. The unkind geotechnical observer might figure that not as many soils engineers needed to attend, but I'm sure the ratio was merely a reflection of the fact that structural engineers are a more prolific breed. As it turned out, it was also a pretty BIG PLUG for MathCad, and who knows, if there were enough converts, maybe the price might drop.

Prof. Pender issued any early call for extensive interaction between audience and presenter, though if at least one of us had been a bit less vocal, he might not have been targeted for this review. Nevertheless, that open format did allow for a full exchange of ideas, and I for one came away the richer for it.

The course commenced with first principles of failure conditions, with particular emphasis on the fundamentals of Total Stress vs Effective Stress, because without comprehending those, it is difficult to progress with further understanding. A good tip of Mick's for the uncertain practitioner was to draw the problem and then draw some free body diagrams, to define the loads in action.

In his presentation, Mick started his approach to these fundamentals from the perspective of slope stability, which, as he showed it, was particularly appropriate, but I suspect could have been an approach that might have scared many of the structural engineers half to death. I say this because, in my years of working with these colleagues, more than once I have received comments suggesting that as a craft, the term "geotechnical" could be substituted for by "witch". However, for those structural engineers wishing to extend their comfort zone, it is worth spending some time "on the dark side". Not too much though, there's enough competition out there as it is.

Suffice to say, for those still unsure (and hoping that structural engineers are both receiving and reading this publication) Total Stress conditions relate to the short term response time for soils. Conversely, Effective Stress conditions relate to the long term response time for soils. To muddy that picture a little however, bear in mind that "long term" for free draining sands is likely to be no different to "short term" for poorly draining clays.

As the reviewer, I would like to exercise some licence, and resurrect a couple of specific topics for contemplation.

The first is on how to deal with determining the passive support from a sloping toe, to a pole retaining wall. Table 5.1 of the notes presents a range of Kp values for a limited selection of influencing parameters, but which appear to heavily penalise the support capacity of the soils. To rationalise this in the past, this engineer has philosophised that the designer could approximate the load case by defining a passive wedge above an envelope extending from the point of inflexion on the pole's embedded length, out to daylight, and then conservatively ignore that portion of the wedge above the level of that daylight point. Unfortunately, past calculations using this approach haven't provided a closed solution. But it was at this point that Colin Ashby (without tears in his eyes) introduced his "onion peel" theory, which, without the benefit of detailed analysis, appears to continue with the principles of failure condition that Mick had earlier introduced, and thence, provide a closed solution.

For the second, this reviewer has often been accosted by structural engineers, wanting to combine both shaft resistance and end bearing in the design of piles founding in cohesive soils, and who have responded in despair when prevented.

The Parton and Olsen paper from the Piling Symposium at Hamilton, 1986, discussed this, and with regard to cast insitu bored piles, said,

"The frictional resistance on the shaft develops rapidly and almost linearly with settlement. It may be fully mobilised when settlement is about 0.5 % of shaft diameter." (say 2.25mm for a 450mm dia pile) *"Thereafter it remains constant or slightly decreases with increasing settlement. On the other hand, the base resistance is seldom fully mobilised until the pile settlement reaches 10% to 20% of the base diameter."* (ie 45 to 90mm for a 450 dia pile)

"Results of pile tests show that if piles are designed to carry a working load equal to 1/2 or 1/3 of the total failure load then it is likely that the shaft resistance will be fully mobilised by the working load (Simons and Menzies, 1977)."

So what it seems to come down to is, that the design should be controlled by the serviceability requirements of the structure. If you can tolerate settlements of up to 20% of the pile diameter, then you can design for both end bearing and skin friction.

It also seems that this sort of concept really only applies to piles in cohesive residual soils and that it is a different scenario for piles socketing into sandstone/soft rock. At that same conference, a paper by Peter Millar reported on a selection of load tests, including some on a series of four heavily instrumented, 400mm diameter socketed piles carried out by Bill Gray in a tunnel at Mohaka. Two of the piles were supported by side friction only, while the other piles included side friction and end bearing resistance. Inspection of figure 3 of that paper clearly indicates that at an applied load of say 500 kN for a side resistance only pile, there was a pile head settlement of just 0.5mm (only 0.125% of the pile diameter). For the same load on an end bearing pile, the displacement was 4.0mm. Although this latter figure is still only 1% of the pile diameter, it is 8 times the displacement for friction only, for the selected load. So for piles socketed into sandstone, there may still be a disparity of settlements between the pile shaft and its base, but the magnitude of either is likely to be so low that the structure on top can tolerate it.

As a result, it seems appropriate to use both shaft resistance AND end bearing for piles socketed into sandstone, but not for piles in residual cohesive soils, unless one is really desperate to make the calculations work, AND can be confident that there are likely to be other built-in contributions to the support of the structure, or that the structure is settlement tolerant.

But for me, Mick's coup de grace was the appendix to chapter 7, "Broms and beyond", as featured in Geomechanics News no 59, June 2000. For the unequipped, the conclusion is that Broms' assumption of an unsupported length of 1.5 pile diameters is too conservative for pole wall design, and that a length of 250mm is appropriate for both the short and long pile cases.

In closing, I would like to repeat the attendees' thanks to Mick for his sterling effort. Having recently returned from a short course on Earth Retaining Structures at Imperial College, London, I am happy to report that we don't lack the quality here, just the frequency (and thankfully, the cost).

International Young Geotechnical Engineers Conference, Southampton, UK

Reporter: Paul Horrey
Golder Associates (NZ) Ltd

The First International Young Geotechnical Engineers Conference was held in Southampton during early September 2000. Over 100 delegates aged 35 or under from 55 countries attended the five-day event, which was hosted by the University of Southampton, in association with the International Society of Soil Mechanics and Geotechnical Engineering and the British Geotechnical Society. Tony Fairclough of Auckland and Paul Horrey of Christchurch braved the long flights and frightening price of UK beer to attend as New Zealand representatives. Following the established format of regional YGE events the conference was based on live-in accommodation at the University. Short technical presentations were given by each delegate on a huge variety of topics ranging from numerical modelling and laboratory projects to natural hazard management, mining, and geotechnical construction problems. Tony and Paul's presentations were printed in the last edition of "Geomechanics News"

Keynote addresses by Dr Suzanne Lacasse, Dr Kerry Rowe, Professor Robert Mair, and Dr Kenji Ishihara gave a perceptive insight into the diverse application of modern geotechnical engineering to real life problems. Other senior member of the UK geotechnical community also gave up their weekend to participate and pass on some of their own experience.

Full day field trips to the Isle of Wight and Dorset were favoured with unseasonably good weather, fantastic geology, and an abundance of public houses. Social events (official and otherwise) were organised each evening. The success of these could be gauged by the physically jaded appearance of many delegates by day 5 of the conference. The conference provided a unique opportunity to meet and get to know others in the same field from very different backgrounds. The challenges of catering for so many people from all over the world were capably met by the hard working organising committee, making for a first class event.

Australasian Young Geotechnical Professionals conferences are held regularly in this part of the world, and the next is scheduled early in 2002, probably in Auckland. They are a great chance for younger members to develop presentation skills and to meet and socialise with other geo-professionals, to the benefit of both delegates and their employers. Watch "Geomechanics News" for details.

ENGINEERING & DEVELOPMENT IN HAZARDOUS TERRAIN

NEW ZEALAND GEOTECHNICAL SOCIETY 2001 SYMPOSIUM

24 – 25 August 2001

University of Canterbury

Christchurch New Zealand

The Management Committee of the New Zealand Geotechnical Society takes pleasure in inviting you to attend our 2001 Symposium entitled "Engineering and Development in Hazardous Terrain". The symposium will feature invited presentations on recent interesting projects encompassing the investigation, design, risk management, and mitigation for developments subject to:

- river and coastal erosion, deposition, scouring.
- slope failures, debris flows, avalanches, rock falls.
- active faulting, earthquake shaking, soil liquefaction
- volcanic activity
- subdivisions, contaminated sites, subsidence, and landfills

Theme sessions will be held over the Friday and Saturday and will be followed by an optional one day field trip on the Sunday. Ample time will be available for social interaction and both formal and informal discussions with colleagues. A trade display featuring latest geotechnical innovations will run concurrently.

Please mark your diaries for this important event and bookmark our website address: www.conference.canterbury.ac.nz/geotechnical/ for up to date information on the symposium.

A call for papers will be mailed to members in late November

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GeoEnvironment 2001

2nd Australia-New Zealand Conference on Environmental Geotechnics
28-30 November, 2001

Newcastle, Australia

FIRST ANNOUNCEMENT & CALL FOR PAPERS

This is the second in a series of conferences on geoenvironmental engineering organised by the Australian and New Zealand Geomechanics Societies. The principal aims of the conference are to foster the development of integrated solutions to geoenvironmental problems by the application of interdisciplinary knowledge, and to promote further education of professionals involved in the delivery of environmental services. These aims will be supported by state-of-the-art workshops run by national and international experts.

The organising committee would like to take this opportunity to invite you to take part in GeoEnvironment 2001. The conference is to be held in Newcastle City Hall and promises to be a stimulating and enjoyable conference offering a range of social and technical functions.

Themes

The conference will run over three days and have the following three broad themes:

- Site investigation and contaminant transport modelling
- Engineering solutions
- Implementation, economic planning and regulatory frameworks

Key Dates

Submission of Abstracts	29 Dec 2000
Notification of acceptance of Abstracts	14 Feb 2001
Submission of Draft Papers	30 Apr 2001
Notification of acceptance of Papers	29 Jun 2001
Submission of Final Papers	31 Aug 2001

Other Activities

The conference will attract some of the most eminent environmental engineers and scientists from Australia, New Zealand and the world. It is hoped that their expertise can be fully utilised by staging a number of pre-conference workshops. Our website outlines a number of proposals for workshops on topical environmental issues. Please visit our website and register your interest in any of these workshops so that arrangements can be made.

Enquires can be addressed to the conference secretariat: ICMS Pty Ltd, 3rd Floor, 379 Kent Street, Sydney, NSW 2000, Australia. Telephone: +61 2 9290 3366 , Facsimile: +61 2 9290 2444, Email: geoenv@icms.com.au

Registration of Interest

Please register your interest in this event by visiting our website at www.icms.com.au/geoenvironment and submit your contact details, abstract and comments.

Standards, Law & Industry News

Standards

BACKGROUND

"....and on the seventh day, God rested. Man decided to get Gods' work peer reviewed by an independent engineer, who wrote a long and verbose specification that could not possibly be achieved on-site."

Out of this grew "The Standard", an even longer and more wordy document with which *"all creatures contained in said dominion shalt comply"*. The Standard set out to cover quality procedures for absolutely every facet of creation, in the event that it should be found some aspect of a particular creation was not quite up to scratch.

AS/NZS 1547:2000 ON-SITE DOMESTIC WASTEWATER MANAGEMENT

General

Although not primarily a geotechnical standard, the new standard AS/NZS 1547:2000 contains a number of "geotechnical" definitions and clauses with relevance to the civil community. Some of these aspects are raised in this article (which should not be regarded as a critique), the purpose of which is to stimulate any NZGN readers who may be affected by them to contact Standards New Zealand. A draft standard was issued as DR 96034 in Australia and New Zealand.

Among the referenced documents listed in the standard are *AS 1289-Methods for testing soils for engineering purposes*, *AS 2758-Aggregates and rock for engineering purposes*, *NZS 3121-Specification for water and aggregate for concrete* and *NZS 4402-Methods of testing soils for civil engineering purposes.*

Definitions

Among the definitions that may be of interest are:

1.6.5 Aggregate
Piles of rock meeting the requirements of AS2758.1 or NZS 3121.

1.6.9 Boulder
A natural, geological entity with middle dimension greater than 600mm.

1.6.16 Dispersive Soil
A soil that has the ability to pass rapidly into suspension in water.

1.6.29 Filter Cloth
Any durable, permeable textile material suitable for use with soil, rock or earth.

1.6.3 Impermeable layer
Soil layer with permeability less than 10% of that of the overlaying soil layer. The term "impermeable" is not to be taken in a literal sense.

1.6.49 Rock
A natural entity of geological origin with middle dimension between 200 mm and 600mm.

1.6.58 Slickensides
The skin or coating formed on (usually) large units of soil, which will show striations or grooves resulting from the periodic rubbing together of the soil units due to shrinkage and swelling in response to moisture change.

Of the above definitions, the contradiction that immediately stands out is the incompatibility (due to size classification) of the terms "Boulder" and "Rock". Later on in the standard, reference is made to "crushed rock". Further problems arise when considering that the term "Aggregate" includes the term for "Rock", presumably as described in 1.6.49. Referenced document NZS 4402 defines boulders as "Particles larger than 200mm...". The broadness of some of the other definitions is worrying, as these terms have rather more specific meanings in civil engineering practice.

Clauses

Among the clauses that may be of interest are the following:-

4.1.3.7.4 Seasons
Within this clause, the following comment is made "*Comment. Many site and soil assessment procedures are not relevant or valid when soils are saturated*". The term "saturated" is not included in the definitions, and so may be taken to include soil beneath the groundwater table in addition to soil subject to capillary action. As many/most New Zealand soils are typically saturated through half the year, the question may then be asked – what site/soil assessment procedures are valid for saturated conditions?

4.1A2.2 Groundwater
This clause states among other things that "...the nature and quality of aquifers (confined or unconfined), water-table heights (seasonal and perched)...shall be assessed". This appears rather rigorous if taken literally, as proper assessment of the aforementioned items may be prohibitively expensive for domestic wastewater systems. From 4.1A3.1 it is assumed

that anecdotal evidence (vegetation, existing bores, soil oxidation) is an acceptable means of defining some of the groundwater conditions previously described.

4.1D3 Size
A further definition is provided in this section describing the size range of "rock" fragments. Material between 200mm and 600mm (previously defined as "Rock") is now defined as "Stones".

Conclusion

There are a number of issues within this standard that the practicing engineer may find cause for concern. Many vague and generalised definitions and statements are made, in addition to several highly specific and constraining clauses. It feels rather like reading an undergraduate textbook, than a document setting out the standards to be achieved by on-site domestic-wastewater management. It is the author's opinion that Standards should endeavour to be consistent with externally referenced Standards, and internally consistent with respect to definitions and clauses.

Jon Sickling
Sinclair Knight Merz

Industry News - ASFE/NZ?

"Why are we being sued so much? We're good at what we do."

That was the quandary faced by US geo-professionals in the last quarter of the 1960s, when their claims-against record had risen so high that no insurer was willing to issue professional liability coverage to them. In response, a group of ten firms established Associated Soil and Foundation Engineers; a not-for-profit trade association whose mission was to identify the causes of the claims and then develop programs, services and materials to help Member Firms reduce their liability losses.

The association's efforts were startlingly effective. By 1985, just 15 years after the group's creation, an independent survey showed that ASFE members not only could obtain professional liability insurance from several sources, they paid less for it than any other US design professionals. And they were more profitable then any others, too! Had ASFE stumbled onto something remarkable; something that permitted its Member Firms to lower their liability risks on the one hand, and increase their profitability on the other? The answer, in a word, is "Yes."

What did ASFE discover? According to its President, Kevin B. Hoppe, P.E., CFO of NTH Consultants, Ltd. *(Farmington Hills, MI)*, "The key realization was that we were *not* particularly good at what we did. We made the mistake of assuming that technical proficiency equated to loss prevention, and that's just not how things work. In fact, the key issue was management proficiency, and that's something the original ASFE members sorely lacked."

The key to recovery, Mr. Hoppe said, was ASFE's creation of programs, services, and materials to help members enhance their management skills sufficiently to greatly lower their liability exposure. "Most of us spend most of our time doing things we were not educated for. Which we were not well trained for and which frankly we might not like to do, such as writing, editing, proofreading, supervision, team leadership, preparing proposals and contracts, implementing client relations programs, and so on."

ASFE reasoned that, if it could help its members manage more effectively, it would help them become better at what they really did. It believed that those skills would help members create the type of positive relationships that could reduce claims frequency (friends usually prefer to work things out instead of sue), expand business opportunities (at lower cost), and enhance profitability. And that, of course, is exactly what happened.

ASFE is still going strong, evolving to reflect the evolution of its Member Firms' practices. ASFE no longer stands for Associated Soil and Foundation Engineers. In fact, the letters now stand for nothing at all; the group's formal name is ASFE, Inc. It adopted its former acronym so as to not put off those within Member Firms who were not geotechnical engineers, and to help maintain the allegiance of those still involved exclusively in the arena of soil and foundation engineering. Today, ASFE's membership comprises firms that provide geoprofessional, environmental, and civil engineering services, and its programs, services, and materials are continually refined and expanded.

ASFE is the only trade association of its type in the U.S. that concentrates exclusively on management issues for technical professionals. As long-time staff director John Bachner points out, "Our members seem better aware than most that what their firms do is fundamentally no different from what other service professionals do, be they doctors, lawyers, auto mechanics, or barbers. They must keep their clients happy, which they do by providing what clients believe is technical competence within a customer-focused context where deliverables are provided on schedule, budgets are met, clients are shown they are appreciated and respected, and so on. In

short, while our members may be engaged in a technical profession, they are in the people business. Those that grasp that fundamental can do exceedingly well."

So what does any of this have to do with what's going on in New Zealand? First, in New Zealand, as elsewhere worldwide, claims and lawsuits comprise an increasing risk, if only because the new global economy's "Better. Faster. Cheaper." mantra places more liability burden on consultants.

Second, New Zealand practitioners, just as their US counterparts, strive to earn a reasonable profit. ASFE has been extremely effective over the years in directing its management guidance in areas where the techniques one uses to lower risk can at the same time enhance profitability.

Third, the extraordinary risk-management and other materials developed by ASFE are almost all applicable to practices in virtually any English-speaking nation. Much the same can be said of the group's programs and services. Educational programs that range from daylong seminars to a new 90-hour, accredited project manager training program, still in development. Services include advertising to attract potential employees to Member Firms, as well as advertising designed to attract clients.

Fourth, ASFE is seeking to expand its international presence encouraged by reports from members in the UK, Canada, Australia and elsewhere that in fact almost everything ASFE has developed is also applicable to them. Principally because "people are people" and because of a common legal heritage.

Fifth, in order to extract more from its available time and funds, ASFE has made a strong commitment to electronic communication and deliverables. It is now possible for members anywhere to quickly access a wide array of materials, just by connecting to the Internet.

Sixth, ASFE has announced a new level of membership, titled eASFE International, or eASFE/I. Firms located outside of North America can join for US$525 per year, and receive free of charge everything ASFE offers electronically. This already amounts to a good bit, and is expanding steadily more, as existing materials are scanned into the system for download in Adobe Acrobat (pdf) format. Included in the mix are its much-heralded newsletter *NewsLog*, its series of *Practice Alert* monographs and continually more publications that can be offered so much more economically in electronic versions only. All ASFE materials are available free of charge, in virtually unlimited quantity, with "regular" Full Membership. As such, project managers can *each* order literally thousands of dollars worth of materials, at no charge. "We want to encourage the formation of local groups that are at least loosely affiliated with us," Mr. Hoppe said, "so we have the ability to communicate more effectively in an era of rapid globalization."

Which gets us to the principal point.

ASFE has adopted a program to enhance relationships with similar organizations in other nations. The current concept is to offer eASFE/I membership for US$250/year to members of similar groups with similar missions. In part to encourage membership in the in-country organizations, and, in part, to create more awareness of ASFE and what it has to offer. It also would make available its "hard copy" publications, audiotapes, CD ROMs and the like at an attractive price for example US$75 instead of the customary non-member rate of US$375.

According to Bachner, "At [US] $250 per year, ASFE would have to save a firm less than two hours' of an attorney's time each year in order to be worthwhile. If we cannot do that, we do not deserve support."

Loss prevention and risk management are not ASFE's only foci. As Mr. Hoppe pointed out, "ASFE's advertising and other outreach efforts can bring both new personnel and new clients to our Member Firms. And the meetings are extraordinarily useful for networking with peers. My firm is not at all alone in being able to say that we've probably picked up well over $1million in business as a consequence of relationships established through ASFE. The prospect of being able to do that on a global basis is exciting."

For more information about ASFE, refer to its website (www.asfe.org), or contact its headquarters via e-mail (info@asfe.org), telephone (301/565-2733), fax (301/589-2017), or post (8811 Colesville Road, Suite G106, Silver Spring, MD 20910).

For information about efforts in New Zealand to establish an ASFE counterpart, contact Grant Murray as the Editor of Geomechanics News. Any support and interest will be passed on to the Management Committee, IPENZ and ACENZ (the most likely existing organisation to encourage closer links to ASFE).

Industry News - The Dirt on the Net

NZGN invite you on an electronic journey. The eager volunteer staff have spent their evenings beavering away on the net, searching for wonderful snippets of e-knowledge to satisfy you, the reader. After wading through search responses laden with family sensitive subject lines, we have filtered the mud from the trash. Geomechanics News should not be held responsible if the links given turn out to be fronting dubious websites for would-be dictators, merchants of low morality, or structural engineers.

(Note that most of the following links contain freely downloadable pdf format documents.)

First stop is the wonderful world of the United States Navy. Connect with your ISP (which stands for Internet Service Provider for those that need the complete non-nerd guide) and type *http://www.efdlant.navfac.navy.mil/criteria/publications_15.htm* and you should find yourself at the Naval electronic publications list. Here you can find technical manuals on just about everything, including "Evaluation and Selection Analysis of Security Glazing for Protection Against Ballistic, Bomb, and Forced Entry Tactics" and the ever popular "Basic Guidelines for Chemical Warfare Hardening of New Military Facilities". Of course you may just settle for downloading in Adobe Acrobat (pdf) format the NAVFAC Soil Mechanics and Foundation Design Manuals. If nothing else, this link should do wonders for alleviating the collective guilty consciences of the nations' Geotech Engineers (integrity plus) who may have resorted to illegal photocopying.

If running through the undergrowth, wearing camouflage and grunting is your thing, then swiftly type *http://www.usace.army.mil/inet/usace-docs/eng-manuals/cecw.htm*. This gets you to a similar webpage for the US Army Corps of Engineers, a list of civil engineering manuals (pdf format) which include a number of very useful Geo friendly titles. The following are a selection you may find useful:

- Seepage analysis and control for dams (1970)
- Construction control for earth and rock-fill dams (1995)
- Retaining and flood walls (1989)
- Design of sheet pile walls (1994)
- Tunnels and shafts in rock (1997)
- Design of pile foundations (1991)

There are many other titles available on bearing capacity, settlement, site investigations, and other aspects of related civil engineering.

For the rock-doctors out there, we recommend you download yourself a copy of Dr. Evert Hoeks' Rock Engineering Notes, from *http://www.rocscience.com/roc/Hoek/Hoek.htm*. This is a good up to the minute rock mechanics text from the undisputed heavyweight of all things rocky. It comes out pretty thick and fast from the printer, and you may need a ream or two of paper on standby.

If driving nails into soil and making banks stand up at obscene angles is your thing, then you may find this link to Federal Highways Administration e-library page makes your heart beat a little quicker. Try *http://www.fhwa.dot.gov/bridge/elibrary.htm* and you will find some reasonable material on soil nailing and bridge foundations. Due credit should go to the editor for finding this one, after surfing the web unfruitfully for hours looking for recent Scottish Sporting Success Stories. Just leave him, he's happy. *(Editors footnote: Scotland are currently the holders of the Calcutta Cup – the oldest prize played for in international sport.)*

If you haven't by now recognised that the pdf format is going to make your life as an engineer a lot more efficient, you should probably be on the Equatorial Guinea swim team. Admittedly you don't get that certain thrill of the unknown you do with a temperamental fax machine. Nevertheless, checking out Adobe's e-paper library can provide all sorts of very useful information. It searches the entire web for freely downloadable pdf's, so if you're really stuck for information on the cyclic triaxial characteristics of potting mix then check this out. The link is *http://searchpdf.adobe.com/*. The search engine is a little frustrating, as it appears that it cannot search for keyword combinations.

Perusing the Electronic Journal of Geotechnical Engineering at *http://www.ejge.com/* is really a window into the underground life of geogeeks. It's a bit like a Spice Girls fan club page for Geotech, except the hall of fame has wise looking portraits of venerable men of soil mechanics. There are a few interesting papers and articles easily accessed through the home page. It is worth keeping an eye on this site, as it is quite possibly the future of Geotech on the net. Very user friendly and fast but it will benefit from more heavyweight technical information.

The last offering from the little helpers at NZGN is the Geotechnical and Geoenvironmental Software Directory at *http://www.ggsd.com/*. It is possible to access all sorts of shareware, freeware, and commerical software. Many of the freebies appear to be proprietary software from european geotextile contractors and suppliers, but it is really worth having a look at this.

We hope that some of these links are useful to you, and that you don't get any discrete calls from the Secret Intelligence Service suggesting that you stop acessing *these kind of sites*. Should you have any trouble with these links, or suggestions for the next NZGN edition, please email J Sickling.

Book Reviews

Proceedings of the International Symposium on Slope Stability Engineering

These two volumes should be on the shelf of every geotechnical consultancy. They are the collection of advanced research that is readily applicable to the increasing demand for solutions in soil rock slope engineering. Each University Engineering Library requires a permanent shelf reference and a separate floating copy for students presenting case study projects.

Headings are:

1. Geological and Geotechincal site investigations
2. Soil slope stability analyses
3. Rock Slope Stability analyses
4. Effects Of Rainfall and Groundwater
5. Effects of Seismicity
6. Design Strength Parameters
7. Slope Stability of Landfills and Waste Materials
8. Stabilization and Remedial Works
9. Stability of Reinforced Slopes
10. Probabilistic Slope Stability
11. Landslide Investigations
12. Landslide Inventory, Hazard Zonation and Rockfall
13. Simulation and analysis of debris flow

In all, there are 221 technical papers from 40 countries.

Keynote Lectures

Prof. K. Ishihara et al. of Science University of Tokyo..
Flow type failures of slopes based on behaviour of anisotropically consolidated sand

Dr. Zuyu Chen et al of China Institute of Water resources and Hydropower Research. Beijing.
Limit Analysis for Slopes : Theory, Method and Application

Prof. D.G. Fredlund et al of University of Saskatchewan, Saskatoon. Canada
Using limit equilibrium concepts in Finite Element Slope Stability Analysis

Prof. D. Leshchinsky of University of Delaware, Newark. U.S.A.
Stability of Geosynthetic Reinforced Steep Slopes

Prof. Mihail Posescu of University of Civil Engineering, Bucharest, Romania.
The Mechanisms, causes and remediation of cliff instability on the western coast of the Black Sea

Prof. H.G. Poulos of UNSW, Australia
Design of Slope Stabilising Piles

The 221 Technical Papers give practicing Geotechnical Engineers up to date practical and challenging methods for Slope Stability Engineering. These include papers on a novel method for monitoring, *Application of Acoustic Methods to Slope Monitoring* by T. Fujiwara – Nippon Koei Co.

Mineralogical, weathering mechanisms, automated measurement of pore water pressure, back analyses based on limit equilibrium, slope failures and earthquakes, the effects of drainage pipes, are all covered.

Case studies are numerous:-

1. Railway embankment failure due to E/Q in Hacinohe N.E. Japan (Distance = 200km from Epicentre).
2. Monsoon induced, cut and fill slope failures along the East West Highway Malaysia (annual rainfall = 3600mm).
3. Juluis Caesar's preliminary Geotechnical Investigations into draining Lake Fucino (Area = 170km^2) and followed up by Emperor Claudius using 30,000 slaves to construct a 10m^2 x 5653m long tunnel to effect the report (page 965).
4. Amazon riverbank failures with flood draw down of 12mH, Peru.
5. Hydrothermal activity on the weathering mechanism of slope failures in Hiroshima and Shimane Prefectures.
6. Pyroclastic flows in Kagoshima Prefecture.
7. Slope failures after windthrows in Kyushu.

These last three have relevance for NZ Geotechnical Engineers practicing in Rotorua (Hydrothermal), Auckland, Rotorua, Nelson and Dunedin (Pyroclastic), East and Central North Island and Canterbury (windthrows).

I make special mention of four papers relevant to the hosting city, Matsuyama, on Shikoku Island.

Matsuyama is near the Median Tectonic Line, a major structural fault. As a city it has landslide problems, compounded by intense living pressure on hillside slopes, for the island is some 80% mountainous. Indeed throughout Japan rock bolting of highway cuttings, rock and snow protection tunnels are a frequent necessity.

F. Nishimura et al, *Groundwater quality $CaCO_3$ content) in Fracture Zone Landslides.* Where Clay minerals from chlorites to smectite are considered.

R. Yatabe et al, *Landslide clay behaviour and countermeasures (sic) works at the fractured zone of Median Tectonic Line.* The Shikoku – Jukando expressway follows

the major fault line. For the clay in the landslide zone the clay was found to be $\varnothing_p = 20° - 35°$. However residual strength was only $\varnothing_r = 8° - 18°$. Drainage wells with spider drains and deep piling are unable to guarantee the FoS for E/Q (and any tectonic creep movement) of the Class 1 Active Fault.

H.Kono et al, *Geological & Soil mechanical study of Sawatari landslide in Ehime.* VLF survey methods were successfully used to locate the existence of a 5m wide major fault in a landslide of 1000m in length and width. This was backed up with Resistivity Profiling.

Y. Momiyama et al, *The general characteristics of landslide along the Median Tectonic Line due to road construction.* Landslides occur on very gentle ~ 10° slopes with $\varnothing = 30°$.

Mining Engineering has notable papers by W. Wehr et al of Germany a *Case study on liquefiable mine tailing sand deposit* which uses the physical characteristics of the excess rate of kinetic energy and velocity field is then plotted.

Qing Yang et al, *Reliability analysis and risk evaluation of the slopes of open pit mine* evaluate the probability of a progressive failure using the Monte Carlo Method.

Indeed probabilistic methods are important for cost benefit versus the life of the engineering structure be it a road, dam or building. There are 8 papers on this topic and include S. Pumjan et al, *Localised probabilistic site characterisation in geotechnical engineering*, dealing with probabilistic modelling of in-situ parameters. D.S. Young a partner of S. Pumjan in Mining Engineering at Michigan Technological University applies the variance in c' and $\varnothing$ to a 30 mH slope @ 30°.

L. Belabed of the University of Guelma, Algeria evaluates the *Overall stability of anchored retaining walls with the probabilistic method.* The appropriate mechanical model to evaluate the overall stability and the required anchor lengths is controversial. Belabed proposes a mechanical failure model on the basis of kinetic theory.

Where slope reinforcing can be done, strain based FEM (Finite Element Method) is used by K. Okabayashi et al in *Stability analysis of reinforced slopes using a strain-based FEM.*

Centrifuge modelling and full-scale tests, 3-D stability analyses and numerical analysis are covered. Materials such as composite fabrics, steel grid and geosynthetics are given due emphasis.

Rock failure types are generally categorised as log spiral (circular), planar, toppling or wedge. Tension cracks and weak clay zones in the rock have a major effect on failure.

Z. Chen in *An upper bound wedge failure analysis method* takes Hoek's force equilibrium method further. Hoek's method requires some assumptions, as the problem is statically indeterminate. Dr. Chen uses a new method based on plasticity originally proposed by Pan in 1980. This method has been applied to the Three Gorges Hydroelectric Power Project in China.

Prof. M. Pender in *Earthquake and Seepage effects on the mobilised shear strength of closely jointed rock,* finds that earthquake effects are more severe than seepage.

This is an essential publication for any geotechnical consultancy and Engineering Library. It will not gather dust.

REVIEW BY:- PAUL FINLAY, WAITAKERE CONSULTING ENGINEERS

Proceedings of the International Symposium on Slope Stability Engineering
Matsuyama On Shikoku Island, Japan, 8 ~ 11 Nov 1999.

And can be ordered from:-

A.A. Balkema
ISBN 90 5809 079 5 (two volumes)

Email: balkema@balkema.nl;
P.O. Box 1675
3000 BR Rotterdam
Netherlands

Advances in Aggregate and Armourstone Evaluation

In the United Kingdom, and elsewhere about the world, the demand for aggregates and armourstone for both construction and coastal/river defences is growing while available resources become more difficult to recover. This book aims to highlight a range of issues for construction materials that are before the aggregate industry, facing growing pressures for environmental sustainability and standardisation of products. The papers contained in the book come from selected presentations to the Extractive Industries Geology Conference at Warwick UK in 1996.

The book is subdivided into three sections:

- Marine sand and gravel geology and resources
- Armourstone evaluation and shingle performance assessment
- Aggregate testing and use of alternative aggregates

Section 1 contains four papers that focus on marine and gravel resources about the United Kingdom and Europe, the use of geophysical exploration methods to identify sand resources in Hong Kong, and the need to understand Quaternary geology as part of the management and licensing of off shore aggregate operations.

The papers on off shore deposits in the UK and Europe are of limited value to the New Zealand situation but the relevant sections on investigation and interpretation provides useful background reading for anybody considering an off shore dredging operation in New Zealand. Off shore aggregate production in New Zealand is limited. Examples include dredging sand in the Kaipara Harbour and project specific dredging for land reclamation such as for the new Marsden Point Wharf. However, for particular areas about New Zealand, such as Auckland, attention is likely to become more focused on off shore sources as land based aggregates become more difficult to source and costly to transport to markets.

Section 2 contains a set of six papers on armourstone for protection works, mainly for coastal defences. Approximately 54 % of the English coastline is protected by some form of coastal defence and many of these areas are over 100 years old and in need of replacement/maintenance. The publication of CIRIA/CUR (1991) *Manual on the use of rock in coastal and shoreline engineering* has had a major impact in the UK on the use and design for armourstone. Yet many problems exist with the specification and testing of armourstone rock. The need for high quality armourstone rock and the many difficult issues surrounding the specification and testing of these products are addressed in several of these papers. A case study from Malaysia is presented on how a specific armourstone quarry was set up to supply rock for a breakwater and groynes for a major coastal development.

Section 3 presents six papers on aggregate testing including two papers on the miro- Deval test for aggregates as an alternative to Los Angeles Abrasion testing. A most useful paper on a statistical study of aggregate testing data is presented along with some discussion on the use of alternative materials for aggregate use (i.e. concrete made with pulverised fly ash and china clay waste).

In conclusion, this book meets its objectives of setting out the current situation for aggregate industry in the UK and Europe with particular regard to off shore production and armourstone for coastal defences. Several issues are raised that need further research and investment. The book has limited application to the New Zealand physical and commercial situation but it does set out useful background and guidance for the assessment of potential off shore aggregate sources and in the specification and assessment of armourstone rock.

This is not a book that is likely to have wide appeal to the New Zealand geotechnical community. However, it is worth knowing it is about and should be read by anyone working to develop offshore aggregate resources or involved in the use and specification of armourstone rock for erosion control.

REVIEW BY:- DOUG JOHNSON, SENIOR ENGINEERING GEOLOGIST, TONKIN & TAYLOR LIMITED

Advances in Aggregates and Armourstone Evaluation
Geological Society Engineering Geology Special Publication No.13

Edited by: - J P Latham
Published by: - The Geological Society London
Date: - 1998
Web shopping on: - http"//bookshop.geolsoc.org.uk
Price £59/US$99

Book Reviewers - Wanted

The NZ Geotechnical Society has a number of recently published books available for review. The publishers supply these books free for the Society to review. We are looking for eager volunteers to review the following books:-

Geotechnical aspects of Underground Construction in soft ground. (O Kusakabe, K Fujita, Y Miyazaki (eds)) 1999. Proceedings of the international symposium, IS - Tokoyo '99.

Influence of Gravity on granular Soil Mechanics. (R Katti, A Katti, D Katti.) 2000. AA Balkema Publishers.

Tunnelling, Management by Design (Alan Muir Wood) 2000

Geological Hazards- their assessment, avoidance and mitigation (Fred Bell) 1999

The reviews are to be succinct and critical appraisals of the books in the order of 1 or 2 A4 pages in length. Reviews will be forwarded to the publishers. Upon completion of the review the book reviewers can keep the book. Now there is a good incentive for you!

If you are interested please contact:

Debbie Fellows, Management Secretary,
Tel 09 817 7759
Email dfellows@xtra.co.nz

Special Interests

Numerical Analysis in Soil Mechanics, Part 2

Sergei Terzaghi, Sinclair Knight Merz

Last article I finished with a comparison of three different models (linear elastic, undrained effective stress, and a total stress) looking at the same problem and a comment that in this follow-up article I would look at some of the reasons for the differences. Two key issues need to be addressed; the question of pore pressures generated as a result of the embankment loading and the impact of incorporating plasticity in to the calculations. In this article I will focus on the issue of pore pressure generation, as it often appears to be poorly understood, particularly in the context of modelling.

I will begin by consideration of the pore water generation under loading of a saturated isotropic linear elastic-perfectly plastic media. This qualification is very important to start with as it simplifies the issues. Such a material can be characterised as two phase, with the solid skeleton characterised by conventional parameters such as a Shear Modulus and Poisson's ratio, which is sufficient to derive any of the other elastic parameters (for example Young's Modulus and bulk modulus). The other phase (fluid) is characterised by a bulk modulus, and an inability to withstand shear. While the model description can be extended to multi-phase conditions, these introduce complexities beyond the scope of this series. It should be pointed out that most fluids under practical consideration have a bulk modulus that is much greater than most soils (at least an order of magnitude and often two or more orders of magnitude) implying a condition of virtual incompressibility.

Straight away with the simplifications made above we are introducing deviations from reality, since the soil skeleton will exhibit very different behaviour in straight shear compared to hydrostatic compression, which means that there is not a direct relationship between Shear Modulus, Young's Modulus, and bulk modulus. In fact, there is considerable doubt whether Poisson's ratio should even be used in soil mechanics. These issues will be explored in later articles. A key point to bear in find is that behaviour in shear is different to behaviour in compression, though, the model described above will not necessarily reflect the difference, except in very specific ways as discussed below.

Returning to considering the model, it is easy to see already that a key difference between an undrained loading situation and a drained loading situation is the consideration of the fluid phase. Under a drained loading situation (ie fully static or rate of loading is sufficiently slow that the soil skeleton carries all the load) the pore fluid only impacts the stress state to the extent of reducing the stresses on the soil skeleton to the so-called effective stress level (total stress less hydrostatic). In this situation it is important to note that the soil skeleton carries all the load. Under undrained loading (ie the load is applied faster than what the pore fluid can move through the soil skeleton, and hence a portion of the load is 'carried' by the pore fluid) the presence of the pore fluid enables re-distribution of stresses, as the fluid can only carry hydrostatic stresses whereas the soil skeleton must carry all the shear stresses.

This critical re-distribution is not modelled in any sort of total stress approach. (It should be noted that in terms of limit state type calculations, this is not a problem). If the problem is such that there is no drainage, then the mean change in stress is fully carried by the pore fluid, while the deviatoric (shear) stresses are carried by the soil skeleton.

However, as the soil approaches failure (or indeed once past threshold strain level), then the shear stresses may induce their own additional pore pressures as a result of any desired volumetric change. This will only be modelled in the elasto-plastic model as a result of changing into the plastic phase. This introduces some additional complexities, as the pore fluid is forcing a condition of zero volume change. Hence any movements in the soil mass is as a result of shear stresses and not mean stress/applied loads.

The condition of incompressibility introduces several other interesting complications. It implies a combined Poisson's ratio of 0.5, though in terms of the elasto-plastic soil, the skeleton will typically have a Poisson's ratio of around 0.25-0.3. The shear modulus will be the same for any conditions (drained or undrained and total stress). More interesting is that while the Young's Modulus for the soil will be on the order of 1.2 times greater for the undrained case compared to the drained case, the comparable confined modulus will be at least 20 times greater in undrained loading. This means that changes in stress that result in a mean increase in stress will result in little or no strain, whereas the shear induced strains will be the same for any case.

The change in stresses experienced by the soil skeleton described above, in combination with the different soil stiffness, will be sufficient by themselves to explain the different observed behaviours. For comparison, the same three models used previously have been modified and interrogated to provide the pore pressures generated for the same loading case. The results are informative.

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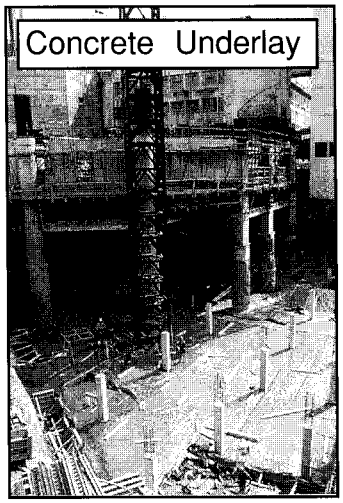

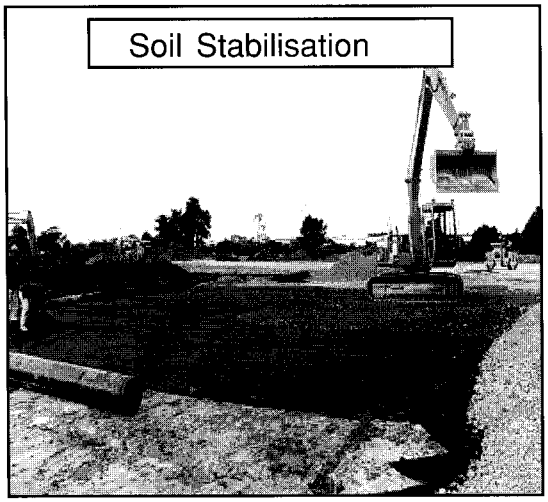

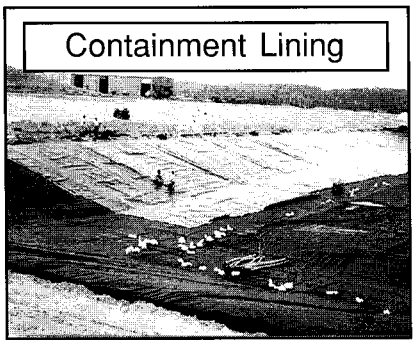

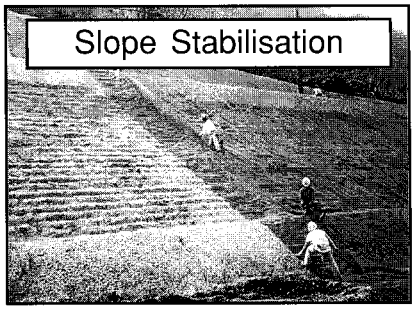

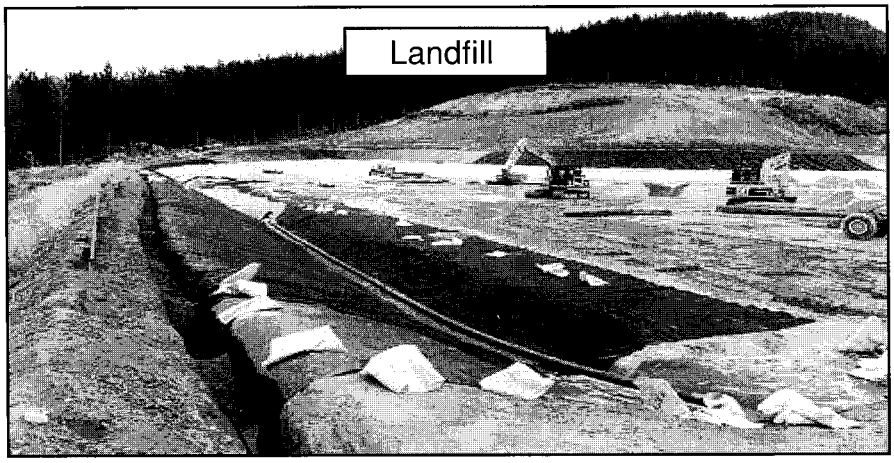

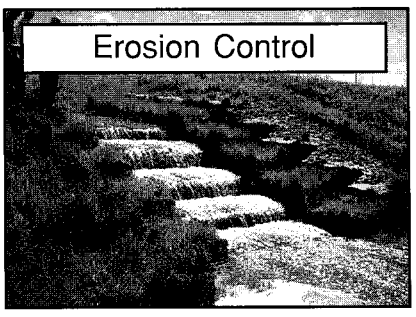

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IECA CORPORATE MEMBER

The linear elastic model (Figure 1) shows almost a classic stress bulb shape. The undrained Mohr-Coulomb (Figure 2) shows that the pore pressures are much more concentrated underneath the embankment, as one would expect in reality. Although the total stress model (Figure 3) is giving essentially nonsense answers it will not matter as the pore pressures are ignored.

Next article I will look at the influence of plasticity on the model, and then maybe combine the influence of the plasticity and pore pressures.

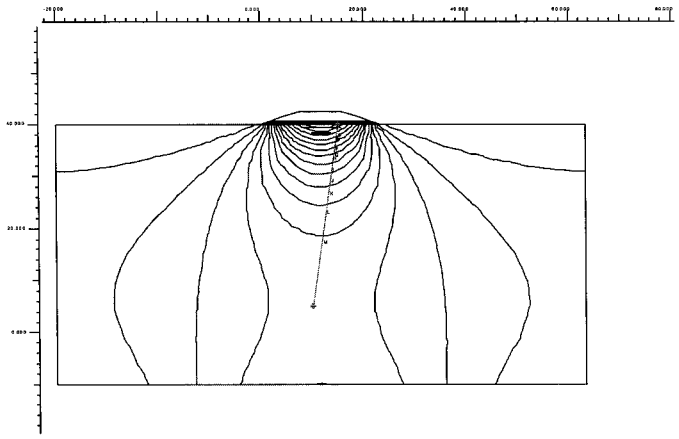

Figure 1. Linear Elastic Model

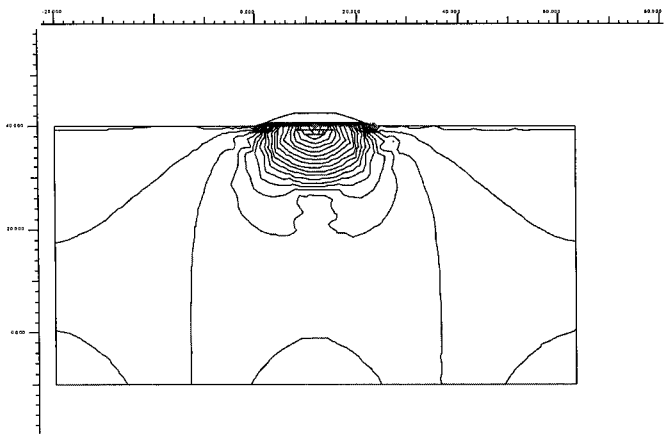

Figure 2. Undrained Mohr Coulomb Model

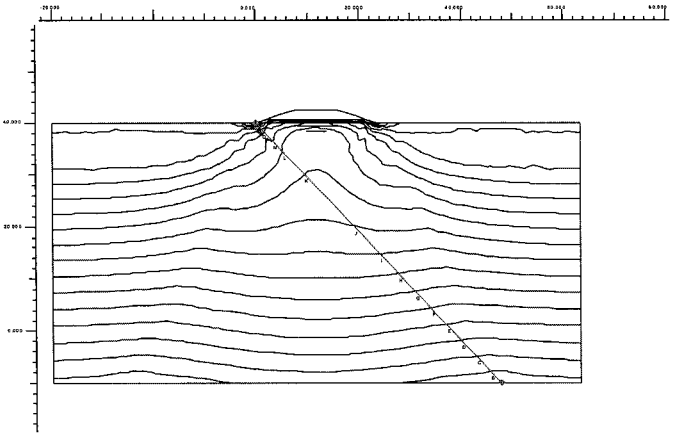

Figure 3. Total Stress Model

Thredbo Coroner's Report

Grant Murray, Sinclair Knight Merz

INTRODUCTION

It was with great intentions of learned endeavour, and the somewhat lofty, self-opinionated belief that there was a need for an astute summarisation, that I set out a few weeks ago to read the Report of the inquest into the deaths arising from the Thredbo landslide. I wanted to find out what caused the tragedy, what lessons could be learned and applied to our practice here in NZ and what messages should be preached to other professionals and those responsible for slope management.

After all, it was a tragic event and it is a horrifying thought that so many people lost their lives as a direct consequence of a geotechnical failure. How many of us have not thought, "There but for the grace of God"?

THE REPORT

The Coroner's Report is a hefty document. It can be downloaded off the internet and amounts to over 200 pages. With more than 100 witnesses and goodness knows how many historic factual documents, records, statements of evidence and cross-examinations for review and consideration you cannot help but be impressed with the way the report has been pulled together. This is not meant as a criticism of the report but it would be easier to understand if some of the important site plans were included and referenced. Having said that, if you are prepared to sift through the whole document, then the site history, circumstances leading up to the landslide and the consequences are graphically pieced together and described.

Any attempt I could make to summarise the report would be inadequate. For those that want to discuss and debate the findings then you really have to read the whole document. The report itself contains a very concise summary. However, if you read only this you miss out on the power of the underlying messages. If you are a Geotechnical Engineer involved in slope engineering and landslide works it will have a profound impact on your activities.

THE EVENT

For those that are not entirely familiar with the event – and I suspect that is precious few – on 30 July 1997, very late at night, a landslide occurred in Thredbo Village, NSW. The initiating landslide was quite small, involving little over 1300m3 of fill materials from a road embankment known as The Alpine Way. Eighteen of the nineteen people that were staying in the two properties impacted by the landslide died.

THE HISTORY

The Alpine Way had been constructed over 40 years previously. I interpreted the evidence presented to, and summarised by, the Coroner as suggesting that for almost all of that time the road embankments were known to be marginally stable and landslide prone, particularly when wet. Despite this fact, over a period of time, development below the road occurred on, or adjacent to, land that was identified in 1962 as an *"unbuildable slip area"*.

The Coroner very clearly sets out that it was not his duty or intent to decide on issues related to responsibility. However, the Coroner did have the power to make recommendations, based on the evidence presented, to those responsible for the Alpine Way and it's unstable nature in the interests of public health and safety. The evidence presented would imply that there was a catalogue of errors leading up to the event. The Coroner, metaphorically speaking, shakes his head in despair when he comments in the summary that the possibility of a landslide causing damage and serious injury was known by the relevant authorities for many years yet no specific recommendation was ever made for rectification. He concludes *"I have been unable to resolve satisfactorily in my mind how this occurred."*

It is quite clear that a road that was originally intended as short term construction access for the Snowy Mountains Hydro Scheme should never have been adopted for public use. The original design philosophy would simply have been to cut the easiest access possible, at the cheapest possible cost, across the steeply sloping hillside by benching and side casting. Do we know of any roads formed in a similar manner in New Zealand?

It is also quite clear that the responsibility for planning and building consent issues was confused and complicated by the fact that there appeared to be a local authority, national park, roading authority, commercial resort and private property owners involved in the decision making process. Based on the evidence presented it is staggering that an essentially urban development was permitted to occur in such a hazardous area. Again one must reflect on the vagaries of the RMA, the Building Act, their application and interpretation by different local authorities and the relative split of responsibilities for roads between Transit and our local councils.

THE MECHANISM

There were two conflicting arguments concerning the triggering mechanism (I think there were more but the Coroner essentially concentrated on the two more credible

postulations). The first, and the cause that the Coroner accepted, was that the fill slope had become saturated by a leaking water main. This was another dramatic instance where, in hindsight at least, it is hard to understand how an essential piece of infrastructure was installed in such a high risk area with little or no design effort. The pipe was asbestos cement with flexible, rubber ring joints and had a 90° bend in the immediate vicinity of the landslide area. Despite the fact that the contractor placed, at his own discretion, a large thrust block, there was compelling evidence presented of the on-going creep of the fill slope. It was contended that the slope moved upwards of 50mm since the pipe was installed in 1984 and that this resulted in joints opening and leaking water into the fill.

The opposing argument presented was that the construction of a retaining wall in early 1997 resulted in some modification to the existing drainage and concentrated a discharge or groundwater flow to the vulnerable fill. As an outsider, remote from the intimate detail of the event, one of the fascinating and compelling aspects of the report is the manner in which the Coroner summarises the opposing arguments as they were presented to him. And then, quite candidly dismisses the argument that did not convince him of its merit. In all honesty, it is hard to understand how anyone would be prepared to support a tenuous position in the face of such intense scrutiny.

Again as an outsider one is tempted to consider critically the arguments and merits of the various submitters. In almost all instances it was difficult to fault the logic of the Coroner but, unfortunately, the same could not be said for some of the expert witnesses. Reading between the lines (and I accept this maybe an unfair interpretation) I was left with the impression that the supporters of the Retaining Wall Theory went to extreme ends to advance their cause in order to deflect criticism and confuse responsibility.

One assumes that the Retaining Wall theorists would argue that the fact that the slope failed shortly after the construction of the wall is a good reason to suspect that there is some link between the wall, changes to the drainage regime and the failure. The Coroner was clearly unconvinced by any of the evidence presented to support this suspicion.

However, with only a vague idea of the relative positions of the wall, the pipeline and the slope failure I would be interested to know whether the impact of the wall construction itself was considered to have any affect on the pipe joints. It strikes me from the description of the wall construction that the installation of the soldier piles and the excavation of the drainage trench – and in particular the use of the temporary sheet piles to support unstable batter slopes - would result in some disturbance.

I'm sure there are many armchair engineers that will debate the various theories and merits of the postulated trigger mechanisms for many years to come. The Coroner was of the opinion that it was the Leaking Water Main theory that was most readily supported by the evidence presented at the inquest. The Coroner goes on to express the belief that the pipe joints had probably been leaking for at least a few months prior to the event. This reader at least was then left to consider whether it was only a coincidence that the wall was completed at roughly the same time.

CONCLUSION

As stated above it was not the Coroner's objective to identify responsibility or blame. There are however some very powerful recommendations. It will be interesting to see whether these recommendations are adopted and given widespread support by the authorities they are aimed at. Even more interesting will be how long it will take for actions and initiatives in Australia to become part of our working lives. Or do we need our own Thredbo?

The Bob Wallace Column

Haven't we done well over the last few months? Geotechnical Engineering has made it onto the front pages of national papers and has been covered in dramatic TV news items. Did we look good, or what?

First up, there was the Nevis Bluff affair. A major rock fall closes the main road into Queenstown and the experts interviewed on the scene state that the road will be opened in three to four days. Three to four weeks later it was still closed. Perhaps we didn't look too good after all.

But having said that it is the TV images that will stick with me longest. Swarms of helicopters buzzing around the sky throwing buckets of water at the mountain. Brave men leaning out of low flying chopper's and throwing bags of explosives at the offending exposures. What drama! What pathos! What nonsense!

I am sure that if we had had a credible air force we would have seen super sonic fighter planes hugging the walls of the Kawarau Gorge and launching sidewinder missiles at the cliff. In fact, now that I think about it, there is every possibility that if Transit had been prepared to go to the expense of painting a large target on the cliff face, the US Air Force may have been prepared to pay them for the practice. With the state of the NZ$ it would not have cost USAF very much and just think of the tourist benefits. The South Island would have been flooded with rednecks for years to come. Birds of a feather and all that.

And then there was the tale of the tail-race on the late night news. The media has finally caught up with the fact that the second tunnel for the Manapouri Tail Race is months behind schedule and millions of dollars over budget. It was all treated very sympathetically by the enquiring Journo's. But the intelligent and free-thinking viewer was left with some nagging questions.

Why, if the first tunnel was designed & built successfully, did they change the method of construction? This is usually only ever done to take advantage of cost savings and production efficiencies.

Why, if the first tunnel was designed & built successfully, is there anything unforeseen or difficult in the ground conditions?

Why, if the first tunnel was designed & built successfully, is there any need for a second? Surely, they didn't deliberately under size the original tunnel with the expectation that mobilising a second time for such a major civil engineering project to retro-fit and upgrade would be economic.

And the big question....Who is paying for the present problems?

To be fair, there maybe perfectly reasonable and readily justifiable answers to these questions. Should we blame the journalist for not painting a balanced picture or should we hang our heads in shame that our message was not delivered to best effect? Perhaps, over the next six months, there will be some examples where the profession displays a somewhat more creditable contribution to society. Or at least considers hiring a Publicist with a PhD on Spin.

Project News

Pavement Imaging Using GPR

Non destructive testing of road pavement using Ground Penetrating Radar (GPR) measurements is an accepted practice in the USA and Europe. The results are presented to the client as profiles showing the thickness and structure of road layering.

For a comprehensive road condition survey, damage analysis or rehabilitation plan, GPR results are combined with the results of bearing capacity (FWD), roughness (IRI) and rutting measurements, as well as other information gathered from the source (e.g. visual inspection, road register data). Analysis results can also be transferred into CAD design software with the appropriate transfer protocols.

GPR is analagous to sonar sounding, but instead of sonar signals, the GPR antenna (T) sends electromagnetic (EM) pulses to the ground, using frequencies of between 10 MHz and 2000 MHz (typically 900 to 1500 MHz for road surveys). Each pulse is reflected at boundaries between materials of different electrical properties (eg asphalt and gravel layer). Some of the energy is reflected back to the surface, and some continues deeper into the ground. The reflected EM signal (waveform) is recorded back at the surface. Specialist processing and interpretation software can then be used to determine layer thickness and determine changes in material properties.

Groundsearch was contracted to perform Ground Penetrating Radar (GPR) surveys over pavement at six sites in Auckland City. The purpose of the survey was to map subgrade thickness and to interpret road structural defects where possible. A combined total of 9970m of GPR data were collected from road sections at Donovan Street, Gloucester Park Road, Howe Street, Mt Wellington Highway, Orakei Road and St Johns Road. Both the left and right hand wheel tracks of each lane were surveyed using the high resolution 1000MHz and 1500MHz antenna. These antennae provide better than 40mm resolution at depths of up to 1m. Test pit information was used to provide depth calibration. This resulted in an average mismatch error of 24mm between the test pit and the GPR section. The maximum error observed was 80mm. Data was presented as interpreted cross sections showing depth and thickness of each layer. Signal amplitude information was also presented, which indicates material quality.

This work is still in progress but current feedback suggested the results are good. The customer has stated that given theright pricing structure they would use this again. The costs for the trial were higher than a standard test pitting survey by a modest margin.

However the GPR method provides a continuous data record of thickness (thickness measurements every 5cm) compared with test pit measurements, which are often spaced many tens of metres apart. There is still some work required to integrate GPR into NZ road investigation methods. Groundsearch have found that the combination of GPR and test pitting can be done for similar cost to conventional methods but with a much higherinformation density.

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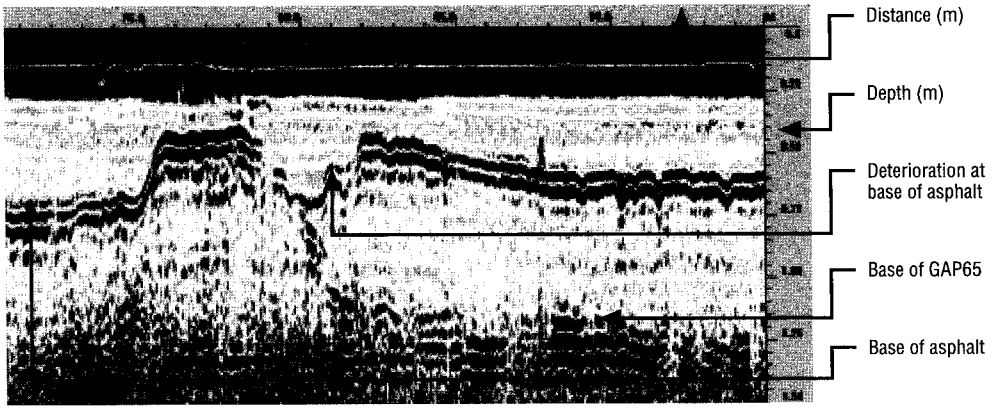

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UV/Filter Piling - Use Of Hydraulic Hammers

Brian Perry was selected by Manukau Wastewater Services Ltd in December 1999 to install the piling for the new $65M UV/Filter Facility which is part of the Manukau Sewerage Treatment Plant Upgrade. This is the largest driven piling project undertaken in Auckland for many years. Construction began in late August after demolition of the existing FGR Tanks, and is currently ahead of programme and due for completion in early November.

An open day was held on 28th September 2000 to demonstrate the latest hydraulic pile driving hammers available and to highlight the high productivities and efficiencies that can be achieved using this equipment.

The key project statistics are:-

- 770No. 360UBP 109(kg/m) & 133(kg/m) piles with a length of between 26 to 30 metres driven to an ultimate load capacity of 2550KN
- 550No. Lx8 and Lx12 permanent sheet piles in lengths from 7.5m to 12m.

Ground conditions consist of up to 5m of fill which overlay up to 4m of recent marine sediments overlying up to 6m of interbedded layers of peat and silty clays. These peat/clay layers overlay up to 10m of sands on top of the Kaawa Formation which consists of very stiff silty clays and dense sands in which the 'H' piles are founded.

Piles are driven using a 4 tonne Junttan hydraulic hammer on crane mounted leaders to install the initial 18 metre length of 'H' pile prior to splicing. After splicing the upper 8-10 metre section, the final drives are achieved using a 9 tonne Junttan hydraulic hammer in 'flying leader' mode. The piles are designed to achieve the ultimate loads in end bearing and sets are being determined using the Hiley Formula.

The Junttan hydraulic hammers have a number of advantages over conventional diesel, air or drop hammers these are:-

- Higher efficiency due to accelerated drop (up to 20% more than free fall hammer of the same weight.
- Greater accuracy and efficiency due to remotely controlled variable drop height and rate of blow per min. These are adjusted according to the soil conditions and type of piles being driven.
- Greater control and ease of operation making the driving process much safer.
- Environmentally clean, no diesel or exhaust gas emissions making them well suited to urban environments
- Low impact noise.

Sheet piles are being installed using an excavator mounted ABI vibrating hammer and are finished off, where required in difficult ground, with a crane mounted ICE 416 high energy vibro hammer.

Pile installation is on the critical path for this project and the project has been resourced to achieve a target production of 15 Nr completed H piles per day. The spread consists of 6 cranes plus the ABI for sheetpile installation, 2 hydraulic hammers, 2 ICE vibro hammers, a piling crew of 20, a welding crew of 10, plus associated survey and weld inspection staff. Splice welds are 10% ultrasonic tested, 25% visually examined and 100% visually scanned.

Further information on the Junttan Hydraulic piling hammers can be obtained from:-

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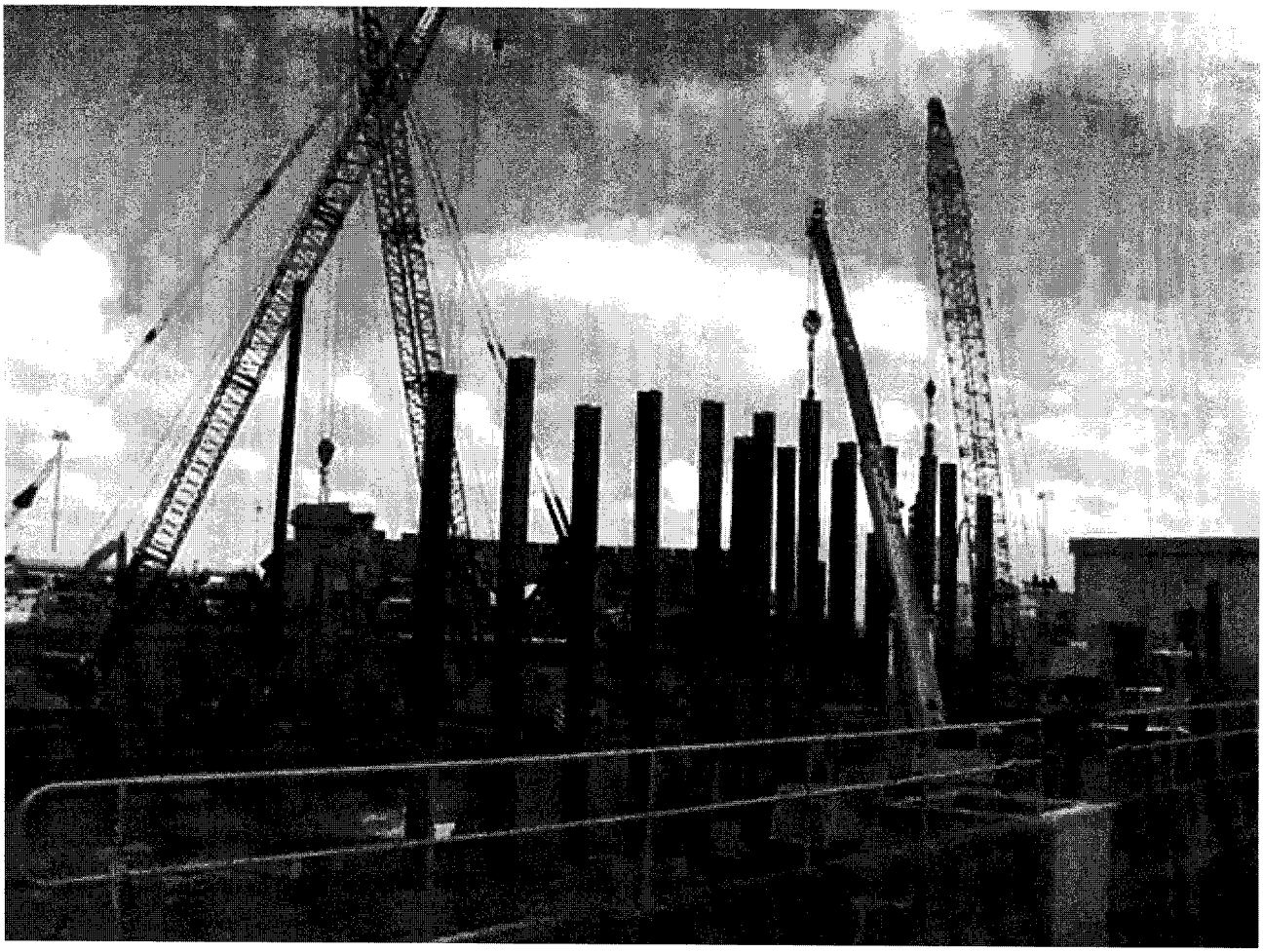

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Like Tonkin & Taylor we are also involved in a never ending search for technical and procedural advancements in construction, but sometimes the answer lies not in our own research facilities but in places on the other side of the world, places like Scandinavia for instance. Because that's where a product that's proven itself in the construction of Sub Arctic roads also became the solution to a building problem in Auckland.

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Martin Fahey, Project Manager of Mainzeal said that, "at the end of the day Polyrock® by James Hardie, being both timely and cost effective, proved to be the perfect solution."

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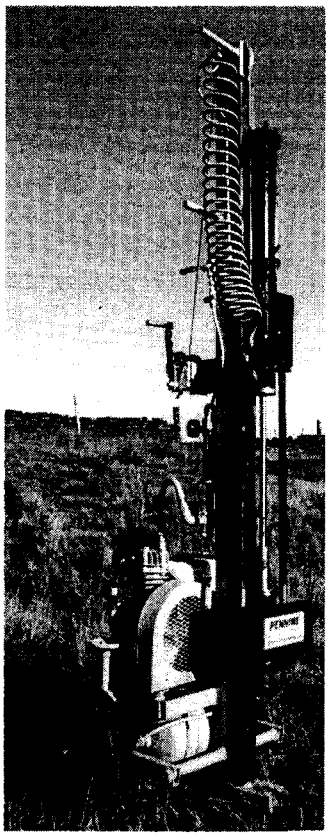

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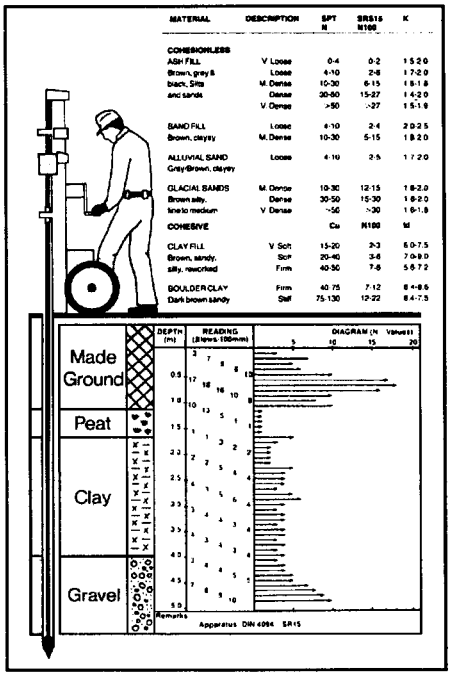

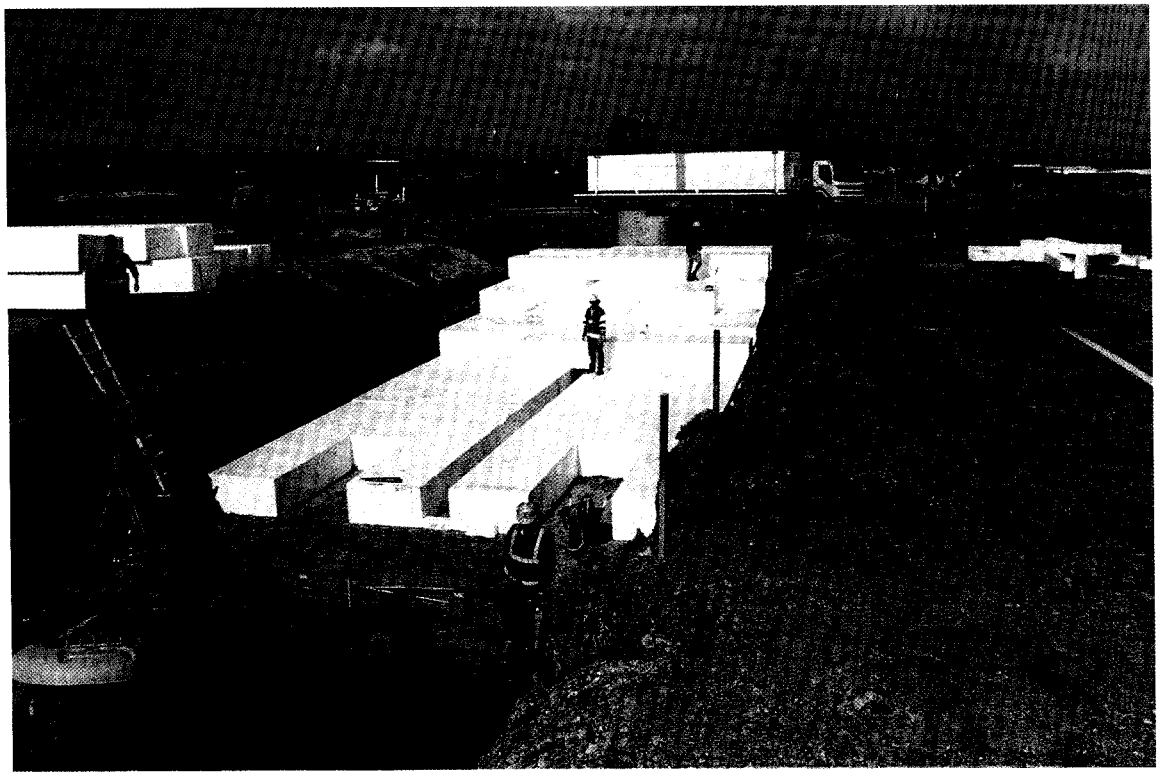

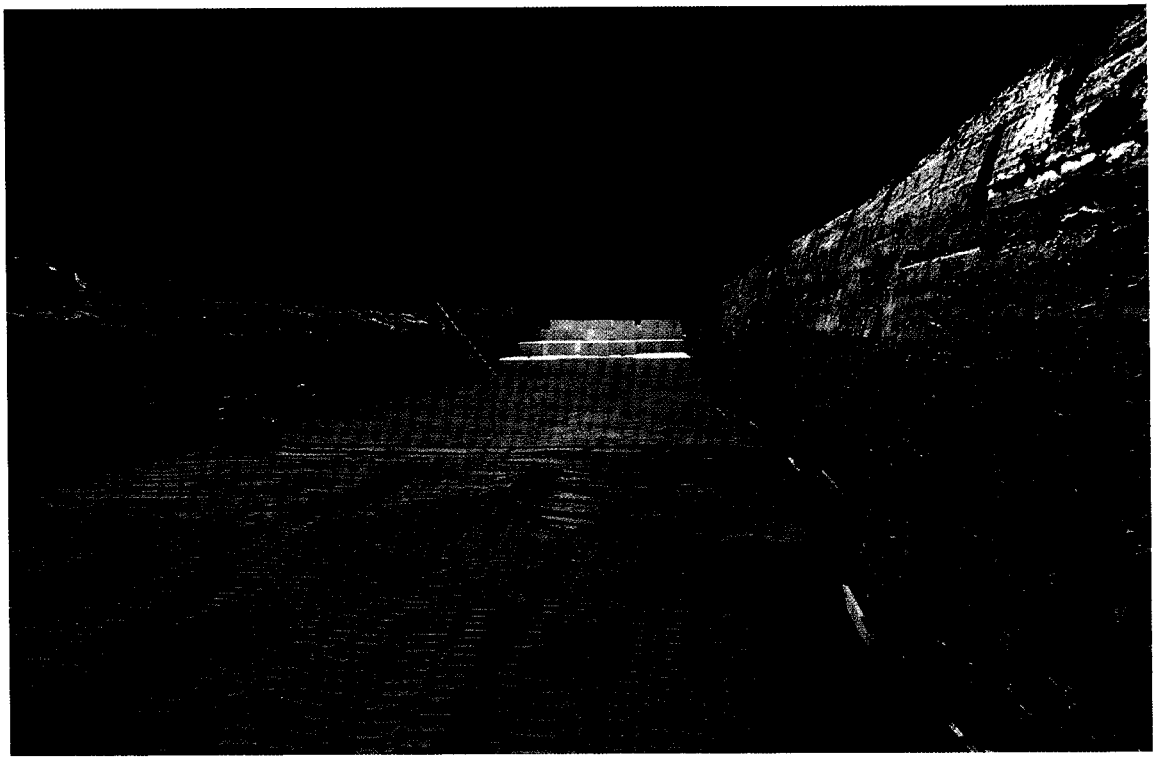

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REFERENCE

1. Peck, R.P., "Influence of Non-Technical Factors on the Quality of Embankment Dams," Embankment Dam Engineering, Casagrande Volume, R.C. Hirschfeld and S.J. Poulos, eds., London, 1972.
2. World Bank Energy Department, "Geological Complications and Cost Over-runs: A Survey of Bank-FinancedHydroelectric Projects," Energy Department Note No. 61, 1985.
3. Nairne, R.S., "Appropriate Risk Allocation in Construction – A Possible Dream?" The BC Professional Engineer, January 1984.
4. Sinclair, B.J., "Design of Site Investigations," Proceedings, H.G. Acres Seminar, Niagara Falls, Ontario, 1981.
5. Kinsten, H.A.D., and E.L. Giles, "Cost Benefits of Pre-Tender Exploratory Investigations," Proceedings of the SANCOT Symposium, 1986.
6. International Tunnel Association Working Group, "ITA Recommendations on Contractual Sharing of Risks," Tunneling and Underground Space Technology, Volume 3, No. 2, 1988.

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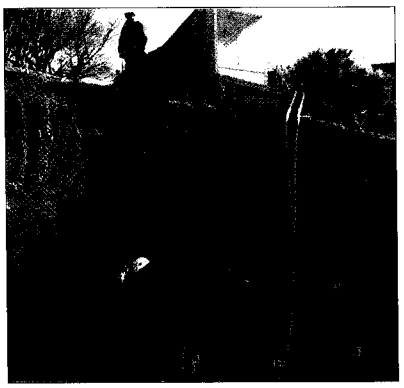

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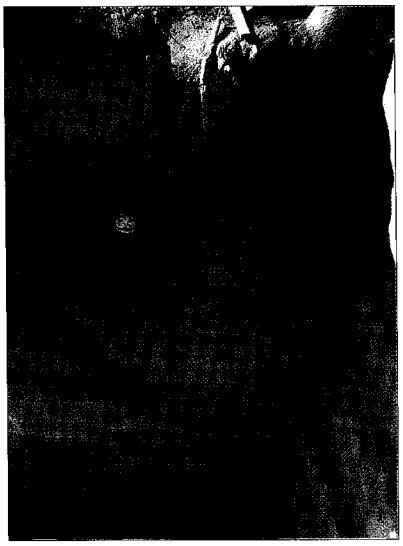

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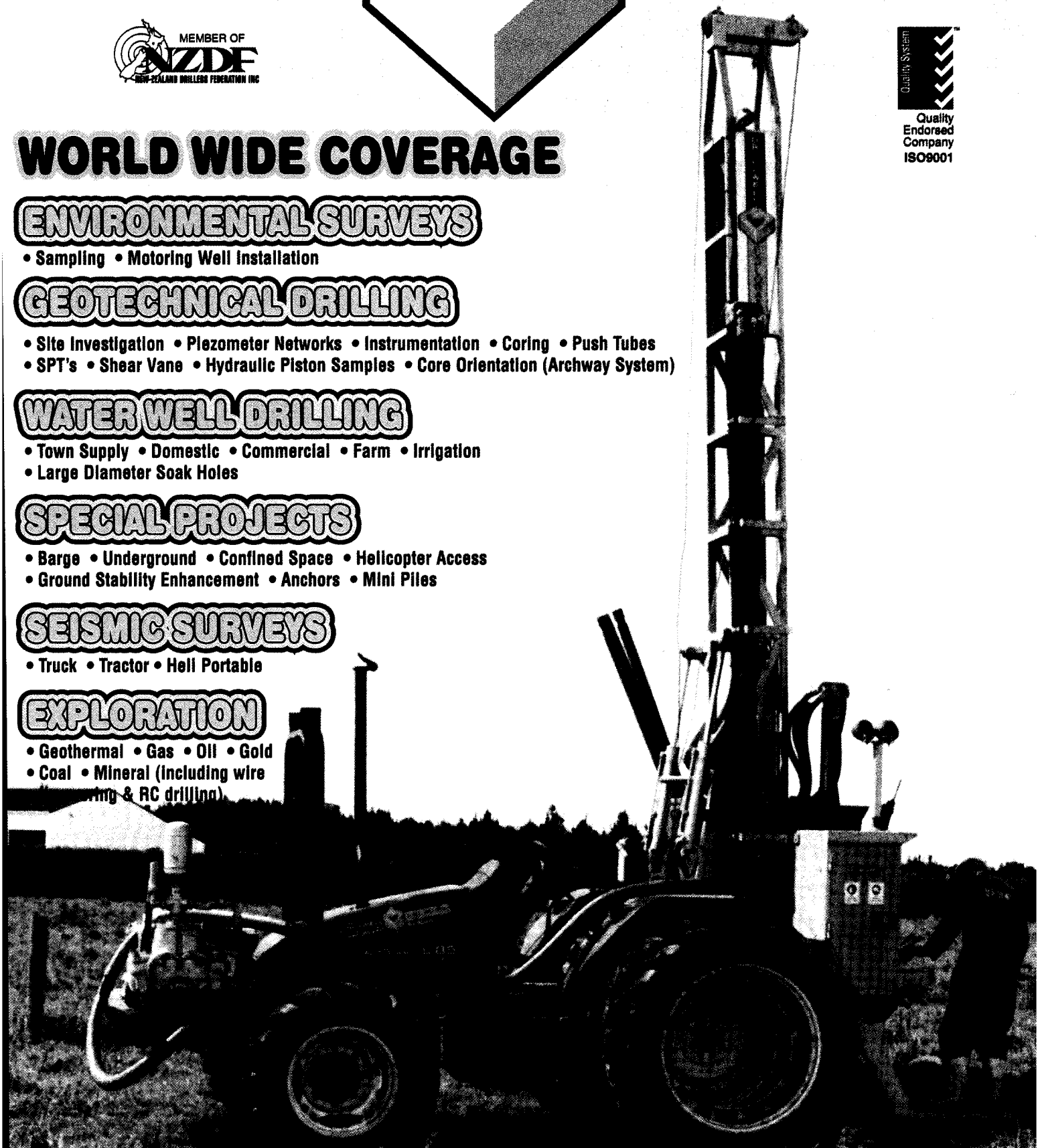

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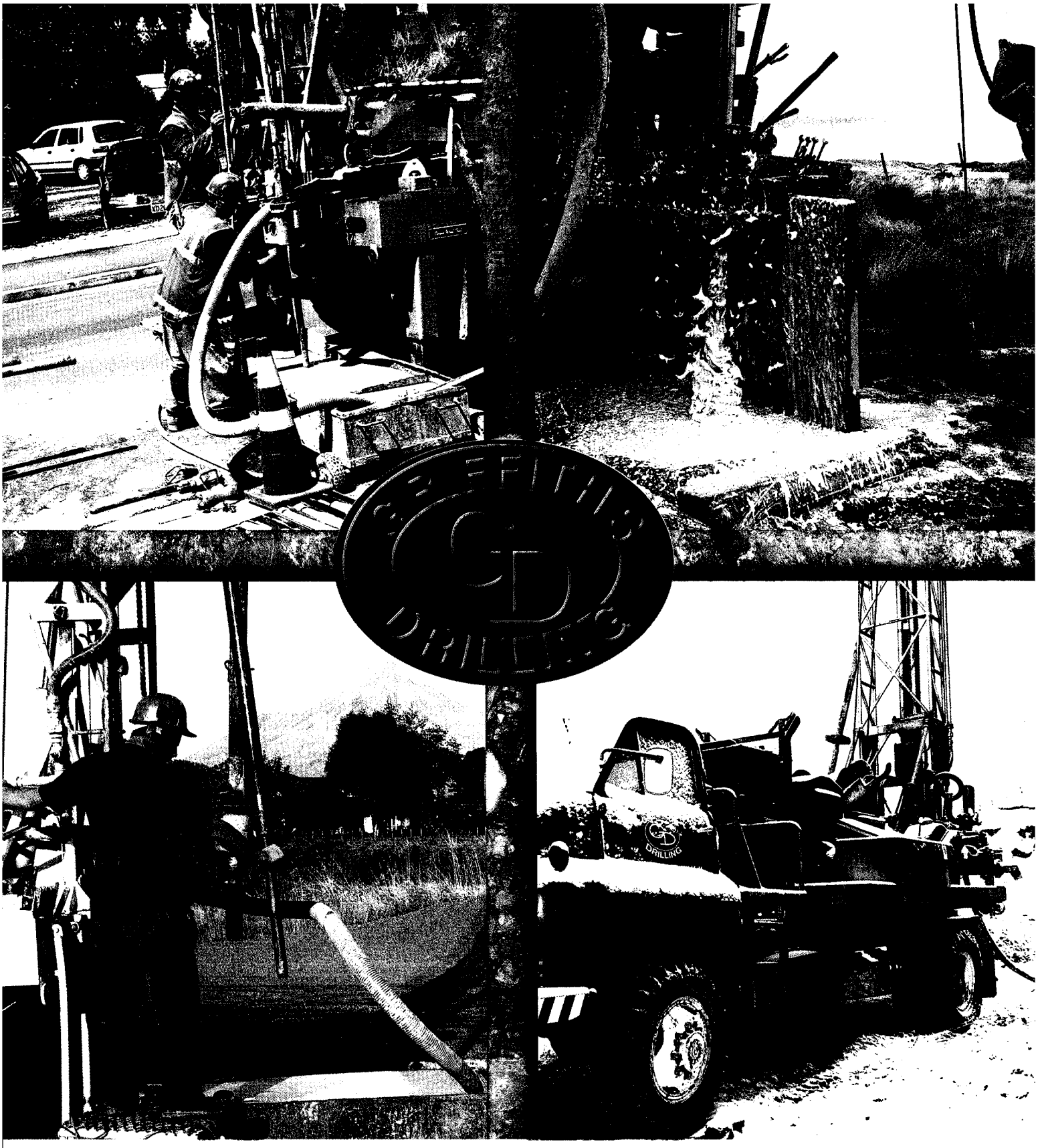

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Auckland Residual Soil - Compressibility Measurement

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INTRODUCTION

The conventional understanding of the compressibility of soil has long been based on data from laboratory prepared artificial materials and the behaviour of undisturbed samples of sedimentary soils. Quite distinct, both in formation processes and mechanical behaviour, are residual soils formed by in situ weathering from parent materials, usually rock, but sometimes other soils. This paper presents initial results from laboratory testing of residual soils from the Auckland area. The main focus is compressibility as measured in one dimensional compression but also inferred from the Young's modulus and Poisson's ratio values measured in drained triaxial compression. One difficulty is the rapid variation in properties of the soils from position to position in the soil profile and even within a sample.

SAMPLING AND INDEX PROPERTIES

The samples tested were taken from two sites around the Auckland area, one in South Auckland and the other on the North Shore. The geological origin of the soil is not certain, but is referred to herein as residual, there being much residual soil in the Auckland area. The possibility of Pleistocene origin cannot be discounted completely, as the distinction between residual and Pleistocene deposits is often not easily made around Auckland.

Both hand-cut block samples and large diameter push-tube samples were obtained. Typical water content and Atterberg limit values are given in Table 1; note the large range in water contents, even though the samples were taken from confined areas no larger than about 5 metres square a few metres beneath the ground surface at the base of a digger excavation.

Soil location	Water content (%)	LL (%)	PL (%)
South Auckland	34 - 45	55-60	28-33
North shore	16-39	48-56	26-32

Table 1. Index properties of the soils tested

EQUIPMENT

Laboratory specimens were hand trimmed with jigs prior to testing. Oedometer specimens 75 mm in diameter and 19 mm tall were tested in a conventional oedometer using a load increment ratio of unity. The oedometer set-up used the fixed ring arrangement. Specimens 75 mm in diameter and up to 140 mm tall, saturated by the application of a back pressure of 700 kPa, were tested in a K_o triaxial cell. This cell follows that described by Davis and Poulos (1963) in which a triaxial cell is modified by installing a loading ram having the same diameter as the specimen and replacing the conventional transparent plastic cell wall with a stiff steel wall. The concept of the cell is illustrated in Figure 1.

A pressure transducer is installed in the cell pressure line and, once the set-up is complete, testing is done with the cell pressure line closed and the cell pressure monitored. The water in the cell can be regarded as incompressible as the 700 kPa back pressure compresses any air trapped in the cell when it is filled with water, the air in these reduced bubbles then diffuses into the cell water. Thus, when loaded with a piston the same diameter as the specimen, the soil is constrained to deform one dimensionally. Drainage occurs from the top of the specimen and the pore water pressure is measured at the bottom, thus the drainage path length is the full height of the specimen, an advantage in determining the coefficient of consolidation in soils with high c_v values such as these residual soils.

OEDOMETER TEST RESULTS

Figure 2 presents the results of a number of conventional oedometer tests for specimens from the South Auckland site, the constrained compression of the specimen is plotted against the vertical stress scale on a natural scale. Figure 3 has the same data plotted against a logarithmic stress scale. It is clear from the natural scale stress plot that the variation in compressibility of these materials for vertical stresses up to about 1700 kPa is modest and yet apparent preconsolidation pressures could be inferred from the plots with the logarithmic stress scale. Estimating these consolidation pressures and going back to the natural scale plot one finds almost no

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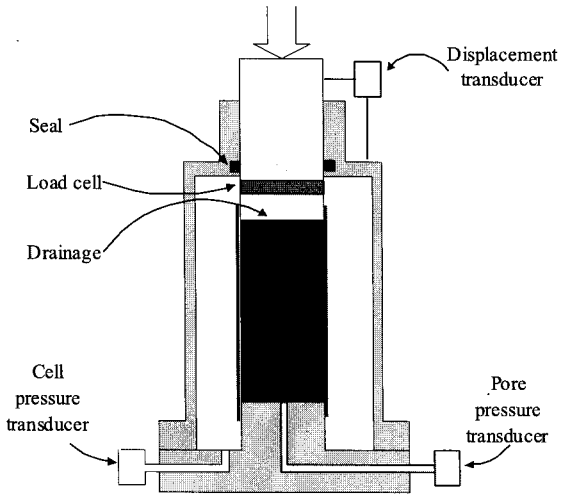

Figure 1. The concept of the K_o triaxial cell

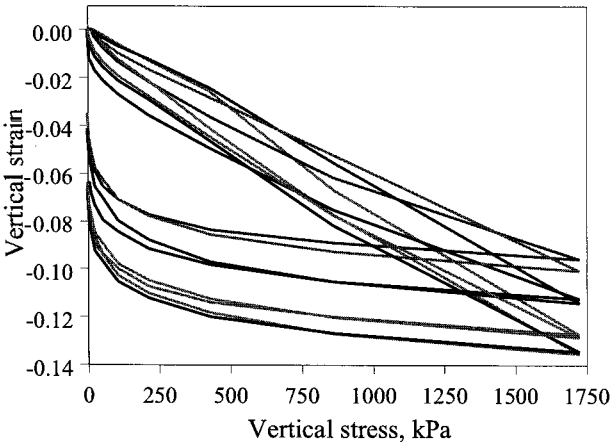

Figure 2. Oedometer test results for the South Auckland site plotted with a natural stress scale

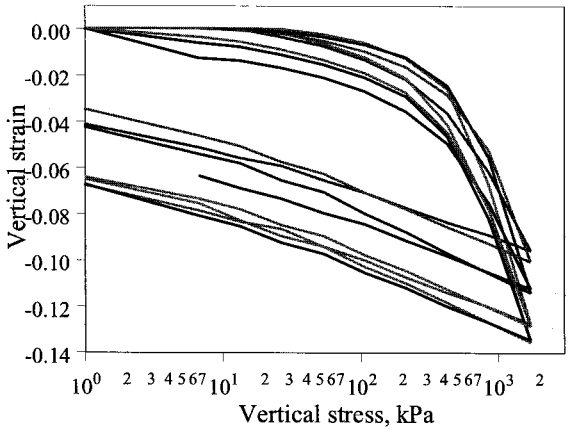

Figure 3. Oedometer test results for the South Auckland site plotted with a logarithmic stress scale

suggestion of yielding or evidence of any change in the compressibility at these stresses, although two of the curves show a slight increase in compressibility after a stress of about 400 kPa. As the soils are of residual origin there is no reason to expect a preconsolidation pressure for the material. Thus the apparent preconsolidation pressures are primarily an artifact of the plotting axes used!

Some of the compression curves in Figure 2 show a change in slope up to vertical stresses of about 200 kPa. This is likely to be a consequence of bedding errors and defects in trimming the specimens which produce some small gaps between the containing ring and the soil. The initial larger compressibility reflects the lack of constraint as the specimen deforms to fill these gaps and any bedding-in between the soil and the porous stones. If a correction is made for this effect then most of the differences between the loading parts of the curves in Figure 2 are accounted for. The settlement of shallow foundations depends on stress changes between a few tens of kPa (the in situ vertical effective stress) and a few hundred kPa. It is in just this stress range that errors from bedding and lack of constraint are most significant. Consequently, the conventional oedometer test is not the most appropriate tool for estimating the settlement of shallow foundations in residual soils.

In Figure 4 the Figure 2 data are replotted in terms of void ratio rather than vertical compression. From Figure 2 we see that there is little difference in the compressibility of the samples and yet from Figure 4 significant differences in the initial void ratios leads to a loss of clarity in the diagram as the various loading and unloading curves overlap.

Another set of data is presented in Figure 5, this time the secant constrained modulus, between 200 and 600 kPa, for specimens from the North Shore site, is plotted against the initial water content. Figures 4 and 5 show that the compressibility or constrained modulus does not correlate closely with water content or void ratio. This demonstrates the inherent variability of these residual soils, a factor which is clearly apparent even within the confines of a roughly cubical 0.4 metre block sample.

K_o OEDOMETER TEST RESULTS

Figures 6 and 7 present a compilation of test results on specimens from the South Auckland site (Fig. 6) and the North Shore site (Fig. 7). In these diagrams there are data from conventional oedometer tests, K_o triaxial tests, and constrained modulus values calculated from the results of drained triaxial tests (discussed in the next section).
Whereas the earlier diagrams presented plots of vertical compression against vertical stress, these diagrams plot the constrained modulus, that is the slope of the compression - vertical stress curves. In Figure 6 the secant constrained modulus, ie the slope of the straight line between the initial and current stress point on the compression curve is plotted. In Figure 7 the tangent constrained modulus, ie the slope of the compression curve at the current stress value is plotted.

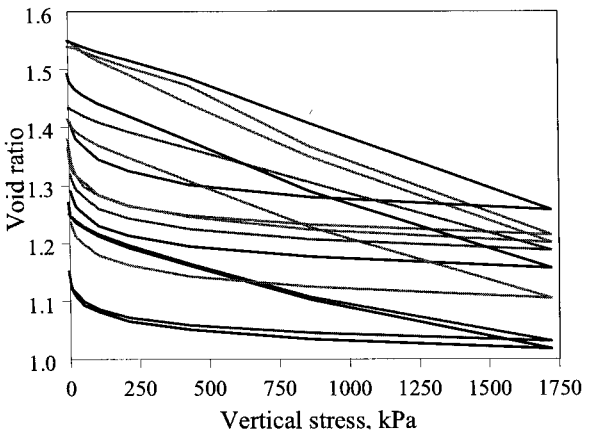

Figure 4. Oedometer test results for the South Auckland site – void ratio versus vertical stress (natural scale)

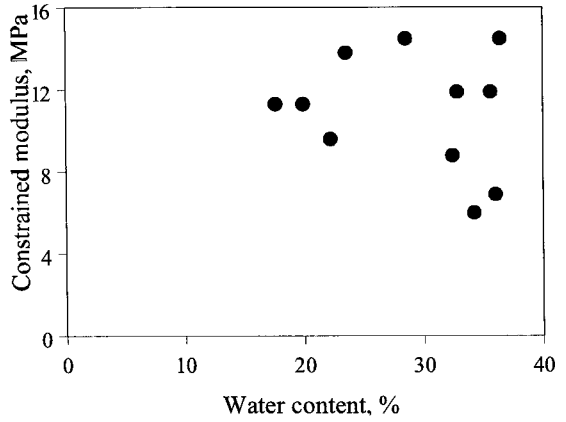

Figure 5. Constrained modulus – water content data for oedometer tests from the North Shore site

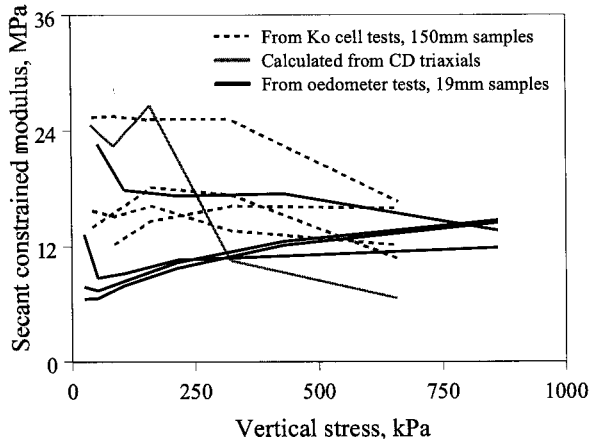

Figure 6. Secant constrained modulus data for the South Auckland site. Oedometer results, K_o triaxial cell results and values calculated from the drained triaxial data.

These curves show that the constrained modulus is not quite constant with increasing vertical effective stress, in some cases it increases slightly as the vertical effective stress increases and in others it decreases slightly. The diagrams illustrate once again the variability of the soils, but both show that the constrained modulus determined from the K_o triaxial cell is greater than that from the conventional oedometer test, particularly in the low vertical stress range. This is presumably a consequence of the lesser significance of bedding errors for the K_o specimens as they are much longer than the 19 mm oedometer specimens, also because there is no initial error associated with the lack of fit in the confining ring. From the point of view of estimating the settlement of shallow foundations these differences between the constrained modulus determined with the oedometer and K_o triaxial cell are significant.

One important difference between the two tests is, of course, the back pressure saturation of the K_o tests. However, this is not thought to be the reason for the differences in constrained modulus but, as explained above, bedding errors and lack of fit are considered a more likely explanation.

The initial testing with the K_o cell also anticipated that bedding errors might be significant, so the compression of the soil was measured with on-specimen displacement transducers as well as with an external transducer. The results showed no significant difference between the constrained modulus values from the internal and external measurements, so the use of internal measurements was discontinued.

DRAINED TRIAXIAL TEST RESULTS

Conventional constant cell pressure drained triaxial tests were done on saturated specimens, 75 mm by 150 mm. The stress-strain curves for North Shore specimens are plotted in Figure 8. Once again the specimen to specimen variability is apparent in that the stiffness of the specimen consolidated at 400 kPa is less than that for the specimen consolidated at 200 kPa, similarly one of the 100 kPa specimens has a stiffness less than that of the 50 kPa specimens.

Figure 9 plots, for specimens from the South Auckland site, the stress paths for the drained triaxial and the K_o triaxial tests. The tangent values of Young's moduli and Poisson's ratios are calculated for the drained triaxial tests where the stress paths cross. Then the tangent constrained modulus is calculated using the following equation:

$$M = \frac{E(1-\nu)}{(1+\nu)(1-2\nu)}$$

where: M is the constrained modulus,
E is Young's modulus,
and ν is Poisson's ratio.

The constrained modulus values so obtained are plotted in Figures 6 and 7. At this stage it is not clear if this is a viable method of estimating soil compressibility for estimating the settlement of shallow foundations. What is clear though is that the drained triaxial approach does give constrained modulus values closer to those obtained with the K_o triaxial

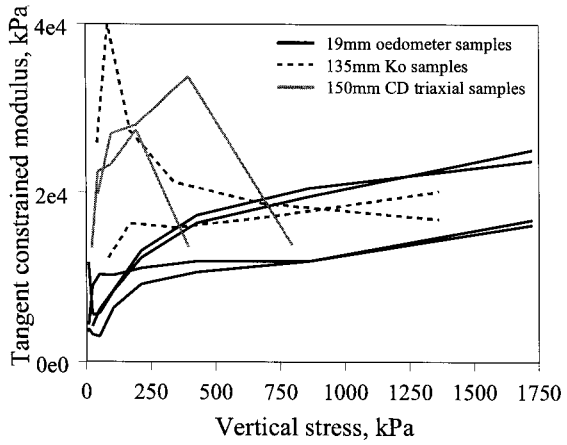

Figure 7. Tangent constrained modulus data for the North Shore site. Oedometer results, K_0 triaxial cell results and values calculated from the drained triaxial data.

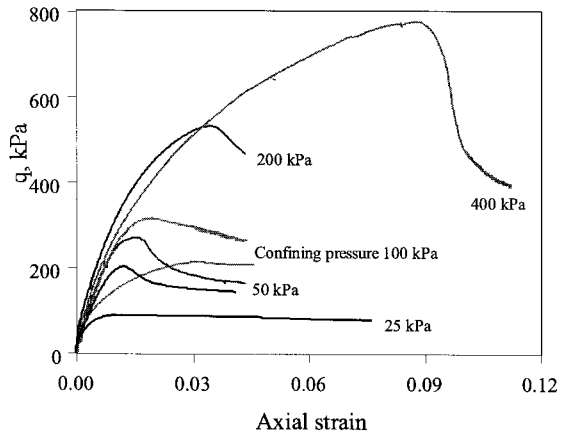

Figure 8. Drained triaxial stress-strain curves from tests on the North Shore samples

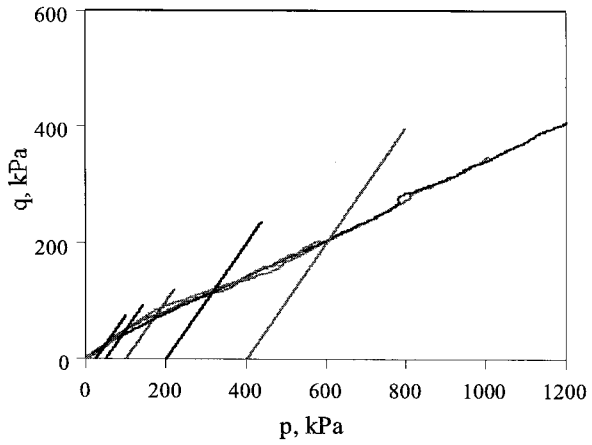

Figure 9. Effective stress paths from constant rate of strain K_0 tests with drained triaxial stress paths superimposed for samples from the South Auckland site. [$q = (\sigma_1 - \sigma_3)/2$, $p = (\sigma'_1 + \sigma'_3)/2$]

cell tests than the conventional oedometer tests. The differences may be a further consequence of the difficulty, because of specimen to specimen variability, of obtaining several specimens with closely similar properties. They might also reflect the need to improve the methods used to estimate tangent stiffness values from laboratory stress-strain curves, current work is looking into this.

DISCUSSION

The loading parts of curves in Figure 2 suggest that the compression of the soil occurs at nearly constant stiffness, particularly if corrections are made for the bedding error effects discussed above. The constrained modulus values plotted in Figures 6 and 7 also show that there is only a modest change in the stiffness with increasing effective stress. However, in all cases the unloading stiffness is very different from the value during loading. Thus even if the compression behaviour of the soil is close to linear the soil itself is not a linear elastic material.

At present this work is ongoing, more information can be found in Pender et al (2000).

CONCLUSIONS

Three major conclusions are reached in this paper:

(i) For Auckland residual soils (and indeed for all residual soils), it is desirable that one dimensional compression data be plotted on linear as well as logarithmic scales. The former are necessary to establish whether a "preconsolidation" pressure actually exists. In the case of the soils tested here no such pressure was found.

(ii) The K_0 triaxial cell, by virtue of the greater length of specimen, gives a more accurate value for the constrained modulus than that obtained from conventional oedometer tests.

(iii) The point to point variability of water content and void in these soils is considerable. Although the compressibility is also variable, these variations do not seem to be related to variations in water content.

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Staff of the Auckland office of URS - Woodward Clyde provided information about and facilitated access to the two sites from which samples were recovered.

REFERENCE

Davis, E. H. and Poulos, H. G. (1963) "Triaxial testing and three dimensional settlement analysis", Proc. 4th. Australia New Zealand Conference on Soil Mechanics and Foundation Engineering, Adelaide, pp. 233-243.

Pender, M. J., Wesley, L. D., Twose, G., Duske, G. D., and Pranjoto, Satyawan (2000) "Compressibility of Auckland Residual soil", Proc. GeoEng2000, paper 1082, Melbourne, November.

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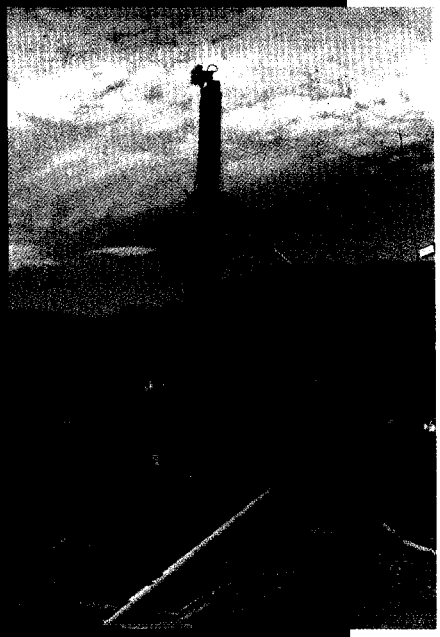

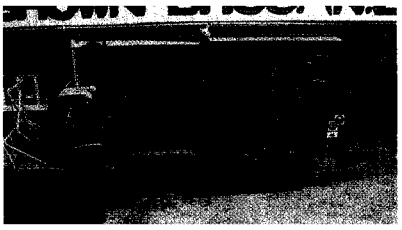

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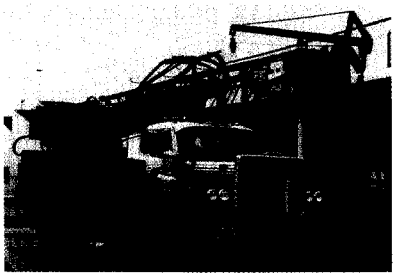

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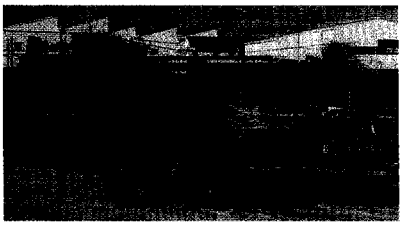

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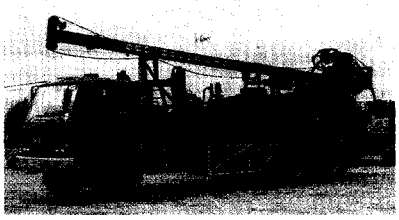

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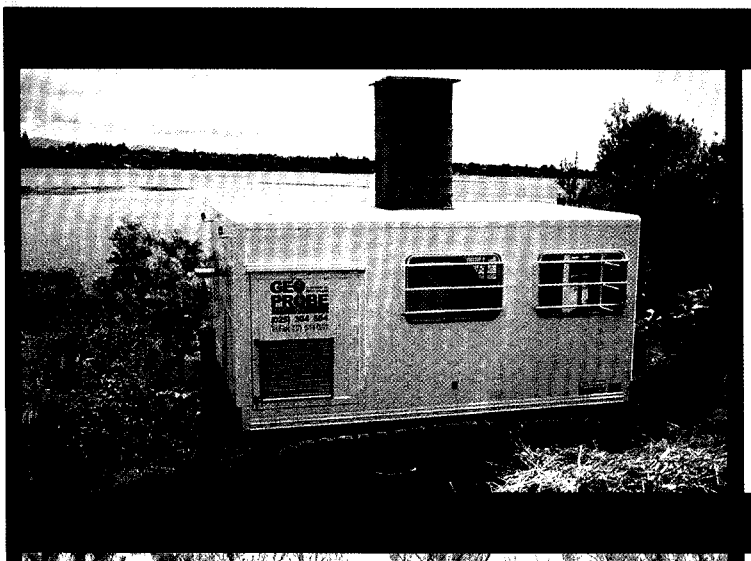

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Quantitative and Non-Quantitative Methods of Estimating Slope Stability

Discussion Paper by Laurie Wesley

INTRODUCTION

While travelling recently, I gave some thought to the question of probabilistic methods of slope stability evaluation, traditional safety factors, and their place in relation to non-analytical methods. I have become somewhat concerned at the increasing faith which geotechnical engineers are tending to put in the numbers produced by analytical methods of slope stability, especially the traditional approach using safety factors. I will start with some comments on safety factors, followed by remarks about probabilistic methods, focussing on a couple of papers from the Symposium on this subject that was held in Auckland in March, 1999.

SAFETY FACTORS AND PROBABILITY OF FAILURE ETC.

My concern here is twofold:-

- Firstly, I am concerned that engineers are putting more faith in analytical answers than in the evidence which visual observation and common sense tell them. If a slope has not moved for several hundred years and shows no evidence of past instability, and is not being affected by engineering works or any other changes in its environment, then it is clearly a slope with a low risk of slip movement. If site investigation and slip circle analysis comes up with a safety factor of 1.1, this doesn't change the evaluation. The slope doesn't suddenly become a high risk slope because of what some number produced by an analytical process says. The fact that the slope has not moved for hundreds of years is a much more telling piece of information than the calculated value of the safety factor. Of course the value of 1.1 would be a warning to the engineer not to meddle with the slope in a way that would make it less stable.
- Secondly, there seems to be a rather blind adherence by geotechnical engineers to preconceived ideas about acceptable values of safety factors, without recognition of the risks these imply, and the need to tailor the values (of the safety factor) to each project, depending on the acceptable level of risk. I am thinking particularly of the value of 1.3 that is frequently quoted as an acceptable value during construction. I do not think it is at all sensible to adopt a safety factor of 1.3 for the construction or the "end of construction" situation without careful consideration of the justification for doing this. If it is the contractor who is accepting the risk, and no one else is affected should failure occur, then that is fine. But if the engineer is adopting 1.3, and the client is blissfully unaware of the fact that this entails risk to his/her project then that is a very different matter. Clients don't know anything about geotechnical engineering. They finance a project in the same way that they buy a car – they expect it to work. If a low safety factor is adopted and a failure occurs, the engineer cannot excuse himself/herself on the basis that it is "accepted practice" to adopt low safety margins during construction. This is like telling the client that it is "accepted practice" to produce faulty products.

It needs to be recognised also that in many cases the "end of construction" safety factor may in fact be the value applying over much of the life of the structure. An embankment built on soft clay, or an embankment built of low permeability clay, may have an "end of construction" safety factor of 1.3, and a "long term" value of 1.7, this latter value applying after full pore pressure dissipation. However, if the time for pore pressure dissipation is years or decades, then the short term (end of construction) value will also apply over a substantial part of the life of the completed structure.

PROBABILISTIC METHODS - THOUGHTS ON THE PAPERS BY MOSTYN & FELL AND MOSTYN & LI

Statistics and probability are not fields I know much about. I took the above two papers with me to read while away, in the hope of becoming a little better informed. Having read them, I still don't really know much more about the mechanics of probabilistic methods - as soon as the authors started producing equations, and talking about *explicit performance functions* and *spatial autocorrelation*, I had to back off. Hopefully, however, I understood enough of the papers to be able to think some rational or semi-rational thoughts on the subject. They include the following:

1. ***Two equally valid methods?*** In the first reference, a presentation is made of what are called "Quantitative" and "Semi-quantitative" methods for estimating the probability of landsliding. "Quantitative" refers to analytical or "deterministic" methods, especially those involving probabilistic theory, and the second to the use of observational methods. Two comments come to mind:-

    - The quantitative method is restricted to probabilistic analyses, with only brief passing reference to the traditional safety factor method. It is clear from the papers that probabilistic methods have a very long

way to go before they can be used as routine tools, and therefore engineers are stuck with the safety factor method. A discussion of quantitative methods ought therefore to give a bit more attention to the safety factor method than is done in the first of these two papers.

- Secondly, a description is made of semi-quantitative methods as though these are an alternative to the quantitative methods. The statement is made in the conclusion of the second reference that "Quantitative and semi-quantitative methods for estimating the probability of landsliding are well developed, and are equally valid" (my emphasis). This seems to me to be quite a misleading statement. Firstly, it is misleading because I don't think the methods are equally valid, and secondly because it implies that one can choose between the methods. Semi-quantitative methods must always be part of the evaluation of landslide risk. We can never leave out visual inspection, geological appraisal, aerial photograph examination, etc, and hang our hats on some number producing method. In other words the starting point, and the most important part of any slope stability evaluation, is always "non-quantitative". We may prefer to think of it as a semi-quantitative method, because of our enthusiasm for numbers. The important point is that the quantitative part of any slope analysis could be left out, but not the semi-quantitative. It is probable that the authors of the paper did not intend the paper to be interpreted the way I am interpreting it, but that is the way it reads.

It might be better to refer to analytical and non-analytical methods, rather than the division into quantitative and semi-quantitative methods, because the most important component in any slope stability evaluation is neither quantitative nor semi-quantitative – it is simple observation, completely devoid of numbers.

2. ***Most slope stability analysis based on deterministic methods?*** In the second reference, the statement is made that "Most slope stability analysis and design is based on "deterministic" approaches, ie a set of design parameters is adopted and the loads and resistances are calculated based on these. A factor of safety is then determined as the available resistance divided by the applied load."

- I do not think this is a true statement. It is only true in relation to built structures like embankments and earth dams. I would be very surprised if it were true of the evaluation of natural slopes. I think most geotechnical engineers would put more faith in visual inspection and geological appraisal etc, than the results of any sort of deterministic methods.
- During the approximately 23 years I worked in engineering practice, I don't think I once evaluated the stability of a natural slope using deterministic methods ie using slip circle analysis. In particular, while I was with Tonkin and Taylor, it was the very definite policy of the company that for natural slopes, visual inspection, site history, aerial photographs, geological appraisal took precedence over slip circle analysis. I was personally involved in evaluating the stability of natural slopes for numerous house sites and subdivisions, and I don't recall ever using number producing methods.
- I have taught my students ever since coming to university, that in evaluating the stability of a natural slope, the factors mentioned above (visual inspection, site history etc), will make up at least 95% of the input used in evaluating the stability of the slope. Slip circle analysis may possibly add an additional 5%. To put it another way, the final evaluation is a matter of judgement, and the "weighting" used in making that judgement is heavily in favour of non-numerical material.
- It follows on from what I have said above that it is not true to imply or state (as Ref 2 does) that risk evaluation or estimation of probability of failure is something new and tending to take the place of the "safety factor" approach. In evaluating house sites, our basic approach ought to always be one of assessing the level of risk, without trying to relate it to some value of safety factor.

3. ***A simple case?*** Let us consider a simple case. The figure below shows a site where Fred Smith plans to build a house. Before proceeding he wants to be assured that his

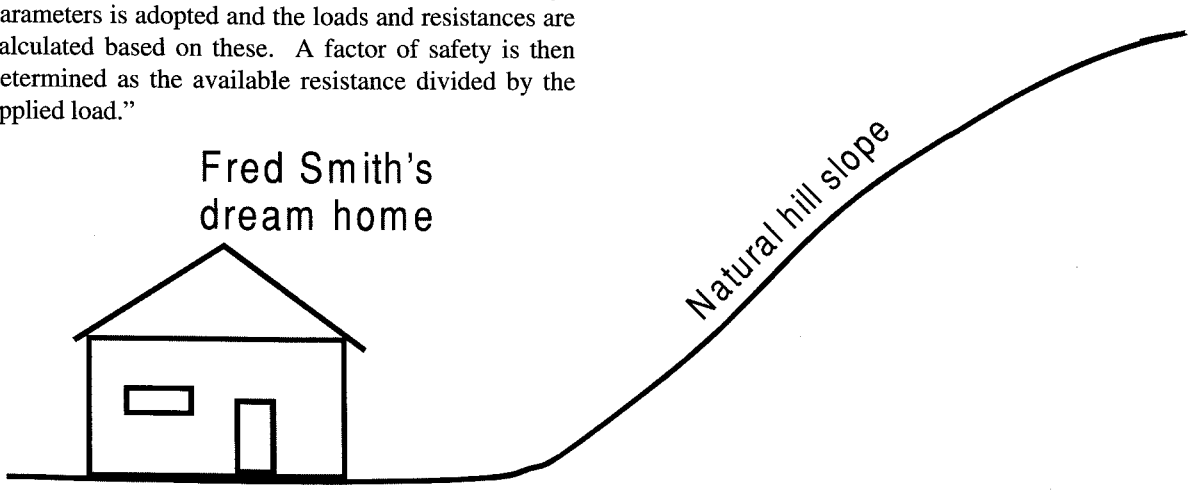

Figure 1. Fred Smith's house site and slope

house will not be at risk from a possible slip in the slope above the house. (In fact, this is not quite correct, Fred Smith hasn't even thought of the possibility of a slip in the slope, but the local Council has, and require a stability report from Mr Smith).

Visual inspection of Fred's property, as well as inspection of aerial photographs of the site, show no evidence of instability, and there are a number of slopes in the neighbourhood with similar inclination, none of which show signs of instability. Some of them have houses built on them. The geology of the site is not of a type which suggests there is a high risk of instability. There are no foreseeable situations where changes of some sort could significantly alter the stability of the slope.

It seems to me that this is the end of the matter. A geotechnical engineer would rightly conclude that the slope is of sufficient stability to allow the construction of Fred Smith's house, and report accordingly. If a geotechnical investigation and the application of a deterministic method came up with any other answer than the above, then it would have to be discounted or at least given very low "weighting" compared with that coming from the other evidence. In other words, the semi-quantitative method wins hands down; the quantitative method can produce what it likes, but it cannot compete with the hard evidence of simple observation.

4. ***A theoretical flaw?*** There seems to me to be a theoretical "flaw" in the application of probabilistic methods to existing slopes, such as that at Fred's house, (ie slopes that are not going to be affected by any engineering works). As I understand it, the probabilistic method produces a probability of failure, commonly expressed as a percent. This percent is a fixed value that is not time dependent. This is to be expected because the data on which it is based does not vary with time. The method simply assumes distributions of some sort in the input parameters with respect to space but not with respect to time. This means that as far as Fred Smith's slope above is concerned, the probabilistic method can say nothing at all about its probability of failure, except by ignoring the most important item of data available, namely that the slope is stable. Figure 2 below shows a graph of the type produced by what I think is called the Monte Carlo method, showing frequency versus safety factor, or some other "performance function". The vertical line indicates the value needed to maintain stability. The safety factor at Fred's site is clearly above the margin necessary for stability. So none of the data to the left of the dividing line can enter into the analysis, as the physical evidence clearly shows it is not present. If this is the case then all the data normally used as the basis for a probability of failure analysis (ie the data to the left of the dividing line) has to be rejected. Thus the logic on which probabilistic analysis of existing stable slopes is based appears to be flawed. And if the slope is unstable, then the probabilistic analysis is not needed, as the answer is already known.

5. ***Before and after completion:*** Following on from the above, is it then the case that once an earth structure has been completed, or a cutting made, the results of the probabilistic analysis must be put aside, provided of course that the structure is successfully completed? If the structure is successfully completed then it means that it exists to the right of the dividing line in Figure 2. Since the input parameters do not vary with time, the structure will always be stable in terms of normal probabilistic theory.

6. ***The pore pressure issue.*** At present, as the authors of the second reference concede, there does not appear to be any substantial difference in the way the pore pressure is entered into the probabilistic analysis, in comparison

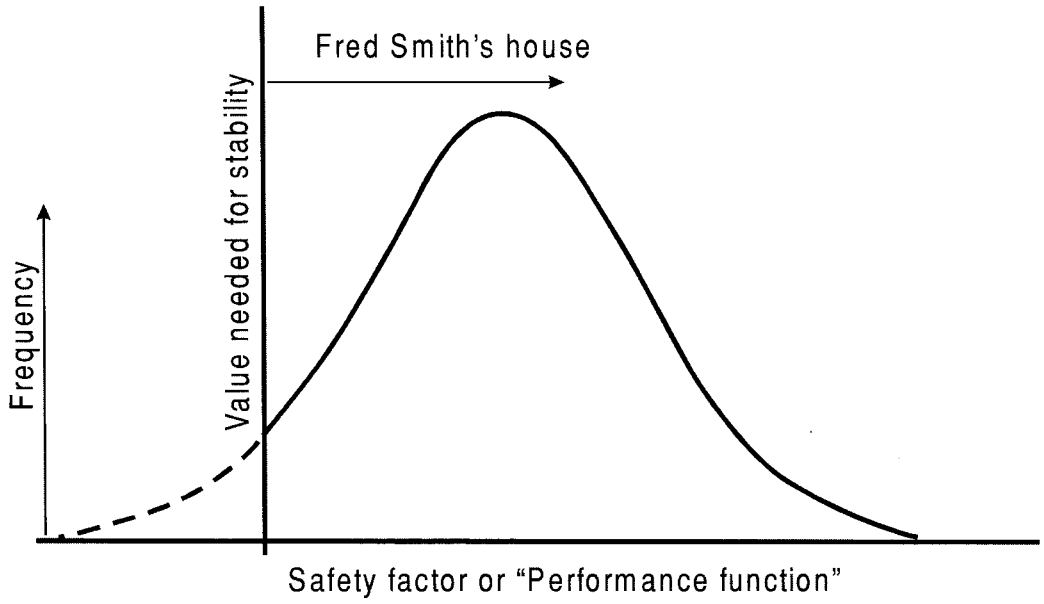

Figure 2. Probability distribution

to the other parameters. This seems a very severe defect, as it is clear that the one parameter that will almost certainly vary with time is the pore pressure. Indeed, it is this very change with time in pore pressure that is responsible for most failures in natural slopes. Until probabilistic methods can provide a more realistic treatment of the pore pressure, they must be regarded with great scepticism. If the pore pressure is entered in a realistic manner, then the probability will have to be expressed in terms of time. The probability of failure will be greater the longer the time span involved.

7. ***The reasons for probabilistic methods.*** In reference 2 the authors argue that probabilistic methods, and associated risk levels, need to be used because of economic pressures. I doubt very much if this is the case. The evidence suggests that clients are becoming far less prepared to accept anything other than top rate performance, and if engineers accept the risk themselves, then should anything go wrong they will be in the firing line. Even in mining engineering, I doubt very much that "a "no failures" approach is generally not able to be carried by the project" (Ref 1). It seems that mining companies take risks to maximise their profits, when in fact they would still be making healthy profits even were they operating without taking high risks. If the risks affect nobody but themselves then of course they are perfectly entitled to take such risks, but in many places around the world, such as Indonesia and Romania, the risks seem to be inflicted on the local population and environment. This is another issue, and I won't pursue it.

REFERENCES

1. Mostyn,G.R. and Fell, R. Quantitative and semi-quantitative estimation of the probability of sliding. Proc. Conf. on Probabilistic Methods in Geotechnical Engineering, Edits Li and Lo, Canberra, Feb. 1993. Balkema.

2. Mostyn, G.R. and Li, K.S. Probabilistic slope analysis - state of play. Proc. Conf. on Probabilistic Methods in Geotechnical Engineering, Edits Li and Lo, Canberra, Feb. 1993. Balkema.

(Note: These were two of the papers included in the New Zealand Geotechnical Society's Symposium on Quantitative Risk Assessment for Slopes held in Auckland in March, 1999).

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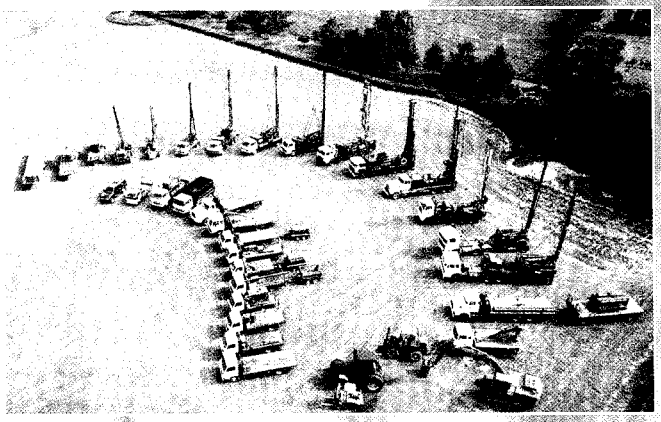

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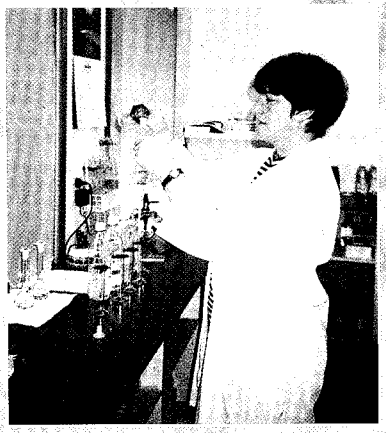

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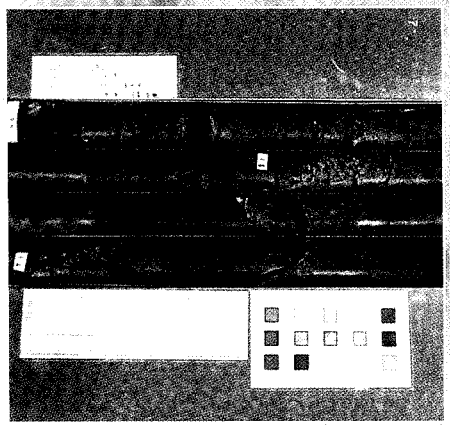

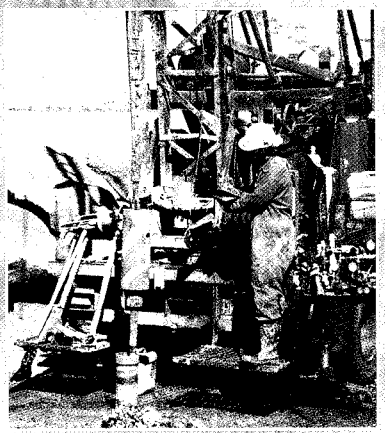

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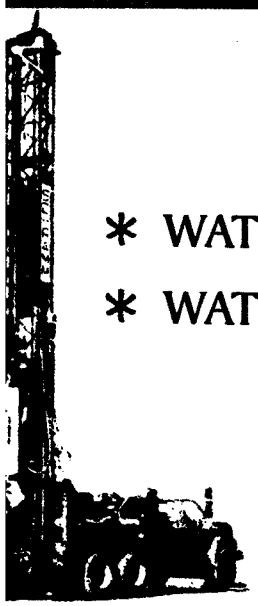

* WATER BORES
* WATER WELLS

Landslide Risk Management Concepts and Guidelines

Australian Geomechanics Society, Sub-Committee on Landslide Risk Management

1 INTRODUCTION

Slope instability occurs in many parts of urban and rural Australia and often impacts on housing, roads, railways and other development. This has been recognised by many local government authorities, and others, and has led to preparation of a number of landslide hazard zoning maps for specific areas, and to the requirement by many local government councils for stability assessments prior to allowing building development. Many such assessments have been based on the paper "Geotechnical Risk Associated with Hillside Development" (Walker *et al*, 1985) which was written by a subcommittee of the Australian Geomechanics Society Sydney Group.

That paper presented a risk classification for slope instability for use in the Sydney Basin (Newcastle-Sydney-Wollongong-Lithgow). It was intended for use by geotechnical consultants, to foster uniformity in the description of risk.

It has become apparent that there are significant deficiencies in the 1985 approach, including:

- The terms are poorly defined
- There was no quantification of risk
- There was no consideration of the potential for loss of life
- The emphasis was on the impact of landsliding occurring on the property to be developed, and did not sufficiently emphasise the importance of landsliding from slopes above a property
- The method was developed for the Sydney Basin and does not necessarily apply to other geological environments. Even within the Sydney Basin there were difficulties in applying the method to areas where very large ancient landslides may be present (e.g. in Wollongong and Newcastle), and to some rock slope situations.

In recognition of this, the National Committee of the Australian Geomechanics Society set up a sub-committee to review what was needed, and establish new guidelines. During this process it became apparent that there is a need for guidance to help practitioners carry out stability assessments for housing allotments, and for use more widely in slope engineering, using risk assessment procedures.

The purpose of this guideline is:

- to establish a uniform terminology;
- define a general framework for landslide risk management;
- provide guidance on methods which should be used to carry out the risk analysis;
- provide information on acceptable and tolerable risks for loss of life.

Such guidelines also have a role in explaining to the public, regulators and the legal profession the process and limitations of Landslide Risk Management.

It is recommended that practitioners and regulators cease using the methods described in Walker *et al* (1985), and follow these guidelines.

2 FRAMEWORK FOR LANDSLIDE RISK MANAGEMENT

2.1 BACKGROUND

Landslide and slope engineering has always involved some form of risk management, although it was seldom formally recognised as such. This informal type of risk management was essentially the exercise of engineering judgement by experienced engineers and geologists. The Walker *et al* (1985) classification system included some risk assessment and treatment concepts.

Procedures for landslide risk assessment have not been standardised in the past, although the use of "risk" or "hazard" zoning maps is widespread internationally. The papers by Varnes (1984), Whitman (1984), Einstein (1988), Morgan *et al* (1992), Fell (1994), Leroi (1996), Wu, Tang & Einstein (1996), Einstein (1997), and Fell & Hartford (1997), give overviews of the subject. Papers by Fell (1992), Moon *et al* (1992) and Moon *et al* (1996) give some examples of landslide risk and hazard assessments in Australia. Flentje & Chowdhury (1999) present an example of quantification of landslide features to enable ranking of the landslides described within a database and to enable assessment of the probability of landslide reactivation.

AS/NZS 4360:1999 "Risk Management" provides a generic framework which has been used as a basis for this guideline. Fell & Hartford (1997) also consider the concepts in some detail, though it should be noted that some of the terminol-

ogy in Fell & Hartford is slightly different to that adopted here.

2.2 RISK MANAGEMENT PROCESS

The Risk Management process comprises three components:

- Risk Analysis
- Risk Evaluation, and
- Risk Treatment.

Figure 1 shows the process in a flow chart form. In simple form, the process involves answering the following questions:

- What might happen?
- How likely is it?
- What damage or injury may result?
- How important is it?
- What can be done about it?

Figure 2 illustrates some of these considerations for a range of simple landslide scenarios.

It is important to recognise that part of the process involves comparing the assessed risks (of property loss and damage, and loss of life) against acceptance criteria. It is recommended that this comparison be carried out with the involvement of the client, owner and regulators.

Sections 3, 4 and 5 discuss the components of the Landslides Risk Management process in more detail.

2.3 RISK MANAGEMENT TERMINOLOGY

There is no single well established terminology in risk management. To further complicate matters, risk management terminology is often misinterpreted and misused, and "risk" means different things to various people and professions.

The ambiguity has been recognised by the International Union of Geological Sciences (IUGS). The Committee on Risk Assessment of their Working Group on Landslides has been developing specific terminology to be used internationally for use in Landslide Risk Management. This terminology, as adopted in this paper, has been designed to be consistent, so far as practicable, with national standards including the Australian New Zealand Standard AS/NZ 4360:1999 for Risk Management.

These definitions are presented in Appendix A. It is recommended that they be used. In addition, usage should be explained in reports, either by providing a copy of the definitions attached to all reports on Landslide Risk Assessment or by defining appropriate key terms in the text.

3 RISK ANALYSIS

3.1 SCOPE DEFINITION

To ensure that the analysis addresses the relevant issues, and to qualify the limits or limitations of the analysis, it is important to define:

- The site, being the primary area of interest
- Geographic limits that may be involved in the processes that affect the site
- Whether the analysis will be limited to addressing only property loss or damage, or will also include injury to persons and loss of life
- The extent and nature of investigations that will be completed
- The type of analysis that will be carried out
- The basis for assessment of acceptable and tolerable risks

It is recommended that these issues should be clearly identified and discussed with the client, preferably before beginning the analysis.

It will be at this stage that a decision should be made as to the degree of quantification that will be undertaken. It is recommended that in all cases it will be important to establish some degree of quantification, even if it is on a crude or preliminary basis. For subsequent ease of communication, it may be appropriate to express the results in a qualitative framework. For assessments involving loss of life, it is recommended that risks be quantified, even if only approximately, to allow comparison with acceptance criteria for the risk of loss of life.

Technical input can also be provided to help other parties (such as owners, accountants and lawyers) to identify:

- The various stakeholders that may be affected (including the owners, occupiers, and regulatory authorities) and their inter-relationships
- The operational and financial constraints
- Legal obligations and responsibilities

3.2 HAZARD IDENTIFICATION

3.2.1 GENERAL PRINCIPLES

Hazard (landslide) identification requires an understanding of the slope processes and the relationship of those processes to geomorphology, geology, hydrogeology, climate and vegetation. From this understanding it will be possible to:

- Classify the types of potential landsliding: the classification system proposed by Varnes (1984) as modified by Cruden & Varnes (1996) forms a suitable system. Its use is recommended and the system has been included in Appendix B for ease of reference. It should be recognised that a site may be affected by more than one type of landslide hazard e.g., deep seated landslides on the site, and rockfall and debris flow from above the site.

- Assess the physical extent of each potential landslide being considered, including the location, areal extent and volume involved.
- Assess the likely initiating event(s), the physical characteristics of the materials involved, and the slide mechanics.
- Estimate the resulting anticipated travel distance and velocity of movement.
- Address the possibility of fast acting processes, such as flows and falls, from which it is more difficult to escape.

Methods which may be used to identify hazards include geomorphological mapping, gathering of historic information on slides in similar topography, geology and climate, (e.g. from maintenance records, air photographs, newspapers, review of analysis of stability etc). Some form of geological and geomorphological mapping is a recommended component of the fieldwork stage when assessing natural landslides, which requires understanding the site whilst inspecting it. Stapledon (1995) and Baynes & Lee (1998) provide further guidance on the role of geology and geomorphology in landslide investigations.

A list of possible hazards should be developed. Consideration must be given to hazards located off site as well as on the immediate site as it is possible for landslides both upslope and downslope to affect a site. It is vital that the full range of hazards (e.g. from small, high frequency events to large, low frequency events) be included in the analysis. Often the risk is dominated by the smaller, more frequent slides. The effects of proposed development should also be considered, as these effects may alter the nature and frequency of possible hazards.

It is important that persons with training and experience in landsliding and slope processes are involved in this stage of the analysis because the omission or under/over estimation of the effects of different hazards will control the outcomes of the analysis.

3.2.2 ESTIMATION OF TRAVEL DISTANCE AND VELOCITY

When assessing risk arising from landsliding, it is important to be able to estimate the distance the slide mass will travel and its velocity. These factors determine the extent to which the landslide will affect property and persons downslope, and the ability of persons to take evasive action.

The travel distance depends on:

- Slope characteristics
  - Height
  - Slope
  - Nature of material
- Mechanism of failure and type of movement, such as
  - Slide, fall, topple etc.
  - Sliding, rolling, bouncing, flow
  - Strain weakening or not
  - Collapse in undrained loading (static liquefaction)
  - Influence of surface water and groundwater
  - Comminution of particles
- Characteristics of the downhill path
  - Gradient
  - Channelisation
  - The potential for depletion/accumulation
  - Vegetation

Information on travel distance from previous events on or near the site may be collected during the site inspection. Predictions of travel distance may be based on the assessed mechanism of future events.

For rotational landslides which remain essentially intact, the method proposed by Khalili *et al* (1996) can be used to estimate the displacement. This is based on the principle of conservation of energy assuming the factor of safety at failure is unity, adopting the residual strength, and the slope geometry to estimate the displacement. The results compare reasonably with case studies. The displacements are greatest for "brittle" failures i.e. where there is a large loss of strength on shearing. The strength loss may be best measured in undrained strength terms, e.g. for soft clays peak and remoulded strengths should be used and for saturated loose (collapsing) granular fills where liquefaction may occur, post liquefaction strengths should be used. For non-circular surfaces, the method may overestimate displacements. Deformation may be modelled for more important projects using finite element, finite difference or distinct element programs.

For slides which break up, and in some cases become flows, the travel distance is usually estimated from the apparent friction angle or "shadow angle" (this being the angle from the horizontal between the top of the slide source, and the toe of the slide debris, ϕ_a, as shown in Figure D1, Appendix D). The most comprehensive data is in Corominas (1996). Other data is presented in Finlay *et al* (1999), Wong & Ho (1996) and Wong *et al* (1997). The data from Finlay *et al* is reproduced in Appendix D.

These methods are only approximate, and the wide scatter of data on apparent friction angles reflects the range of topographical, geological and climatic environments, different slide mechanisms and limited quality of data from which the methods are derived If these methods are to be used for predictions, much judgement will be required and it is important to try to calibrate the methods with landslide behaviour in the study area. It is often useful to allow for a range of travel distances in the calculation and express that range in probabilistic terms. For example:

Travel Distance	**Probability**	**(determined for a particular site)**
<20m	0.2	
20m – 30m	0.6	
30m – 40m	0.2	
	1.0	

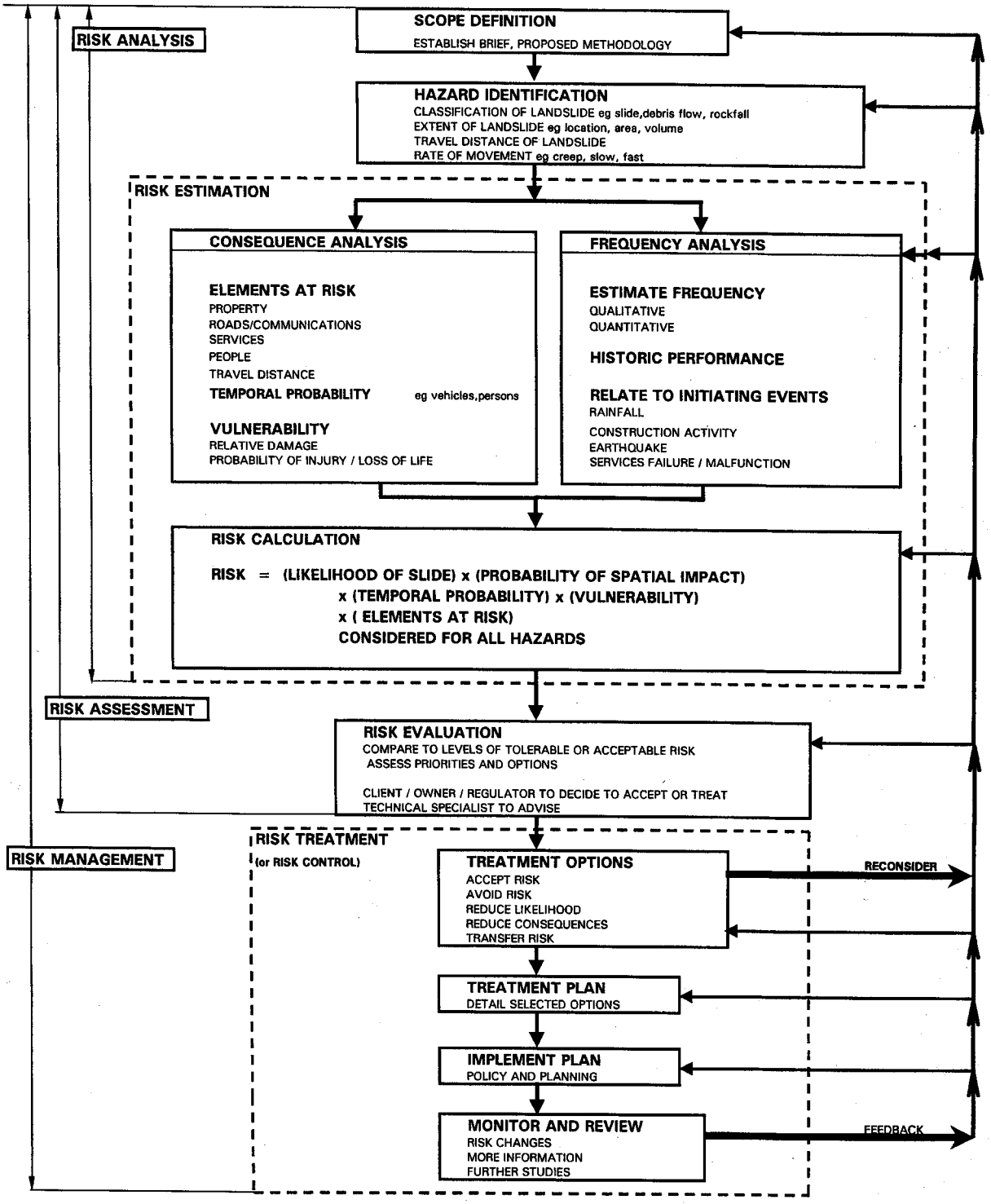

Figure 1. Flowchart for Landslide Risk Management

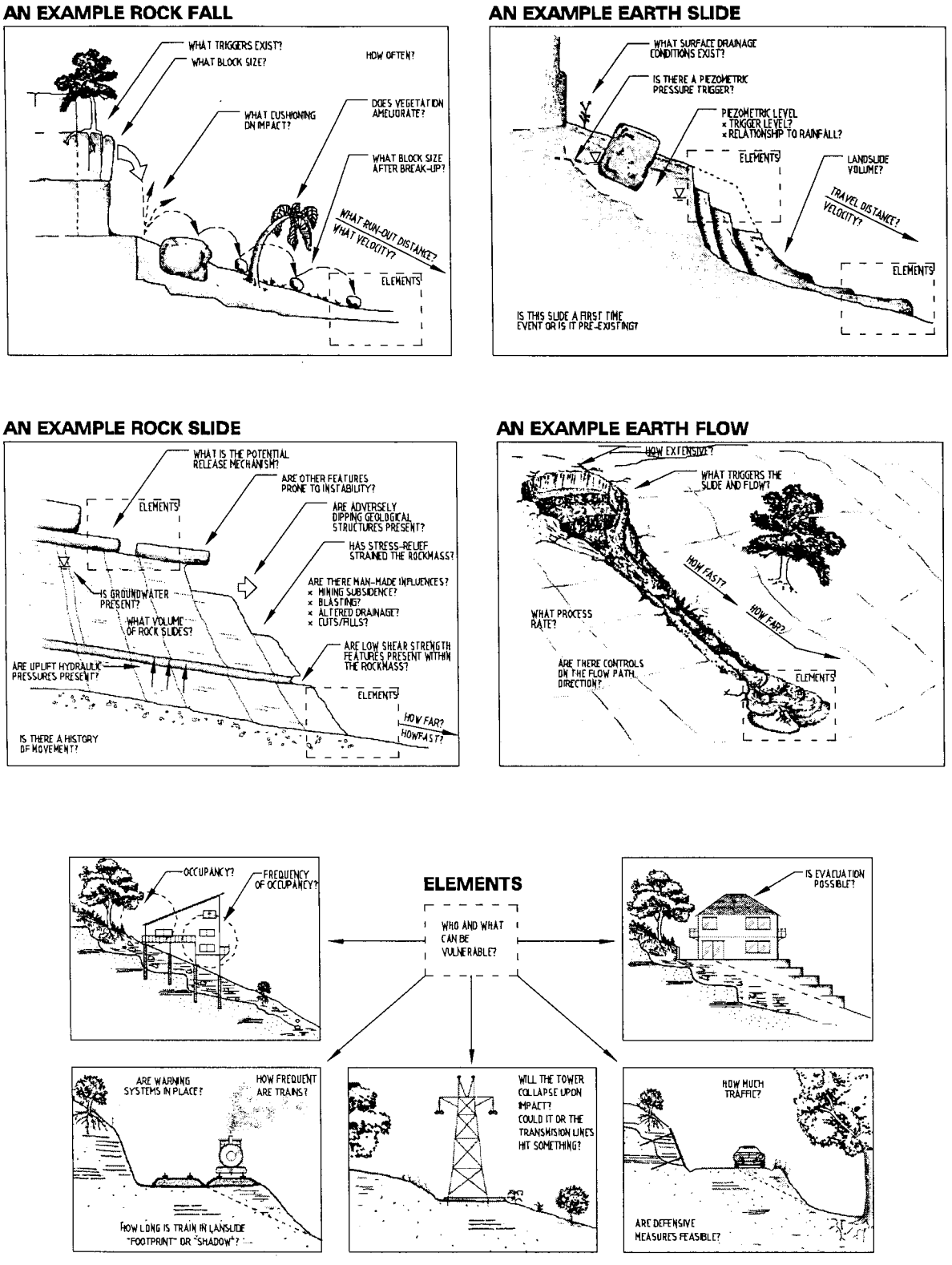

Figure 2. Examples of Landslide Risk Assessment Issues

There are more sophisticated computer programs available to model flows (e.g. Hungr (1996, 1998)), but these are not yet commercially available. For boulder falls, there are commercially available computer programs, such as the Colorado Rockfall Simulation Program (CRSP).

Some of the methods described above also allow estimation of slide velocity, but in most cases it is sufficient to classify likely movement velocity in broad descriptive terms based on the slide classification, such as using the terms given in Appendix B.

3.3 FREQUENCY ANALYSIS

This is usually the most difficult part of the process and will require the majority of effort. It is, however, the key step in Risk Analysis.

The frequency of landsliding can be expressed as:

- The annual frequency of occurrence of landsliding in a nominated part of the landscape (a study area or particular slope facet) based on previous rates of occurrence.
- The probability of an existing landslide moving or a particular slope, cut or fill failing in a given period (e.g. a year), based on an understanding and analysis of the controls on stability.
- The driving forces exceeding the resisting forces in probability or reliability terms, expressing it as an annual frequency.

Different levels of site investigation may be used to assess the frequency such as

- Inspection and observation.
- Mapping (ranging from large-scale regional to small-scale structural), production of sections and interpretation of the geological, hydrogeological, geomorphological and engineering history of the site and environs to form appropriate models.
- The collection of data on history, movement, occurrence, seismicity, rainfall etc using sources such as old newspapers, eyewitness accounts, historical records, previous survey plans, published data, reports etc
- Subsurface investigations such as using pits, drilling, piezometers, monitoring etc to assess geometry, strength, groundwater conditions etc.

Each level of investigation allows increased understanding of the landslide hazards, and therefore of the frequency or probability of occurrence. Stapledon (1995) provides useful lists of investigation questions and emphasises the importance of geological models.

It is considered reasonable to form a judgement as to the hazard and frequency at any level of investigation.

Having made an assessment, if the resulting risks appear unacceptable, then further investigations may help to resolve uncertainties and to formulate an engineering solution. However, in many circumstances a reasonable engineering decision may be reached without more detailed investigations.

There are a variety of methods of estimating frequency from the disparate sets of information that may be assembled. These are detailed in Appendix C (based on Mostyn & Fell (1997) and Baynes & Lee (1998)) and may be summarised as follows:

- Observation and experience – in which the site is viewed, the geology and geomorphology mapped, and a practitioner forms a judgement as to the probability based on experience.
- Inventories – involving the statistics of large number of landslides in time and space and using the relative frequency to predict quantitatively, or ranking to predict qualitatively.
- Triggering – in which the triggering event is identified and the probability of that event equated to the probability of landslide, eg rainfall events.
- Cause and effect – in which a geomorphological understanding is expressed mathematically, eg process rates.
- Deterministic/Probabilistic – in which a deterministic stability model is generated and the inputs are expressed in probabilistic terms.

A combination of methods may be appropriate for any particular landslide hazard. The methods are usually limited by the data available at a particular level of study.

The common types of landslide hazards and the methods which have been found to be useful to assess the likely frequency are summarised in Table 1.

Many landslide assessments are carried out on the basis of initial studies only. Even if extensive investigation is carried out, assessing the probability of landsliding (particularly for an unfailed natural slope) is difficult and involves much uncertainty and judgement. In recognition of this uncertainty, it has been common practice to report the likelihood of landsliding using qualitative terms such as "likely", "possible" or "unlikely".

When qualitative terms are used to describe landslide likelihood, it is recommended that the basis of the assessment is explained and a judgement made of the indicative probability. For example, the basis for assessing the likelihood of landslides within an area to be unlikely may be because the assessor sees no evidence of instability on the site and is unaware of landslides on similar slopes (geology, geomorphology) in the area and elsewhere. The quality of the assessment depends on both the knowledge (the ability to recognise what is a similar slope) and experience (seen or knows about the performance over time of many similar slopes) of the assessor. In these circumstances an experienced

Hazard Scenario	Applicable Methods (1)
	Natural Slopes
First time slides and shallow existing slides not identified specifically	Hazard zones based on geomorphological mapping and interpretation should identify areas or slope facets more prone to failure. Frequency may be derived from inventories of the historic occurrence of landslides in a part of the landscape. Associations of occurrences with major triggers such as rainfall and seismic events may also form part of the analysis.
Rockfalls, boulder falls and debris flows	Hazard zones based on geomorphological mapping and interpretation should identify areas prone to failure and knowledge of apparent friction angles should indicate travel distances. Frequency may be derived from knowledge of the process rate within hazard zones allowing assessment of recurrence intervals. Associations of occurrences with major triggers such as rainfall and seismic events may also form part of the analysis. Note that process rate may change with time.
Deep slides in rock or soil	Geomorphological mapping of slide and environs should establish extent, geometry, controls and potential area of influence. Movement is likely to reflect piezometric response to rainfall or other source of water. Appropriate soil/rock mechanics principles will assist formulation of a geotechnical model. Frequency may be derived from regional studies of similar occurrences, geological history of site and timing of major movements, records of movement measurements, and/or recurrence interval of triggers such as rainfall patterns or seismic events in conjunction with stability analyses.
	Constructed Slopes
Cuts and fills	Geological and geotechnical mapping and inspection should establish typical performance of similar cuts or fills and information on existing failures. Data collection on the controls on stability (especially defects in rock masses) may provide statistical information for analysis. Engineering assessment of construction quality, performance history, drainage adequacy etc is useful. Frequency may be derived from statistics of similar cuts or fills, recurrence intervals of triggers such as rainfall patterns, or seismic events. Deterministic/probabilistic analyses based on geological and geotechnical data and soil/rock mechanics may be useful for very important cuts or fills in conjunction with other methods.

Note (1): Choice of applicable method may depend on whether a preliminary study or more detailed study is being carried out.

Table 1 Methods for estimating the frequency of landsliding

assessor may be able to judge that the annual probability of a landslide at a site is likely to be less than 10^{-3} on the basis that:

- the assessor has knowledge of at least 100 similar slopes over an average period of 10 years;
- the slopes are likely to have been subject to some extreme rainfall events.

The example illustrates that individual stability assessments cannot be made in isolation and the role of knowledge, experience and judgement in the assessment. A different assessor, with different knowledge and experience, may arrive at a different judgement. Additionally, care is needed when assessing the long term behaviour of cut slopes in clays if only a short term history is available due to the possibility of delayed failures.

Where links between qualitative terms and indicative probabilities are made, the link should be explained and defined. An example of such a link is given in Appendix G.

Purely qualitative assessments of relative likelihood (without even an indicative link to probability) may be used to rank likelihoods of landslide hazards in a particular area. However, they do not allow the risks to be quantified and do not allow comparison of landslide risk with risks associated with other hazards (e.g. floods, fires, car accidents etc). As discussed in Section 3.5.2 at least indicative quantification of likelihood is recommended where there is concern about loss of life.

Where there is knowledge of previous slope failures it may be possible to assess frequency directly. For example: if the failure of an old roadfill behind a house is thought possible and there is knowledge of one or two road fills which have failed on average each year out of one or two thousand in similar geological, topographic, and climatic environments, an indicative annual probability of failure of 10^{-3} may be applied. Alternatively, collation of the failure history may enable a simplistic calculation, such as: If 10 fill slopes out of an estimated 250 slopes are known to have failed over a 20 year period, the indicative annual probability would be 1 in 500 ($2x10^{-3}$), assuming all slopes are similar.

If on the other hand the roadfill was new, known to be well designed and constructed, the failure might be considered less likely than might be suggested by the knowledge of fill performance and an indicative annual probability of failure of 10^{-5} might be applied on the basis that it is judged to be two orders of magnitude less likely.

Such estimates of probability may be sufficient to enable identification of potentially high risk situations, which once identified can be studied in more detail.

completed quantitatively or by the use of qualitative terms.

A semi quantitative analysis (where the likelihood is linked to an indicative probability) or a qualitative analysis may be used.

- As an initial screening process to identify hazards and risks which require more detailed consideration and analysis.
- When the level of risk does not justify the time and effort required for more detailed analysis.
- Where the possibility of obtaining numerical data is limited such that a quantitative analysis is unlikely to be meaningful or may be misleading.

The terms to be used should be defined for a specific project. Appendix G gives an example of qualitative terminology which should be used unless a site specific terminology is needed. These terms are not consistent with those in the Walker *et al* (1985) paper. Whilst other terms may be used if required, there will be advantages in adoption of Appendix G by most practitioners. For some assessments it may be useful to develop a simpler system with less terms for likelihood, consequence and risk. Whatever terminology is to be used, terms must be defined and this may be done by attaching as an appendix of definitions to the report. In some cases dual descriptors for likelihood, consequence and risk can be useful to reflect the uncertainty in the estimates.

3.5.3 SEMI QUANTITATIVE RISK ESTIMATION FOR LOSS OF LIFE

Risk for loss of life should be quantified because the risk acceptance criteria used in society for loss of life are quantified. To assist in this regard, some indicative annual probabilities are given for the likelihood terms in Appendix G, so that some consistency between loss of life and property risk calculation can be retained. The probabilities are only approximate, and one order of magnitude either way from the indicative values would be possible.

In some situations where risk of loss of life is identified as an issue in semi quantitative analysis, it may be possible to take immediate risk reduction measures without further assessment. If this is not possible it is recommended that quantitative analysis be carried out. Quantification will enable the risk to be evaluated against risk acceptance criteria (Section 4.2.2). Loss of life as a result of landslides often involves combinations of events. Quantifying the risk may involve multiplying together many quantified judgements. It is good practice to explain the basis of the judgements and the uncertainty involved.

The important first stage for the landslide risk assessor is to identify whether loss of life is an issue. If the assessor has little experience of the hazard or of quantitative risk analysis, it may be useful to involve another person with more experience of these areas.

3.6 SENSITIVITY ANALYSIS AND UNCERTAINTY

As estimates made for an analysis will be imprecise, sensitivity analyses are useful to evaluate the effect of changing assumptions or estimates. Wherever possible, such assumptions and the resulting sensitivity should be stated or expressed in the report. Variation in the estimate of risk by one or two orders of magnitude, or perhaps three orders of magnitude at low risks, will not be uncommon. The resulting sensitivity may aid judgement as to the critical aspects requiring further investigation or evaluation.

If a sensitivity analysis is not carried out, it is good practice to explain some of the limitations and uncertainty in the risk estimates. In detailed studies, the uncertainty can be formally modelled.

4 RISK ASSESSMENT / RISK EVALUATION

Risk Evaluation is the final step in the Risk Assessment process (Figure 1).

4.1 OBJECTIVE AND PROCESS OF RISK EVALUATION

Risk analysis alone has limited benefits and it is normal to carry the process to the next stages of risk evaluation and risk treatment.

The main objectives of risk evaluation are usually to decide whether to accept or treat the risks and to set priorities. The decision is usually the responsibility of the owner/client/ regulator. Involvement of those indirectly affected is desirable. Non- technical clients may seek guidance from the risk assessor on whether to accept the risk. In these situations, risk comparisons, discussion of treatment options and explanation of the risk management process can help the client make their decision.

Risk evaluation involves making judgements about the significance and acceptability of the estimated risk. Evaluation may involve comparison of the assessed risks with other risks or with risk acceptance criteria related to financial, loss of life or other values. Risk evaluation may include consideration of issues such as environmental effects, public reaction, politics, business or public confidence and fear of litigation. In a simple situation where the client/owner is the only affected party, risk evaluation may be a simple value judgement. In more complex situations, value judgements on acceptable risk appropriate to the particular situation are still made as part of an acceptable process of risk management.

Risk acceptance for a quantitative analysis is likely to be based, at least partly, on quantitative values with consideration of the uncertainty and defensibility of the assessment. For a qualitative or semi quantitative assessment the acceptance criteria may be qualitative. Explaining the acceptance criteria adopted facilitates review and may make the judgement more defensible. With the wide variety of issues which need to be considered, and the varying attitudes to risk, it may not be possible to pre-define acceptance criteria.

Assessment of the risk may involve consideration of values such as:

(a) For property or financial losses:
- Cost benefit ratio
- Financial capability
- Annualised cost
- Corporate impact
- Frequency of accidents

(b) For loss of life
- Individual risk
- Societal risk, e.g. as frequency versus number of deaths (known as f-N) or cumulative frequency versus number of deaths (known as F-N) criteria. (Refer to Fell & Hartford (1997) for further explanation and examples).
- Annualised potential loss of life
- Cost to save a life.

It is desirable, if not essential, that the risk analyst be involved in the decision making process because the process is often iterative, requiring assessment of the sensitivity of calculations to assumptions, modification of the development proposed and revision of risk mitigation measures.

4.2 ACCEPTABLE AND TOLERABLE RISKS

It is important to distinguish between acceptable risks which society desires to achieve, particularly for new projects, and tolerable risks which they will live with, even though they would prefer lower risks. This applies to both property and loss of life.

4.2.1 PROPERTY

Factors that affect an individual's attitude to acceptable or tolerable risk will include:

- Resources available to treat the risk.
- Whether there is a real choice, e.g. can the person afford to vacate a house despite the high risk.
- The individual's commitment to property and relative value.
- Age and character of the individual.
- What exposure the individual has had to risk in the past, especially risk associated with landslides.
- Availability of insurance.
- Regulatory or policy requirements.
- Whether the risk analysis is believed.

Acceptable and tolerable risks for property loss and damage must be determined by the client, owner and if appropriate, regulator.

Appendix G gives an example of qualitative risk terms which could be used for risk to property. Other terms may be defined and used. The "example and implications" shown in Appendix G are a hypothetical example for a particular situation. It is for the owner/client and regulating authority (e.g. local government council) to assess what is acceptable. The amount of investigation required, and cost of treatment, is not necessarily related to the level of risk. For example, if the high risk is associated with a single large boulder on a steep slope, it may be relatively easy to remove the boulder and reduce the risk.

4.2.2 LOSS OF LIFE

There are no established individual or societal risk acceptance criteria for loss of life due to landslides in Australia or internationally. It is possible to provide some general principles and some information from other engineering industries, e.g. petrochemical and dams. These can be used to obtain a general appreciation of the risks and to suggest some acceptance criteria for landslides. Nonetheless, the decision on risk acceptability (or tolerance) must be made by the client, owner, regulator and those at risk, where they are an identified group.

There are some common general principles that can be applied when considering tolerable risk criteria. These are taken from IUGS (1997):

(a) The incremental risk from a hazard should not be significant compared to other risks to which a person is exposed in everyday life.

(b) The incremental risk from a hazard should, wherever reasonably practicable, be reduced: i.e. the As Low As Reasonably Achievable (ALARA) principle should apply.

(c) If the possible loss of large numbers of lives from a landslide incident is high, the probability that the incident might actually occur should be low. This accounts for society's particular intolerance to incidents that cause many simultaneous casualties and is embodied in societal tolerable risk criteria.

(d) Persons in society will often tolerate higher risks than they regard as acceptable when they are unable to control or reduce the risk because of financial or other limitations.

(e) Higher risks are likely to be tolerated for existing slopes than for planned projects, and for workers in industries with hazardous slopes, e.g. mines, than for society as a whole.

These principles are common with other hazards such as Potentially Hazardous Industries (PHI) and dams. There are other principles that are applicable only to risks from slopes and landslides:

(f) Tolerable risks are thought to be higher for naturally occurring landslides than those from engineered slopes, but this has not been proven.

(g) Once a natural slope has been placed under monitoring, or risk mitigation measures have been executed, the tolerable risks may approach those of engineered slopes.

(h) Tolerable risks may vary from country to country and within countries, depending on historic exposure to landslide hazard, the system of ownership and control of slopes and natural landslide hazards, and the risks a person is exposed to in everyday life.

There is reasonable consistency between the PHI and various dam authorities in acceptable individual risk criteria. These are summarised in Appendix H, which is taken from Fell & Hartford (1997). Based on this, it might reasonably be concluded that the following criteria apply to constructed slopes.

Situation	Suggested Tolerable Risk for Loss of Life
Existing Slopes	10^{-4} person most at risk 10^{-5} average of persons at risk
New Slopes	10^{-5} person most at risk 10^{-6} average of persons at risk

Acceptable risks are usually considered to be one order of magnitude smaller than the above Tolerable Risks.

The situation for societal risk is more contentious. Some organisations (e.g. Great Britain Health and Safety Executive, and NSW Department of Planning) only use qualitative terms for societal risk.

The Australian National Committee on Large Dams have criteria which were published in ANCOLD (1994). These are under review. The most recent published draft of the review is shown in Appendix H. This is subject to further review. In the absence of other information this might be used as an indication of the societal risks.

Fell & Hartford (1997) gives some details on the use of societal risk plots when considering individual and societal risk criteria. It should be remembered that (taken from IUGS 1997):

(i) Estimates of risk are inevitably approximate and the acceptance criteria should not be considered as absolute values. The assessed risk may span the acceptance criteria. Judgement is needed as to whether that may be acceptable in the light of the defensibility of the assessment. Variations by up to, say, one order of magnitude may be appropriate for the acceptance criteria for particular circumstances.

(ii) Tolerable risk criteria, such as those published for PHI and dams, are themselves not absolute boundaries. Society shows a wide range of tolerance to risk and the risk criteria are only a mathematical expression of general societal opinion.
There may be cases where risks higher than the upper limit tolerable risk criteria are adopted, because the ALARA principle, or Best Practical Technology (BPT), indicates it is not practicable to further reduce the risk.

(iii) It is often useful to consider several different tolerable risk criteria (e.g. individual and societal risk, cost to save a life, etc).

(iv) It must be recognised that risk estimation is only one input to the decision process. Owners, society and regulators will also consider political, social and legal issues in their assessments and may consult the public affected by the hazard.

(v) The risk can change with time because of natural processes and development. For example:
- Removal of debris from slopes can lead to reduction in risk
- Removal of vegetation by natural processes (e.g. fire or human intervention) can lead to an increase in risk
- Construction of roads on, below or above a slope may increase the probability of landsliding and/or the elements at risk, and hence the risk.

(vi) Extreme events should be considered as part of the spectrum of events. Inclusion of extreme events is important in assessing the triggers (landslides, earthquake), the size of the landslide and the consequences. However, often it is the smaller, more frequent, landslides that contribute most to risk, not the extreme event.

4.3 SUMMING THE RISK FROM SEVERAL HAZARDS

Care needs to be taken when assessing the risk from individual slopes, to take into account whether the risk needs to be considered along with the risk from other slopes to which the public is exposed. For example, it is usually more relevant to sum the risk from all landslides for persons travelling on a highway between their home and destination, than to only consider the risk from one slope.

Appendix I provides some insight to this issue.

4.4 LIMITATIONS, BENEFITS AND DEFENSIBILITY OF RISK ASSESSMENT

There are a number of limitations to risk assessment for slopes and landslides:

- The judgement content of the inputs to any analysis may result in values of estimated risks with considerable inherent uncertainty.
- The variety of approaches that can reasonably be adopted to analyse landslide risk can result in significant difference in outcome for the same situation when considered separately by different practitioners.
- To complete a risk assessment, time and skills are required to make and interpret the field observations and develop the insight and understanding of the slope process applicable. Greater experience and understanding of the processes will improve the reliability of the analysis.
- Revisiting an analysis can lead to significant change due to increased data, a different method or changing circumstances.
- The consequences of an inability to recognise a significant hazard will be underestimation of the risk.
- The results of an assessment are seldom verifiable, though peer review can be useful.
- The methodology is currently not widely accepted and thus there sometimes is an aversion to its application.

- It is possible that the cost of the analysis may outweigh the benefit of the technique in making a decision, especially where complex detailed sets of data are required. However, this is really an issue of matching the analysis method to the scale of problem and the resources available.
- There may be difficulty in completing a quantitative analysis due to the difficulty of obtaining sufficient data for reliable evaluation of the frequency of events.
- It is difficult to accurately analyse risk for low probability events.

Most of the above limitations are inherent in any approach to assessing landslides. Risk analysis has the benefit of encouraging a systematic approach to a problem and promoting a greater understanding of consequences. In many situations, an indicative estimate of the probability of a hazard (such as using those given in Appendix G) and an assessment of the consequences can be readily conducted.

As noted above, some of the inputs to the analysis may be largely judgmental. Even so, it is important that the judgement be "defensible" by reporting the basis or logic on which the judgement is based. Thus the "defensibility" of the assessment becomes a measure of the quality of the information available/used and the methods used. Methods for developing defensible subjective probability assessments are discussed by Roberds (1990).

5 RISK MANAGEMENT / RISK TREATMENT

Risk Treatment is the final stage of the Risk Management process and provides the methodology of controlling the risk.

5.1 RISK TREATMENT

At the end of the evaluation procedure, it is up to the client or policy makers to decide whether to accept the risk or not, or to decide that more detailed study is required. The landslide risk analyst can provide background data or normally acceptable limits as guidance to the decision maker, but as discussed above, should not be making the decision. Part of the specialist advice may be to identify the options and methods for treating the risk.

5.1.1 TREATMENT OPTIONS

Typical options would include:

- *Accept the risk;* this would usually require the risk to be considered to be within the aeceptable or tolerable range.
- ***Avoid the risk;*** this would require abandonment of the project, seeking an alternative site or form of development such that the revised risk would be acceptable or tolerable.
- ***Reduce the likelihood;*** this would require stabilisation measures to control the initiating circumstances, such as reprofiling the surface geometry, groundwater drainage, anchors, stabilising structures or protective structures etc. After implementation, the risk should be acceptable or tolerable, consistent with the ALARA principle.
- ***Reduce the consequences;*** this would require provision of defensive stabilisation measures, amelioration of the behaviour of the hazard or relocation of the development to a more favourable location to achieve an acceptable or tolerable risk.
- ***Monitoring and warning systems***; in some situations monitoring (such as by regular site visits, or by survey), and the establishment of warning systems may be used to manage the risk on an interim or permanent basis. Monitoring and warning systems may be regarded as another means of reducing the consequences.
- ***Transfer the risk;*** by requiring another authority to accept the risk or to compensate for the risk such as by insurance.
- ***Postpone the decision;*** if there is sufficient uncertainty, it may not be appropriate to make a decision on the data available. Further investigation or monitoring would be required to provide data for better evaluation of the risk and treatment options. It should be made clear that this situation is temporary while the further work being carried out. During this period, the situation is being temporarily accepted even though the risks may not be acceptable or tolerable.

The relative costs and benefits of the options need to be considered so that the most cost effective solutions, consistent with the overall needs of the client, owner and regulator, can be identified. Combinations of options or alternatives may be appropriate, particularly where relatively large reductions in risk can be achieved for relatively small expenditure. Prioritisation of the options is likely to assist with selection.

Guidance on good engineering practice for hillside design and construction is given in Appendix J which has been adapted from Walker *et al* (1985).

5.1.2 TREATMENT PLAN

A treatment plan for each option may be used to explain how the option will be implemented.

Where possible, each plan needs to identify responsibilities for each party during and after implementation, the extent of work required, cost estimates and programme, performance measures and the expected outcome. The level of detail will depend on the priority for the option and stage of the evaluation process. There may be interaction between a number of parties to resolve all of these issues, such as the planner, the owner and the regulator.

A treatment plan may include an emergency plan, which should establish from the outset the sequence of events that will be initiated if warning signs indicate a potential instability. It should establish what the different warning levels will be and, depending on which level is achieved,

- establish the hierarchy for dealing with the emergency and the lines of communication that will be used,
- send out the appropriate warnings to those who may be affected,

- ensure the warnings are understood in the context of the risk and
- ensure that personnel, materials and equipment will be available within an acceptable time for dealing with the instability.

An effective treatment plan aids implementation and should be developed on an explicit basis where possible. However, for some cases a treatment plan may not be necessary.

5.2 MONITOR AND REVIEW

Monitoring of the treatment plan and risks is needed to ensure the plan is effective and that changes in circumstances do not alter risks. Factors which affect the likelihood and consequences may change with time. Thus, ongoing review of the treatment is essential for the management process.

Construction of stabilisation measures may yield further data or show that assumed subsurface models were not appropriate. Hence, during construction it is reasonable for the design to be reviewed and the risks to be reassessed.

It is essential to reconsider all stages of the analysis, assessment and prioritisation as the treatment plan evolves and is implemented. The results of monitoring may enable feedback for reassessment of the risks.

6 HAZARD ZONING

Risk assessment principles can be applied to producing maps showing hazard zones. This involves:

- Generation of maps summarising observations on geology, geomorphology, and in particular the distribution of landslide processes including use of local records, interpretation of photographs and field observations. Engineered slopes should also be identified. This is known as the process map.
- Collection of information on the landslide hazards identified from the above.
- Analysis of potential hazards including first time slides, deep seated existing slides, rock debris flows, cuts and fills.
- Identification of areas that may be impacted by such hazards.
- Transformation of the process map to a hazard map identifying the potential for spatial impact and probability of occurrence for all the hazards.

The maps should be accompanied by a description of each class of landslide hazard.

To convert this to risk, the person using the zoning maps would need to define the elements at risk, identify which hazards affect the elements, estimate temporal probability and the vulnerability, and then calculate the risk.

Where the elements at risk are not well defined, it is usually impractical to prepare a risk zoning map. Where an area is already developed, risk zoning may be practical for risk to property. An example of a study involving quantitative hazard and risk zoning in Australia is provided by Leiba, Baynes & Scott (in press).

7 CONCLUSION AND RECOMMENDATION

It is considered that risk assessment methods for landslides and slopes have been developed to a level that they are applicable in practical terms and form a useful tool to complement engineering judgement. The level of analysis possible will vary from project to project and may increase as further data becomes available.

It is recommended that these guidelines be adopted and that the use of the methods outlined in Walker *et al* (1985) be discontinued.

Risk assessment reports should define the terminology and approach being adopted. In some cases this may be achieved in the text of the report. In other cases it may be useful to include one or more of the following:

- Appendix A (entire or extract);
- List or table explaining terms used for likelihood, consequences and risks. Appendix G is an example;
- Appendix J;
- Figure 1.

Use of material from this paper will simplify presentation and establish some uniformity of practice; the above pages have been annotated with a footnote to facilitate their direct reuse.

The development of risk assessment methods is continuing, and practitioners should refer to published literature for improvements in the methods.

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REFERENCES

ANCOLD (1998). ANCOLD Guidelines on Risk Assessment. Position Paper on revised criteria for acceptable risk to life. ANCOLD Working Group on Risk Assessment.

AS/NZS 4360:1999 (1999). Australian/New Zealand Standard, Risk Management Standards Australia, Standards New Zealand.

Baynes, F.J. and Lee, M. (1998). Geomorphology in landslide risk analysis, an interim report. Proceedings of the Eighth Congress of the Int. Assoc. of Engineering Geologists, ed. Moore and Hungr, Balkema, pp.1129-1136.

Bunce, C.M., Cruden, D.M. and Morgenstern, N.R. (1997). Assessment of the hazard from rockfall on a highway. Canadian Geotechnical Journal Vol. 34, No.3, pp.344-356.

Corominas, J. (1996). The angle of reach as a mobility index for small and large landslides. Canadian Geotechnical Jnl. V33, pp.260-271.

Cruden, D.M. (1997). Estimating the risks from landslides using historical data; in "Landslide Risk Assessment", Cruden and Fell (eds.), Balkema, pp. 277-284.

Cruden, D.M. and Varnes, D.J. (1996). Landslide types and processes: in "Landslides Investigation and Mitigation". Eds K. Turner, R.L. Schuster. Transportation Research Board Special Report 247, National Research Council, Washington, DC.

Einstein, H.H. (1988). Special Lecture, Landslide risk assessment. Proc. 5th Int. Symp. On Landslides, Lausanne, Switzerland. A.A. Balkema, Rotterdam, The Netherlands, Vol.2, pp.1075-1090

Einstein, H.H. (1997). Landslide Risk – Systematic approaches to assessment and management in Landslide Risk Assessment, Cruden and Fell (eds.), Balkema, Rotterdam, pp. 25-50.

Fell, R. (1992). Some landslide risk zoning schemes in use in Easter Australia and their application. In Geotechnical Risk – Identification, Evaluation and Solutions. Proc. Sixth Australia-New Zealand Conf. on Geomechanics, Christchurch. The New Zealand Geomechanics Society, pp. 505-512.

Fell, R. (1994). Landslide risk assessment and acceptable risk, Canadian Geotechnical Journal, 31, pp.261-272.

Fell, R. and Hartford, D. (1997). Landslide Risk Management; in "Landslide Risk Assessment", Cruden and Fell (eds.), Balkema, Rotterdam, pp.51-110.

Finlay, P.J., Fell, R. and Maguire, P.K. (1997). The relationship between the probability of landslide occurrence and rainfall. Canadian Geotechnical Journal, Vol.34, No.6, pp.811-824.

Finlay, P.J., Mostyn, G.R. and Fell, R. (1999). Landslides: Prediction of Travel Distance and Guidelines for Vulnerability of persons. Proc. 8th Australia New Zealand Conference on Geomechanics, Hobart, Australian Geomechanics Society, ISBN 1 86445 0029, Vol.1, pp.105-113.

Flentje, P.N. and Chowdhury, R.N. (1999) Quantitative Landslide Hazard Assessment in an Urban Area. Proc. Eighth Australia New Zealand Conf. on Geomechanics. Ed Dr Nihal Vitharana Feb. 15-17, Hobart, Tasmania. Institution of Engineers, Australia.

GEO (1995). New Priority Classification System for Soil Cut Slopes, Geotechnical Engineering Office Special Project Report SPR 6/95, Civil Engineering Department, Hong Kong.

Hungr, O., and Evans, S.G., (1996). Rock Avalanche runout prediction using a dynamic model, Proc. 7th Int. Symp. On Landslides, Trondheim Vol.1, pp.233-238.

Hungr, O., (1998). Mobility of landslide debris in Hong Kong: Pilot back analyses using a numerical model. Report prepared for the Geotechnical Engineering Office, Hong Kong.

IUGS (1997). Quantitative Risk Assessment for Slopes and Landslides – the State of the Art. IUGS Working Group on Landslides, Committee on Risk Assessment, in Landslide Risk Assessment, Cruden and Fell (eds.), Balkema, Rotterdam, pp.3-12.

Khalili, N., Fell, R., Tai, K.S., (1996). A simplified method for estimating failure induced deformation, Proc. 7th Int. Symp. On Landslides, Trondheim, Vol.2, pp.1263-1268.

Leiba, M., Baynes, F.J. and Scott, G. (in press) "Quantitative Landslide Risk Assessment of Cairns, Australia". Proc. Eighth International Symposium on Landslides, Cardiff.

Leroi, E. (1996). Landslide hazard – Risk maps at different scales: objectives, tools and developments. In Landslides, Proc. Int. Symp. On Landslides, Trondheim, 17-21 June (Ed. K. Senneset), pp.35-52.

Moon, A., Robertson, M. and Davies, W. (1996). Quantifying rockfall risk using a probabilistic toppling failure model. Proc. 7th Int. Symp. On Landslides, Trondheim, 17-21 June 1996. K. Senneset Edition. Balkema Rotterdam.

Moon, A.T., Olds, R.J., Wilson, R.A. and Burman, B.C. (1992). Debris flow zoning at Montrose, Victoria. In Landslides, Proc. Sixth Int. Symp. On Landslides, February, (Ed. D.H. Bell), Christchurch, New Zealand. A.A. Balkema, Rotterdam, The Netherlands, Vol.2, pp.1015-1022.

Morgan, G.C., Rawlings, G.E. and Sobkowicz, J.C. (1992). Evaluating total risk to communities from large debris flows. In Geotechnique and Natural Hazards. Proc. Geohazards '92 Symposium, BiTech Publishers, Canada, pp.225-236.

Morgan, G.C. (1997). A regulatory perspective on slope hazards and associated risks to life, in Landslide Risk Assessment, Cruden and Fell (eds.), Balkema, pp.285-295.

Morgan, M.G., Fischhoff, B., Bostrom, A., Lave, L. and Atman, C.J. (1992). Communicating risk to the public. Environmental Science and Technology, Vol. 26, No. 11, pp.2048-2056.

Mostyn, G. and Fell, R. (1997). Quantitative and Semi quantitative estimation of the probability of landslides, in Landslide Risk Assessment, Cruden and Fell (eds.), Balkema, Rotterdam, pp.297-316.

Roberds, W.J. (1990). Methods for developing defensible subjective probability assessments. Proceedings of

the Transportation Research Board, No. 1288, National Research Council, Washington DC, pp.183-190.

Roberds, W.J., Ho, K. and Leung, K.W. (1997). An integrated methodology for development and risk management for development below potential natural terrain landslides, in Landslide Risk Assessment, eds. D.M. Cruden and R. Fell, Balkema, pp.333-346.

Stapledon, D.H. (1995). Geological modelling in landslide investigation. Keynote paper. Proc. 6th Int. Symp. on Landslides, Christchurch, Balkema Vol.3, pp.1499-1523.

Varnes, D.J. and The International Association of Engineering Geology Commission on Landslides and Other Mass Movements (1984). Landslide hazard zonation: A review of principles and practice. Natural Hazards, Vol.3, Paris, France. UNESCO, 63p.

Walker, B.F., Dale, M., Fell, R. Jeffery, R., Leventhal, A., McMahon, M. Mostyn, G. and Phillips, A. (1985). Geotechnical Risk Association with hillside development. Australian Geomechanics News, No.10, pp.29-35.

Whitman, R.V. (1984). Evaluating calculated risk in geotechnical engineering. ASCE Journal of Geotechnical Engineering, 110(2), pp.145-188.

Wong, H.N. and Ho, K. (1996). Travel distances of landslide debris. Proc. 7th Int. Symp. on Landslides, Trondheim, Vol.1, pp.417-422.

Wong, H.N., Ho, K. and Chan, Y.C. (1997). Assessment of consequences of landslides, in Landslide Risk Assessment, Cruden and Fell (eds.), Balkema, Rotterdam, pp.111-149

Wu, T.H., Tang, W.H. and Einstein, H.H. (1996). Landslide Hazard and Risk Assessment in Landslides Investigation and Mitigation. Eds. K. Turner, R.L. Schuster. Transportation Research Board Special Report 247, Academy Press.

APPENDIX A

DEFINITION OF TERMS

INTERNATIONAL UNION OF GEOLOGICAL SCIENCES WORKING GROUP ON LANDSLIDES, COMMITTEE ON RISK ASSESSMENT

Risk – A measure of the probability and severity of an adverse effect to health, property or the environment.
Risk is often estimated by the product of probability x consequences. However, a more general interpretation of risk involves a comparison of the probability and consequences in a non-product form.

Hazard – A condition with the potential for causing an undesirable consequence (*the landslide*). The description of landslide hazard should include the location, volume (or area), classification and velocity of the potential landslides and any resultant detached material, and the likelihood of their occurrence within a given period of time.

Elements at Risk – Meaning the population, buildings and engineering works, economic activities, public services utilities, infrastructure and environmental features in the area potentially affected by landslides.

Probability – The likelihood of a specific outcome, measured by the ratio of specific outcomes to the total number of possible outcomes. Probability is expressed as a number between 0 and 1, with 0 indicating an impossible outcome, and 1 indicating that an outcome is certain.

Frequency – A measure of likelihood expressed as the number of occurrences of an event in a given time. See also Likelihood and Probability.

Likelihood – used as a qualitative description of probability or frequency.

Temporal Probability – The probability that the element at risk is in the area affected by the landsliding, at the time of the landslide.

Vulnerability – The degree of loss to a given element or set of elements within the area affected by the landslide hazard. It is expressed on a scale of 0 (no loss) to 1 (total loss). For property, the loss will be the value of the damage relative to the value of the property; for persons, it will be the probability that a particular life (the element at risk) will be lost, given the person(s) is affected by the landslide.

Consequence – The outcomes or potential outcomes arising from the occurrence of a landslide expressed qualitatively or quantitatively, in terms of loss, disadvantage or gain, damage, injury or loss of life.

Risk Analysis – The use of available information to estimate the risk to individuals or populations, property, or the environment, from hazards. Risk analyses generally contain the following steps: scope definition, hazard identification, and risk estimation.

Risk Estimation – The process used to produce a measure of the level of health, property, or environmental risks being analysed. Risk estimation contains the following steps: frequency analysis, consequence analysis, and their integration.

Risk Evaluation – The stage at which values and judgements enter the decision process, explicitly or implicitly, by including consideration of the importance of the estimated risks and the associated social, environmental, and economic consequences, in order to identify a range of alternatives for managing the risks.

Risk Assessment – The process of risk analysis and risk evaluation.

Risk Control or Risk Treatment – The process of decision making for managing risk, and the implementation, or

enforcement of risk mitigation measures and the re-evaluation of its effectiveness from time to time, using the results of risk assessment as one input.

Risk Management – The complete process of risk assessment and risk control (*or risk treatment*).

Individual Risk – The risk of fatality or injury to any identifiable (named) individual who lives within the zone impacted by the landslide; or who follows a particular pattern of life that might subject him or her to the consequences of the landslide.

Societal Risk – The risk of multiple fatalities or injuries in society as a whole: one where society would have to carry the burden of a landslide causing a number of deaths, injuries, financial, environmental, and other losses.

Acceptable Risk – A risk for which, for the purposes of life or work, we are prepared to accept as it is with no regard to its management. Society does not generally consider expenditure in further reducing such risks justifiable.

Tolerable Risk – A risk that society is willing to live with so as to secure certain net benefits in the confidence that it is being properly controlled, kept under review and further reduced as and when possible.

In some situations risk may be tolerated because the individuals at risk cannot afford to reduce risk even though they recognise it is not properly controlled.

Landslide Intensity – A set of spatially distributed parameters related to the destructive power of a landslide. The parameters may be described quantitatively or qualitatively and may include maximum movement velocity, total displacement, differential displacement, depth of the moving mass, peak discharge per unit width, kinetic energy per unit area.

Note: Reference should also be made to Figure 1 which shows the inter-relationship of many of these terms and the relevant portion of Landslide Risk Management.

APPENDIX B

LANDSLIDE TERMINOLOGY

The following provides a summary of landslide terminology which should (for uniformity of practice) be adopted when classifying and describing a landslide. It has been based on Cruden & Varnes (1996) and the reader is recommended to refer to the original documents for a more detailed discussion, other terminology and further examples of landslide types and processes.

Landslide:
The term ***landslide*** denotes "the movement of a mass of rock, debris or earth down a slope". The phenomena described as landslides are not limited to either the "land" or to "sliding", and usage of the word has implied a much more extensive meaning than its component parts suggest. Ground subsidence and collapse are excluded.

Classification of Landslides:
Landslide classification is based on Varnes (1978) system which has two terms: the first term describes the material type and the second term describes the type of movement.

The material types are ***Rock***, ***Earth*** and ***Debris***, being classified as follows:-

The material is either rock or soil.

Rock: is "a hard or firm mass that was intact and in its natural place before the initiation of movement".

Soil: is "an aggregate of solid particles, generally of minerals and rocks, that either was transported or was formed by the weathering of rock in place. Gases or liquids filling the pores of the soil form part of the soil".

Earth: "describes material in which 80% or more of the particles are smaller than 2mm, the upper limit of sand sized particles".

Debris: "contains a significant proportion of coarse material; 20% to 80% of the particles are larger than 2mm, and the remainder are less than 2mm".

The terms used should describe the displaced material in the landslide before it was displaced.

The types of movement describe how the landslide movement is distributed through the displaced mass. The five kinematically distinct types of movement are described in the sequence *fall*, *topple*, *slide*, *spread* and *flow*.

Figure B1 gives examples of the types of movement.

Combining the two terms gives classifications such as Rock fall, Rock topple, Debris slide, Debris flow, Earth slide, Earth spread etc.

(a)

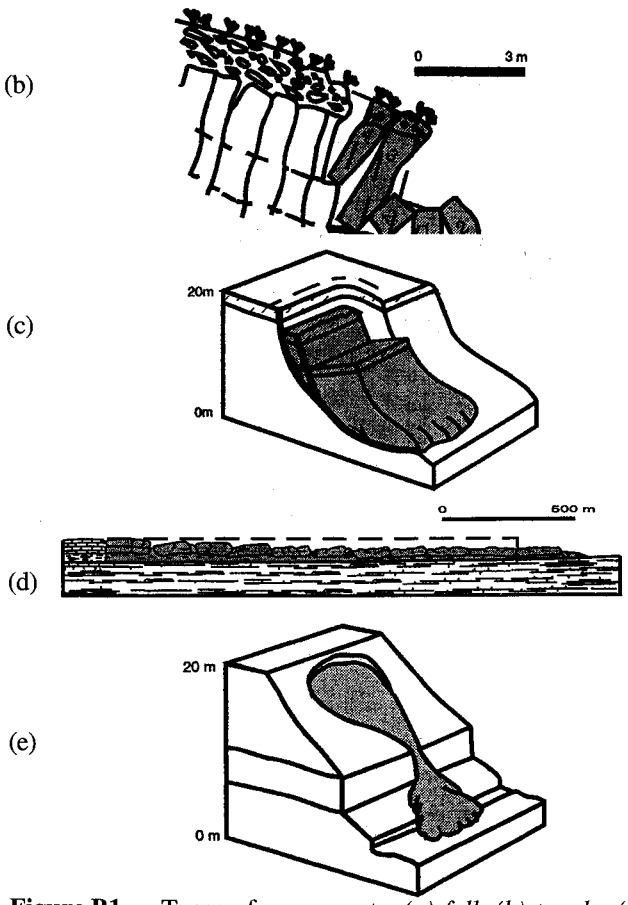


Figure B1 Types of movement: *(a) fall, (b) topple, (c) slide, (d) spread, (e) flow.* broken lines indicate original ground surfaces; arrows show portions of trajectories of individual particles of displaced mass; scales indicative for example chosen only (from "Landslides", copyright Registration Number 427735 of Consumer and Corporate Affairs, Canada, by kind permission of the author D.M. Cruden).

The name of a landslide can become more elaborate as more information about the movement becomes available. To build up the complete identification of the movement, descriptors are added in front of the two-term classification using a preferred sequence of terms. The suggested sequence provides a progressive narrowing of the focus of the descriptors, first by time and then by spatial location, beginning with a view of the whole landslide, continuing with parts of the movement, and finally defining the materials involved. The recommended sequence, as shown in Table B1, describes activity (including state, distribution and style) followed by descriptions of all movements (including rate, water content, material and type). Definitions of the terms in Table B1 are given in Cruden & Varnes (1996).

Second or subsequent movements in complex or composite landslides can be described by repeating, as many times as necessary, the descriptors used in Table B1. Descriptors that are the same as those for the first movement may then be dropped from the name.

For example, the very large and rapid slope movement that occurred near the town of Frank, Alberta, Canada, in 1903 was a *complex, extremely rapid, dry rock fall – debris flow.* From the full name of this landslide at Frank, one would know that both the debris flow and the rock fall were extremely rapid and dry because no other descriptors are used for the debris flow.

The full name of the landslide need only be given once; subsequent references should then be to the initial material and type of movement; for the above example, "the rock fall" or "the Frank rock fall" for the landslide at Frank, Alberta.

Landslide Features:

Varnes (1978, Figure 2.1t) provided an idealised diagram showing the features for a *complex earth slide – earth flow*, which has been reproduced here as Figure B2. Definitions of landslide dimensions are given in Cruden & Varnes (1996).

Activity

State	Distribution	Style
Active	Advancing	Complex
Reactivated	Retrogressive	Composite
Suspended	Widening	Multiple
Inactive	Enlarging	Successive
Dormant	Confined	Single
Abandoned	Diminishing	
Stabilised	Moving	
Relict		

Description of First Movement

Rate	Water Content	Material	Type
Extremely rapid	Dry	Rock	Fall
Very rapid	Moist	Earth	Topple
Rapid	Wet	Debris	Slide
Moderate	Very Wet		Spread
Slow			Flow
Very slow			
Extremely slow			

Note: Subsequent movements may be described by repeating the above descriptors as many times as necessary. These terms are described in more detail in Cruden & Varnes (1996) and examples are given.

Table B1 Glossary for forming names of landslides

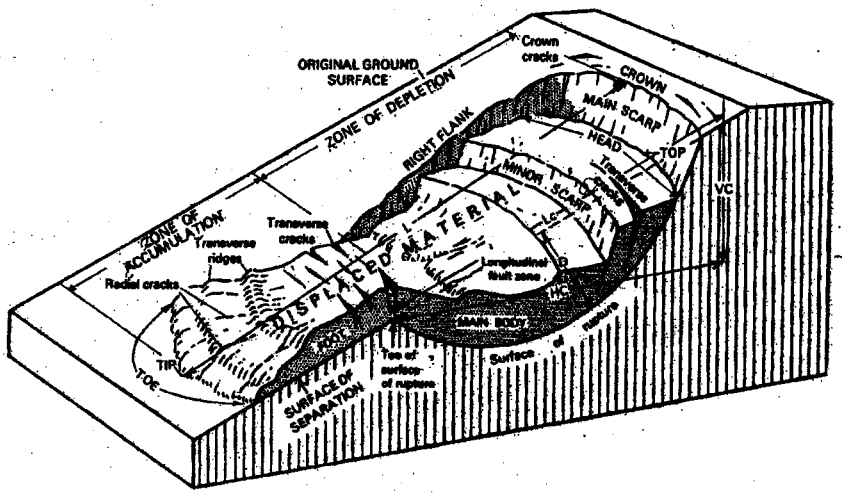


Figure B2: Block Diagram of Idealised Complex Earth Slide-Earth Flow (Varnes 1978, Figure 2.1t)

Rate of Movement:

Figure B3 shows the velocity scale proposed by Cruden & Varnes (1996) which rationalises previous scales. The term "creep" has been omitted due to the many definitions and interpretations in the literature.

Velocity Class	Description	Velocity (mm/sec)	Typical Velocity	Probable Destructive Significance
7	Extremely Rapid	5×10^{3}	5 m/sec	Catastrophe of major violence; buildings destroyed by impact of displaced material; many deaths; escape unlikely
6	Very Rapid	5×10^{1}	3 m/min	Some lives lost; velocity too great to permit all persons to escape
5	Rapid	5×10^{-1}	1.8 m/hr	Escape evacuation possible; structures; possessions, and equipment destroyed
4	Moderate	5×10^{-3}	13 m/month	Some temporary and insensitive structures can be temporarily maintained
3	Slow	5×10^{-5}	1.6 m/year	Remedial construction can be undertaken during movement; insensitive structures can be maintained with frequent maintenance work if total movement is not large during a particular acceleration phase
2	Very Slow	5×10^{-7}	15 mm/year	Some permanent structures undamaged by movement
	Extremely **SLOW**			Imperceptible without instruments; construction **POSSIBLE WITH PRECAUTIONS**

Figure B3: Proposed Landslide Velocity Scale and Probable Destructive Significance

REFERENCES AND ACKNOWLEDGEMENT

Cruden, D.M., & Varnes, D.J. (1996), "Landslide Types and Processes", Ch.3 in "Landslides. Investigation and Mitigation", Eds Turner, A.K. and Schuster, R.L. Special Report 247, Transport Research Board, National Research Council, Washington D.C. Extracts reprinted above by kind permission of the authors and publishers. Copies of the publication can be obtained from "Transport Research Board, National Research Council, 2101 Constitution Avenue, N.W., Washington D.C. 20418, USA.

IAEG (International Association of Engineering Geology) Commission on Landslides, (1990). Suggested nomenclature for landslides, Bulletin IAEG, No. 41, pp.13-16.

Varnes, D.J. (1978). Slope Movement Types and Processes. In *Special Report 176: Landslides: Analysis and Control* (R.L. Schuster and R.J. Krizek, eds.), TRB, National Research Council, Washington, D.C., pp.11-33.

WP/WLI (International Geotechnical Societies' UNESCO Working Party on World Landslide Inventory) (1990). A suggested method for reporting a landslide. Bulletin IAEG, 41, pp.5-12

WP/WLI (International Geotechnical Societies' UNESCO Working Party on World Landslide Inventory) (1993). A suggested method for describing the activity of a landslide. Bulletin International Association of Engineering Geology, 47: 53-57.

WP/WLI (International Geotechnical Societies' UENSCO Working Party on World Landslide Inventory) (1994). Multilingual Glossary for Landslides, Bitech Press, Vancouver, in press.

APPENDIX C

FREQUENCY ANALYSIS
A REVIEW OF THE METHODS AVAILABLE TO ESTIMATE THE PROBABILITY OF LANDSLIDING

(1) Assessment of the historic record of landsliding

In the simplest form this method consists of recording the number of landslides which occur each year in an area of interest, such as along a road or railway. It may be extended to include the type of sliding, e.g. on natural or constructed slopes, or on cuts and fills, and characteristics such as volume or area of landsliding. Chowdhury & Flentje (1998) discuss the use of a database to record such data in a systematic way.

Examples of this approach are given in: Morgan *et al* (1992) where the historic record of landsliding was used to assess the magnitude and probability of debris flows; Fell, Finlay & Mostyn (1996(a)), where records collected by the Geotechnical Engineering Office of Hong Kong were used to estimate the annual average probability of cut, fill, and retaining wall failures; examples which include rockfall are described in Moon *et al* (1992), Cruden (1997) and Moon *et al* (1996).

This method can be a useful way of estimating the average annual probability of landsliding, but usually does not discriminate between individual slopes and does not allow for the dependence of the landsliding on triggering factors, such as rainfall. A long representative period of record is needed, and even then there are potentially difficulties because of the non-linear relationship between the triggering event, e.g. rainfall and number of landslides, the influence of development, changes in vegetation, and runon and runoff of water. However, it can be a very valuable method for smaller landslides (e.g. in road cuts and fills), and as a check on more sophisticated methods.

(2) Empirical methods based on slope instability ranking system

These are methods which are devised by expert groups, and often are used for prioritising remedial works on roads, railways, and other constructed slopes. Examples are given in Koirala & Watkins (1988) and GEO (1995) for Hong Kong, and Mackay (1997) for railways. However, these are usually based on judgement for the factors to be included, may not be properly calibrated and therefore are often inaccurate, and are unable to quantify the probabilities. Flentje & Chowdhury (1999) indicate ranking of a landslide database on the basis of derived parameters such as volume, frequency or "hazard".

(3) Relationship to geomorphology and geology

This method is based on the principle put forward by Varnes (1984) that the past and present are guides to the future:

- hence it is likely that landsliding will occur where it has occurred in the past, and
- landslides are likely to occur in similar geological, geomorphological and hydrological conditions as they have in the past.

The method is the one most widely used in hazard and risk zoning studies, and is often performed with a judgemental, experience based approach, without quantification of the probability. Hence, the outputs are in qualitative terms, e.g. low, medium, high hazard or risk. Baynes & Lee (1998) discuss the role of geomorphology in landslide risk assessment.

The general issues in estimating the probability of landsliding in this method are discussed in Hutchinson (1988), Leroi (1996), and Soeters & Van Westen (1996). Some examples for specific projects are given in Siddle *et al* (1991), Carrera *et al* (1991 and 1992). Some details are given in Fell & Hartford (1997). Examples of where this method has been developed to a semi-quantitative level include Moon *et al* (1992) and Fell *et al* (1996(b)).

The use of geomorphology, geology and landslide records can be extended to include other factors such as slope angle, slope drainage, slope age, presence of groundwater, and evidence and history of instability; provided records are kept of such data. This was done by Finlay (1996) and reported in Fell *et al* (1996(a)) using the Geotechnical Engineering Office's

(GEO) data for 3,000 landslides in Hong Kong. In this approach the probability of landsliding for individual slopes was assessed, using factors calibrated on the past performance of the slopes over a 10 year period.

In some cases quantification was possible on a reasonably rigorous basis (e.g. for slopes or cuts) but in others, a considerable degree of judgement was necessary. It also became apparent that the quality of the data was a limitation, because of difficulties in obtaining information on a slope failures in difficult conditions (e.g. rain, darkness etc).

(4) Relating the historic record of landsliding to rainfall intensity and duration and frequency

These methods relate the historic occurrence of landsliding to rainfall intensity and duration, and in some cases, to antecedent rainfall. They have been used in rural areas (e.g. by Siddle *et al*, 1985, Kim *et al* 1992) to delineate rainfall which is likely to lead to extensive landsliding.

Lumb (1975(a)), Brand *et al* (1984) and Premchitt *et al* (1994) have developed methods for relating rainfall intensity for 1 hour to 24 hours, with and without antecedent rainfall, to predict the incidence of landsliding in constructed and natural slopes in Hong Kong. These, and the Kim *et al* (1992) methods have largely been developed to determine what rain conditions lead to extensive landsliding, so that warning systems can be instituted to keep the population away from the high hazard areas in such times. Fell *et al* (1988) carried out a similar study for Newcastle, NSW. Flentje & Chowdhury (1999) have related reactivation of existing landslides in an area of North Wollongong to antecedent rainfall and have derived Antecedent Rainfall Percentage Exceedance Time (ARPET) curves which give a measure of the probability. Threshold values of antecedent rainfall have been suggested for movement and "catastrophic failure".

Where the population of slopes is known, these methods can crudely estimate the average annual probability of any slope failing.

These methods generally have their uses, but are unable to allow discrimination between the relative probability of landsliding for different slopes within the population. In addition, they rely on the principles outlined in (3) above which may or may not apply, and need to be carefully applied to determine the critical rainfall duration and period of antecedent rainfall. For example, Premchitt *et al* (1994) have found that the 1 hour intensity is the most critical factor for Hong Kong's relatively small, shallower slides in constructed and natural slopes and that antecedent rainfall was not important, but Fell *et al* (1988) found that the prediction was best using antecedent rainfall up to 30 to 60 days for the larger, deeper landslides in their study area in Newcastle (NSW). Flentje & Chowdhury (1999) found 90 day antecedent rainfall gave a good predictor for reactivation of existing landslides in North Wollongong.

Finlay (1996), reported in Finlay *et al* (1997), has extended these approaches to relate the number of landslides to the rainfall intensity, duration and antecedent rainfall, using records of landsliding in Hong Kong taken by the Geotechnical Engineering Office, and very detailed rainfall data (5 and 15 minute data was used).

The concept developed allows the prediction of the number of landslides which may occur for say a 1 in 100 AEP rain event, within a given area. However, in this case (and probably more generally), the incidence of landsliding varies non linearly with rainfall and is markedly affected by data from a small number of heavy rain events. This makes the extrapolation uncertain. In addition, it becomes apparent that a critical feature is the areal extent of the rain event, yet such data is seldom available. As for the other examples of this method, it is not possible to assess the probability of landsliding of individual slopes, only the average (assuming the population of slopes is known).

The methods described above have been extended by some authors to include the slope of the ground, potential depth of sliding, and piezometric pressure parameters which are limited to rainfall and infiltration. Examples are given in Keefer *et al* (1987), Omura & Hicks (1992). These methods have the apparent virtue of properly modelling the sliding process, e.g. for shallow sliding leading to debris slides. However, they invariably oversimplify the piezometric pressure component of the analysis, which in fact dominates the calculation, by for example:

- using constant infiltration rates and/or permeability
- ignoring the non-linear effects of partial saturation on infiltration
- ignoring the heterogeneity of the slope – e.g. ignoring layering in the soil, root holes, infiltration from the rock below the soil, etc.
- not modelling 3-dimensional (or sometimes even 2-dimensional) effects across and up and down slope
- not modelling the rainfall intensity-duration properly.

These simplifications are necessary for analysis, but in the process of simplification, reality may be lost.

A further difficulty is that the analytical models sometimes do not model the actual slide mechanisms properly, and are really modelling detachment (sliding), not the landslide flow initiation which is often what is critical for the slope.

Unless such methods are calibrated by field performance of the slopes, which in effect lumps the variables together, they are not any better, and are probably worse, than the apparently less rigorous methods described above.

An important factor which should be considered is the relationship of landsliding to the ability of the surface drainage system to carry rainfall runoff. This is particularly important for road and rail line fills if the culverts do not have a high capacity. In these cases the frequency of sliding may directly relate to the annual exceedance probability of the drainage system being over-taxed.

(5) Direct assessment based on expert judgement

There are few examples of this approach in the literature. A form of this approach has been used in portfolio risk assessments for dams in Australia and USA. In this case the average annual failure rate (by slope instability) of dams is known from historic data (Foster *et al* 1998), and the probability for an individual dam is assessed from this as a starting point, allowing for steepness of the slope (or factor of safety), slope deformation, seepage etc.

(6) Modelling the primary variable e.g. piezometric pressure

The method outlined in Fell *et al* (1991) is an example of this approach, where piezometric levels recorded over some period (in that case 3 years) are related to rainfall, and the probability of various piezometric levels being reached is assessed by analysing the modelled piezometric levels for the period of record (in that case 100 years). Other examples are given in Haneberg (1991) and Okunushi & Okumura (1987).

The method is ideal in principle for a single, relatively deep-seated landslide. However, in reality it is difficult to achieve any accuracy in the modelling because of the complex infiltration processes involved, heterogeneity of the soil and rock in the slope, and groundwater seeping into the slide from below. It is also apparent that a lengthy period of calibration (years) is likely to be necessary, to experience a range of rainfall and piezometric conditions.

(7) Application of formal probabilistic methods

There has been extensive research into formal probabilistic analysis of slopes. The state of the art for these calculation methods is well established, and the methods can be applied with confidence. Mostyn & Fell (1997), Li 1991, 1992(a) and 1992(b) give overviews. The application of such methods should include consideration of the following aspects to give realistic outcomes:

- surface and subsurface geometry
- hydrogeology
- variation of pore water pressures with time
- material strengths
- discontinuities in rocks, including persistence
- spatial variation of parameters.

In addition, the problem must be viewed as a system of potential failure surfaces rather than just a single, sometimes critical, failure surface. Various "levels" of data on uncertainty should be used, these range from pure judgemental to statistically robust parameter estimation.

It should be noted that often the greatest area of uncertainty is the prediction of pore pressures in a slope, and no degree of sophistication on the uncertainty in shear strength and geometry can give realistic answers unless a defensible, properly modelled assessment is made of pore pressures. In other cases, properties such as defect persistence are most critical and must be modelled correctly.

This is difficult to achieve and leads back to the more subjective methods or to a combination of subjective and analytical methods. Roberds (1990) describes methods for developing defensible subjective probability assessments.

Quantitative and semi-quantitative methods for estimating the probability of landslides are well developed and are equally valid.

The method to be used will depend on the level of the study, e.g. feasibility or detailed design; and on whether the slope is constructed or natural. Quantitative methods are more applicable to constructed slopes, where detailed investigation, laboratory testing and monitoring/prediction of pore pressures is possible. For natural slopes, semi-quantitative methods are more likely to be applicable.

APPENDIX C

REFERENCES

Baynes, F.J. and Lee, M. (1998). Geomorphology in landslide risk analysis, an interim report. Proceedings of the Eighth Congress of the Int. Assoc. of Engineering Geologists, ed. Moore and Hungr, Balkema, pp.1129-1136.

Brand, E., Premchitt, J. and Phillipson, H. (1984). Relationship between rainfall and landslides in Hong Kong. In Landslides, Proc. Fourth Int. Symp. On Landslides, Toronto, Canada. BiTech Publishers, Vancouver, Canada.

Carrera, A., Cardinali, M., Detti, R. Guzzetti, F., Pasqui, V. and Reichenbach, P. (1991). GIS techniques and statistical models in evaluating landslide hazard. Earth Surface Processes and Landforms, 16(5), pp.427-445.

Carrera, A., Cardinali, M. and Guzzetti, F. (1992). Uncertainty in assessing landslide hazard and risk. ITC Journal, 1992-2, pp.172-183.

Chowdhury, R.N. and Flentje, P.N. (1998). A landslide database for landslide database for landslide hazard assessment. Proc. Second Intnl Conf. on Environmental Management. Feb.10-13. Wollongong, Australia. Eds Sivakumar, M. and Chowdhury, R.N., pp.1229-1239, Elsevier, London.

Cruden, D.M. (1997). Estimating the risks from landslides using historical data; in "Landslide Risk Assessment", Cruden and Fell (eds.), Balkema, pp. 277-284.

Fell, R., Mostyn, G., O'Keeffe, L. and Maguire, P. (1988). Assessment of the probability of rain induced landsliding. Fifth Australia-New Zealand Conference on Geomechanics, pp.72-77.

Fell, R., Chapman, T.G. and Maguire, P.K. (1991). A model for prediction of piezometric levels in landslides. Proc. Conf. On slope Stability Engineering Developments and applications. The Institution of Civil Engineers, Isle of Wight, pp.37-42.

Fell, R., Finlay, P.J. and Mostyn, G.R. (1996(a)). Framework for assessing the probability of sliding of cut slopes. Proc. Seventh Int. Symp. on Landslides, Trondheim. Balkema, Rotterdam, The Netherlands, Vol.1, 201-208.

Fell, R., Walker, B.F. and Finlay, P.J. (1996(b)). Estimating the probability of landsliding. Proc. 7th Australia New Zealand Conf. On Geomechanics, Adelaide. Institution of Engineers Australia, Canberra, pp.304-311.

Fell, R. and Hartford, D. (1997). Landslide Risk Management, in "Landslide Risk Assessment", Cruden and Fell (eds.), Balkema, Rotterdam, pp.51-110.

Finlay, P.J. (1996). The risk assessment of slopes. PhD Thesis, School of Civil Engineering, University of New South Wales.

Finlay, P.J., Fell, R. and Maguire, P.K. (1997). The relationship between the probability of landslide occurrence and rainfall. Canadian Geotech Journal Vol.34, No.6, pp.811-824.

Flentje, P.V. and Chowdhury, R.N. (1999). Quantitative Landslide Hazard Assessment in an Urban Area. Proc. Eighth Australia New Zealand Conf. on Geomechanics. Ed. Dr. Nihal Vitharana. Feb.15-17, Hobart, Tasmania. Institution of Engineers, Australia.

Foster, M.A., Fell, R. and Spannagle, M. (1998). Analysis of embankment dam incidents UNICIV Report No. R-374, The University of New South Wales, Sydney 2052. ISBN: 85841 3493.

GEO (1995). New Priority Classification System for Soil Cut Slopes, Geotechnical Engineering Office Special Project Report SPR 6/95, Civil Engineering Department, Hong Kong.

Haneberg, W. (1991). Observation and analysis of pore pressure fluctuations in a thin colluvial landslide complex near Cincinnati, Ohio. Engineering Geology, 31, pp.159-184.

Hutchinson, J.N. (1988). General Report: Morphological and geotechnical parameter of landslides in relation to geology and hydrogeology. In Landslides, Proc. Fifth Int. Symp. on Landslides, (Ed. C. Bonnard), Lausanne, Switzerland, Vol.1, pp.3-35.

Keefer, D.K., Wilson, R.C., Mark, R.K., Brabb, E.E., Brown, W.M., Ellen, S.D., Harp, F.L., Wieczoreck, O.F., Alger, C.S. and Zatkin, R.S. (1987). Real time landslide warning during heavy rainfall. Science, Vol.238, pp.921-925.

Kim, S.K., Hong, W.P. and Kim, Y.M. (1992). Prediction of rainfall-triggered landslides in Korea. In Landslides, Proc. Sixth Int. Symp. on Landslides, Christchurch, New Zealand. A.A. Balkema, Rotterdam, The Netherlands.

Koirala, N.P. and Watkins, A.T. (1988). Bulk appraisal of slopes in Hong Kong. In Landslides, Proc. Of the Fifth Int. Symp. on Landslides (Ed. C. Bonnard), Lausanne, Switzerland. A.A. Balkema, Rotterdam, The Netherlands, Vol.2, pp.1181-1186.

Leroi, E. (1996). Landslide hazard – Risk maps at different scales: objectives, tools and developments. In Landslides, Proc. Int. Symp. on Landslides, Trondheim, (Ed. K. Senneset), pp.35-52.

Li, K.S. (1991). Point estimate methods in geotechnics. Proc. Asian Pacific Conference on Computational Mechanics, Hong Kong, 1: 827-832.

Li, K.S. (1992(a)). A point estimate method for calculating the reliability index of slopes. Proc. 6th Australia-New Zealand Conference on Geomechanics, Christchurch, pp.448-451.

Li, K.S. (1992(b)). Some common mistakes in probabilistic analysis of slopes. Proc. 6th International Symposium on Landslides, Christchurch, 1: 475-480.

Lumb, P. (1975). Slope failures in Hong Kong. Quarterly Journal of Engineering Geology, 8, pp.31-65.

MacKay, C.H. (1997). Management of rock slopes on the Canadian Pacific Highway, in Landslide Risk Assessment, Cruden, D.M. and Fell, R. Eds. Balkema, pp.271-276.

Moon, A.T., Olds, R.J., Wilson, R.A. and Burman, B.C. (1992). Debris flow zoning at Montrose, Victoria. In Landslides, Proc. Sixth Int. Symp. on Landslides (Ed. D.H. Bell), Christchurch, New Zealand. A.A. Balkema, Rotterdam, The Netherlands, Vol.2, pp.1015-1022.

Moon, A., Robertson, M. and Davies, W. (1996). Quantifying rockfall risk using a probabilistic toppling failure model. Proc. 7th Int. Symp. On Landslides, Trondheim, 17-21 June 1996. K. Senneset Edition. Balkema Rotterdam.

Morgan, G.C., Rawlings, G.E. and Sobkowicz, J.C. (1992). Evaluating total risk to communities from large debris flows. In Geotechnique and Natural Hazards, Proc. Geohazards '92 Symposium. BiTech Publishers, Canada, pp.225-236.

Mostyn, G. and Fell, R. (1997). Quantitative and Semi quantitative estimation of the probability of landslides in Landslide Risk Assessment, Cruden and Fell (eds.), Balkema, Rotterdam, pp.297-316.

Okunishi, K. and Okumura, T. (1987). Groundwater models for mountain slopes. In Slope Stability (Eds. M.G. Anderson and K.S. Richards). Wiley, pp.265-285.

Omura, H. and Hicks, D. (1992). Probability of landslides in hill country. In Landslides, Proc. Sixth Int. Symp. on Landslides, Christchurch, New Zealand. A.A. Balkema, Rotterdam, The Netherlands.

Premchitt, J., Brand, E.W. and Chen, P.Y.M. (1994). Rain-induced landslides in Hong Kong 1972-1992. Asia Engineer, June, pp.43-51.

Roberds, W.J. (1990). Methods for developing defensible subjective probability assessments. Proceedings of the Transportation Research Board, No. 1288, National Research Council, Washington DC, pp.183-190.

Siddle, H.J., Jones, D.B. and Payne, H.R. (1991). Development of a methodology for landslip potential mapping in the Rhondda Valley. In Slope Stability, Isle of Wight, (Ed. R.J. Chandler). Thomas Telford, p.137-148.

Siddle, R.C., Pearce, A.J. and O'Loughlin, C.L. (1985). Hillslope Stability and Landuse. American Geophysical Union, Water Resources Monograph.

Soeters, R. and van Westen, C.J. (1996). Slope instability recognition, analysis, and zonation. In Landslides Investigation and Mitigation. Transportation Research Board, National Research Council (Eds. A.K. Turner and R.L. Schuster), pp.129-177.

Varnes, D.J. and the International Association of Engineering Geology Commission on Landslides and Other Mass Movements (1984). Landslide hazard zonation: A review of principles and practice. Natural Hazards, vol.3, Paris, France. UNESCO, 63p.

APPENDIX D

METHOD FOR ESTIMATING TRAVEL DISTANCE OF LANDSLIDES – FOR SLIDES WHICH BREAK UP, AND FOR FLOWS

The following information is from Finlay *et al* (1999).

Allowance should be made for the likely mechanics of movement. It should be noted that for fills which are well compacted (and hence dilatant in shear), the larger values of the apparent friction angle $\emptyset_a$ or F (=tan $\emptyset_a$) are likely to apply, while for fills which may collapse and flow, the low values of F are likely to apply. Risk calculations should allow for the uncertainty in the estimated F (=tan $\emptyset_a$) value.

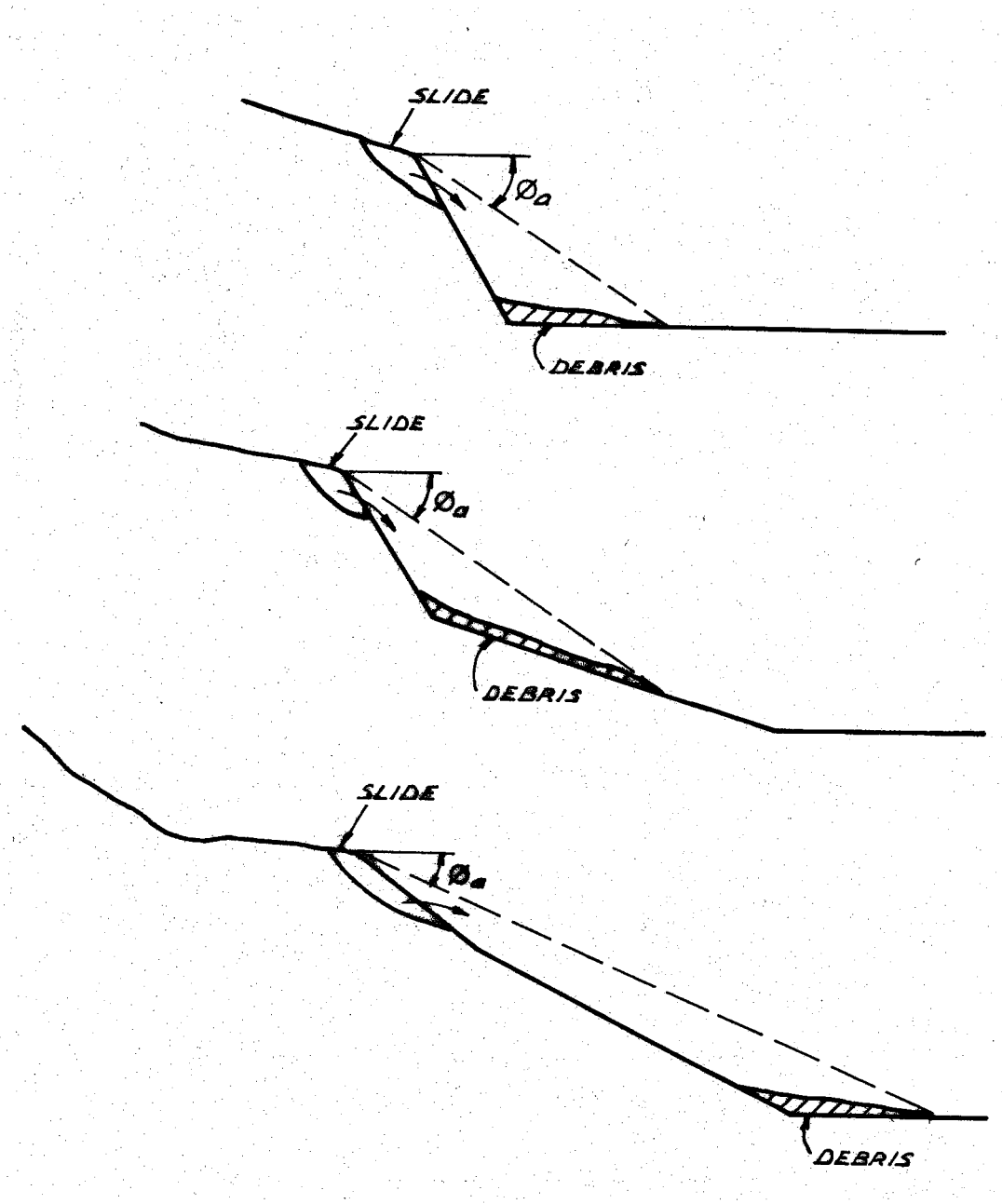


Figure D1 Examples of the Apparent Friction Angle $\emptyset_a$

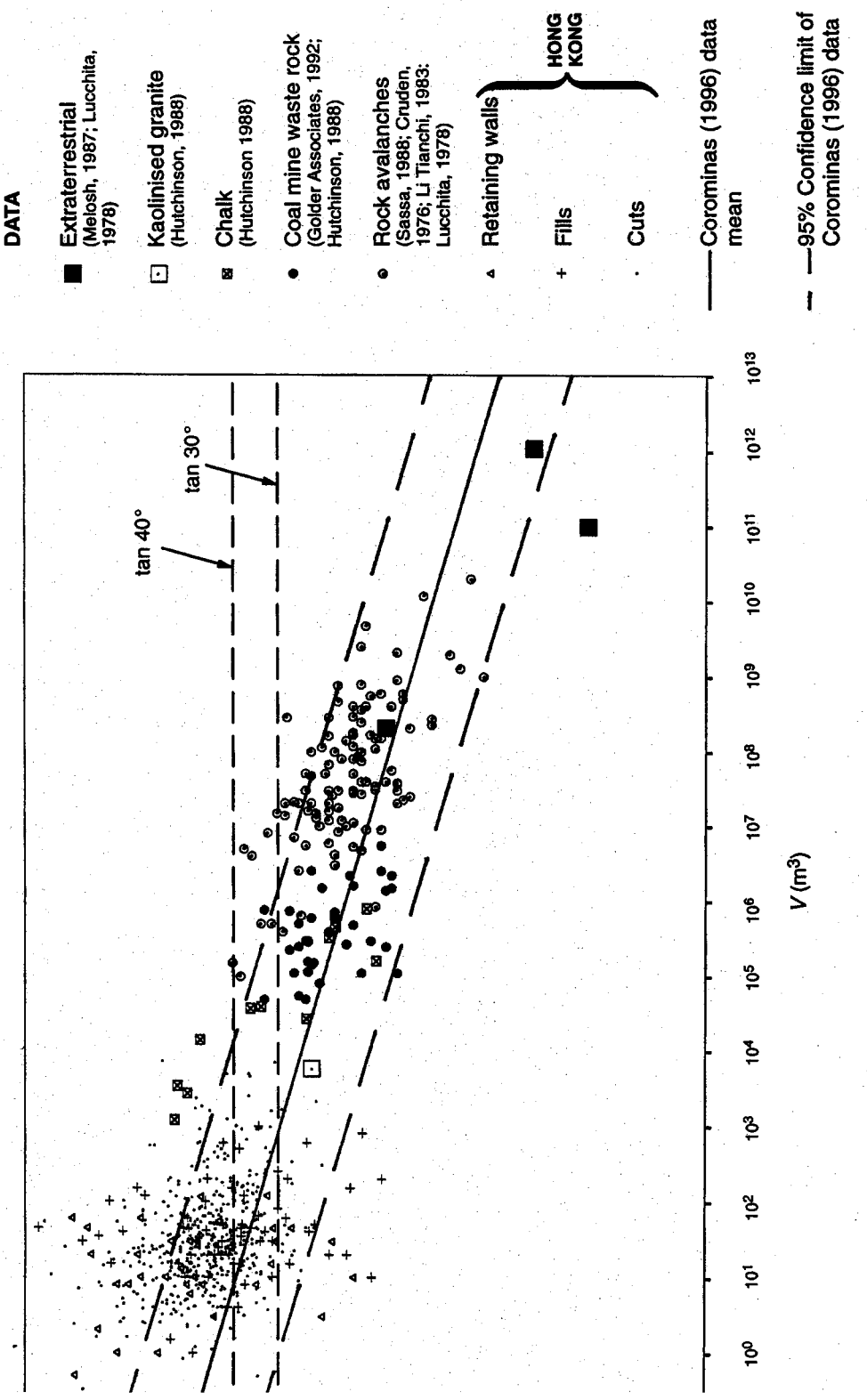
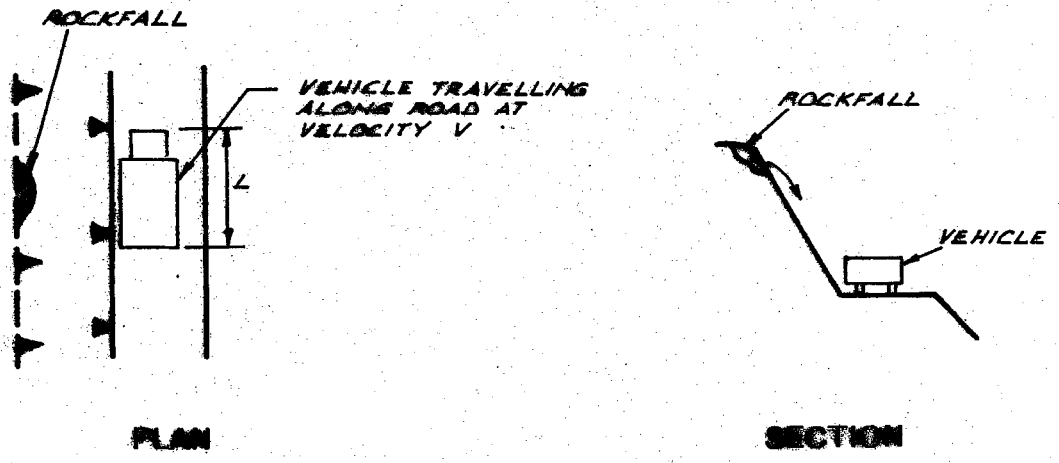


Figure D2: Plot of logF versus logV for various landslides (Finlay *et al* 1999)
where F = $\tan\phi_a$ (refer to Figure D1)
V = volume of the landslide

Note: The Hong Kong data has been derived from landslide records. It should be used with care as it includes some wash outs and channelised flows as well as incipient failures.

APPENDIX E

METHOD FOR CALCULATING THE PROBABILITY OF A ROCK FALLING ONTO A MOVING VEHICLE



$$P_{(S)} = 1-(1-(P_{(S:H)}))^{N_R} \quad \text{.....(E1)}$$

WHERE

$P_{(S)}$ = PROBABILITY OF ONE OR MORE VEHICLES BEING HIT
$P_{(S:H)}$ = PROBABILITY OF A VEHICLE OCCUPYING THE PORTION OF THE ROAD ONTO WHICH ROCK FALLS
N_R = NUMBER OF ROCK FALLS/DAY

AND

$$P_{(S:H)} = \frac{N_v}{24} \cdot \frac{L}{1000} / V_v \quad \text{.....(E2)}$$

N_V = NUMBERS OF VEHICLES/DAY
L = LENGTH OF VEHICLE (m)
V_V = VELOCITY OF VEHICLE/HOUR (km/hour)

NOTE: N_R can be estimated from maintenance records, impact marks on the roadway, the geology, geometry of the slope. Allowance should be made for proximity to the slope, and the presence or absence of rock catch ditches.

APPENDIX F

SUMMARY OF HONG KONG VULNERABILITY RANGES FOR PERSONS, AND RECOMMENDED VALUES FOR LOSS OF LIFE FOR LANDSLIDING IN SIMILAR SITUATIONS

The following table is adapted from Finlay *et al* (1999).

Case	Range in Data	Recommended Value	Comments
Person in Open Space			
If struck by a rockfall	0.1 – 0.7	0.5	May be injured but unlikely to cause death
If buried by debris	0.8 – 1.0	1.0	Death by asphyxia almost certain
If not buried	0.1 – 0.5	0.1	High chance of survival
Person in a Vehicle			
If the vehicle is buried/crushed	0.9 – 1.0	1.0	Death is almost certain
If the vehicle is damaged only	0 – 0.3	0.3	High chance of survival
Person in a Building			
If the building collapses	0.9 – 1.0	1.0	Death is almost certain
If the building is inundated with debris and the person buried	08 – 1.0	1.0	Death is highly likely
If the debris strikes the building only	0 – 0.1	0.05	Very high chance of survival

The above data should be applied with common sense, taking into account the circumstances of the landslide being studied. Judgement may indicate values other than the recommended value are appropriate for a particular case.

APPENDIX G

LANDSLIDE RISK ASSESSMENT – EXAMPLE OF QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

Qualitative Measures of Likelihood

Level	Descriptor	Description	Indicative Annual Probability
A	ALMOST CERTAIN	The event is expected to occur	$>\approx 10^{-1}$
B	LIKELY	The event will probably occur under adverse conditions	$\approx 10^{-2}$
C	POSSIBLE	The event could occur under adverse conditions	$\approx 10^{-3}$
D	UNLIKELY	The event might occur under very adverse circumstances	$\approx 10^{-4}$
E	RARE	The event is conceivable but only under exceptional	$\approx 10^{-5}$
F	NOT CREDIBLE	circumstances. The event is inconceivable or fanciful	$<10^{-6}$

Note: "≈" means that the indicative value may vary by say ±½ order of magnitude, or more.

Qualitative Measures of Consequences to Property

Level	Descriptor	Description
1	CATASTROPHIC	Structure completely destroyed or large scale damage requiring major engineering works for stabilisation.
2	MAJOR	Extensive damage to most of structure, or extending beyond site boundaries requiring significant stabilisation works.
3	MEDIUM	Moderate damage to some of structure, or significant part of site requiring large stabilisation works.
4	MINOR	Limited damage to part of structure, or part of site requiring some reinstatement/stabilisation works.
5	INSIGNIFICANT	Little damage

Note: The "Description" may be edited to suit a particular case.

Qualitative Risk Analysis Matrix – Level of Risk to Property

LIKELIHOOD	CONSEQUENCES to PROPERTY				
	1: CATASTROPHIC	2: MAJOR	3: MEDIUM	4: MINOR	5: INSIGNIFICANT
A – ALMOST CERTAIN	VH	VH	H	H	M
B – LIKELY	VH	H	H	M	L-M
C – POSSIBLE	H	H	M	L-M	VL-L
D – UNLIKELY	M-H	M	L-M	VL-L	VL
E – RARE	M-L	L-M	VL-L	VL	VL
F – NOT CREDIBLE	VL	VL	VL	VL	VL

Risk Level Implications

Risk Level		Example Implications(1)
VH	VERY HIGH RISK	Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to acceptable levels; may be too expensive and not practical
H	HIGH RISK	Detailed investigation, planning and implementation of treatment options required to reduce risk to acceptable levels
M	MODERATE RISK	Tolerable provided treatment plan is implemented to maintain or reduce risks. May be accepted. May require investigation and planning of treatment options
L	LOW RISK	Usually accepted. Treatment requirements and responsibility to be defined to maintain or reduce risk
VL	VERY LOW RISK	Acceptable. Manage by normal slope maintenance procedures

Note: (1) The implications for a particular situation are to be determined by all parties to the risk assessment; these are only given as a general guide.

(2) Judicious use of dual descriptors for Likelihood, Consequence and Risk to reflect the uncertainty of the estimate may be appropriate in some cases.

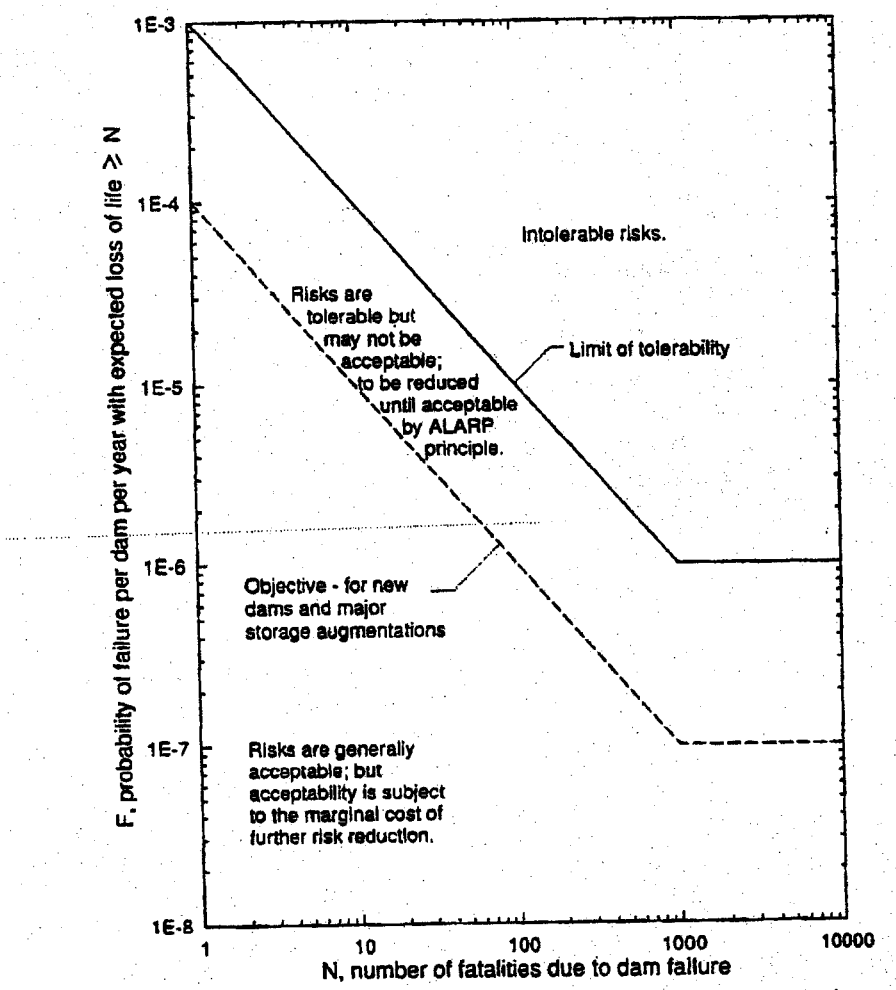
APPENDIX H

ACCEPTABLE AND TOLERABLE RISK CRITERIA

(a) Summary of Individual Risk Criteria (taken from Fell & Hartford, 1997)

Source	Lower Bound (Acceptable)	Upper Limit (Tolerable)
Health and Safety Executive (1989a)	10^{-6} of dangerous dose equivalent to 0.33×10^{-6}	10^{-5} of dangerous dose equivalent to 0.33×10^{-5}
Health and Safety Executive (1988)	10^{-6} broadly acceptable	10^{-3}, divide between just tolerable and intolerable 10^{-4} any individual member of public from large scale industrial hazard
New South Wales Department of Planning (1994)		10^{-6} residential 5×10^{-5} residential
Hong Kong Government Planning (1994)	Not defined	10^{-5}
BC Hydro (1993)		10^{-4}
ANCOLD (1994) Existing dams		10^{-5} average 10^{-4} person most at risk
USBR (Von Thun, 1996)	None stated	
Finlay and Fell (1997)	10^{-5} to 10^{-6} 10^{-3} to 10^{-4} acceptable for property	10^{-3} tolerated

(b) ANCOLD Amended Interim Societal Risk Criteria (ANCOLD 1998) (Subject to Further Revision)



Important note : Where fatalities are expected, as part of a risk-based decision at a specific dam, consultation with the affected public is required as part of the final decision process.

APPENDIX I

SUMMING RISKS FROM A NUMBER OF LANDSLIDE HAZARDS

The following is taken from Fell & Hartford (1997)

Considering risk due to landsliding on a highway, how do we sum the risk? What criteria should be considered? If we have a highway between Towns A and B which is 30km in length and has the following landslide hazards:

- Rockfall from 40 engineered cuttings
- Debris slides from 25 natural slopes
- Potential large scale (say 1 million m^3) fast moving landsliding from natural slopes on the highway at one location
- Potential collapse of 5 fills supporting the road.

Let us also assume the owner of this highway is also responsible for a further 2000km of highways in the state.

Some questions which need to be answered for the management of landslide risk are:

Is it required that acceptable individual risk and societal risk criteria are met for

(i) each landslide hazard above, i.e. each cutting or single debris slide

OR (ii) for all cuttings (only) on the highway from A to B

OR (iii) for all landslide hazards on the highway from A to B

OR (iv) is it required that the acceptable risk criteria are met for all landslide hazards under the management of the owner of the highway properly modelling the traffic and hazard to represent the overall picture?

OR (v) is it required that the acceptable risk criteria are met for all landslide hazards on all roads in the state regardless of the owner?

(vi) what if there are other hazards on the road(s), e.g. flood, avalanche, bridge collapse? Should these risks be added to the risks due to landslide?

(vii) should the risk due to landsliding affecting the population in their place of residence and workplace, for example, also be considered.

To the authors, it seems clear that:

(i) and (ii) Are clearly not the case, since persons driving from A to B are not particularly interested in each cutting, or individual debris slide (or cuttings separate from debris slides). They are interested in the landslide risk in travelling from A to B.

(iii) May be applicable, but only if the highway is "special", and separate in the mind of the population.

(iv) Would seem more likely to apply, if it is one of many similar highways.

(v) Would seem possible – the public are unlikely to differentiate between different owners of highways.

(vi) Should in principle apply. In practice, apart from avalanches, the other hazards may contribute little to risk.

(vii) Would seem unlikely to be required of society in most situations, as they would possibly separate the highway hazard from the others. However, the situation would be less clear in a place like Hong Kong, where government imposes controls on all landslide related works, and is known to do so by the population.

APPENDIX J

SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

	GOOD ENGINEERING PRACTICE	*POOR ENGINEERING PRACTICE*
ADVICE		
GEOTECHNICAL ASSESSMENT	Obtain advice from a qualified, experienced geotechnical consultant at early stage of planning and before site works.	Prepare detailed plan and start site works before geotechnical advice.
PLANNING		
SITE PLANNING	Having obtained geotechnical advice, plan the development with the risk arising from the identified hazards and consequences in mind.	Plan development without regard for the Risk.
DESIGN AND CONSTRUCTION		
HOUSE DESIGN	Use flexible structures which incorporate properly designed brickwork, timber or steel frames, timber or panel cladding. Consider use of split levels. Use decks for recreational areas where appropriate.	Floor plans which require extensive cutting and filling. Movement intolerant structures.
SITE CLEARING	Retain natural vegetation wherever practicable.	Indiscriminately clear the site.
ACCESS & DRIVEWAYS	Satisfy requirements below for cuts, fills, retaining walls and drainage. Council specifications for grades may need to be modified. Driveways and parking areas may need to be fully supported on piers.	Excavate and fill for site access before geotechnical advice.
EARTHWORKS	Retain natural contours wherever possible.	Indiscriminant bulk earthworks.
CUTS	Minimise depth. Support with engineered retaining walls or batter to appropriate slope. Provide drainage measures and erosion control.	Large scale cuts and benching. Unsupported cuts. Ignore drainage requirements
FILLS	Minimise height. Strip vegetation and topsoil and key into natural slopes prior to filling. Use clean fill materials and compact to engineering standards. Batter to appropriate slope or support with engineered retaining wall. Provide surface drainage and appropriate subsurface drainage.	Loose or poorly compacted fill, which if it fails, may flow a considerable distance including onto property below. Block natural drainage lines. Fill over existing vegetation and topsoil. Include stumps, trees, vegetation, topsoil, boulders, building rubble etc in fill.
ROCK OUTCROPS & BOULDERS	Remove or stabilise boulders which may have unacceptable risk. Support rock faces where necessary.	Disturb or undercut detached blocks or boulders.
RETAINING WALLS	Engineer design to resist applied soil and water forces. Found on rock where practicable. Provide subsurface drainage within wall backfill and surface drainage on slope above. Construct wall as soon as possible after cut/fill operation.	Construct a structurally inadequate wall such as sandstone flagging, brick or unreinforced blockwork. Lack of subsurface drains and weepholes.
FOOTINGS	Found within rock where practicable. Use rows of piers or strip footings oriented up and down slope. Design for lateral creep pressures if necessary. Backfill footing excavations to exclude ingress of surface water.	Found on topsoil, loose fill, detached boulders or undercut cliffs.
SWIMMING POOLS	Engineer designed. Support on piers to rock where practicable. Provide with under-drainage and gravity drain outlet where practicable. Design for high soil pressures which may develop on uphill side whilst there may be little or no lateral support on downhill side.	
DRAINAGE		
SURFACE	Provide at tops of cut and fill slopes. Discharge to street drainage or natural water courses. Provide general falls to prevent blockage by siltation and incorporate silt traps. Line to minimise infiltration and make flexible where possible. Special structures to dissipate energy at changes of slope and/or direction.	Discharge at top of fills and cuts. Allow water to pond on bench areas.
SUBSURFACE	Provide filter around subsurface drain. Provide drain behind retaining walls. Use flexible pipelines with access for maintenance. Prevent inflow of surface water.	Discharge roof runoff into absorption trenches.
SEPTIC & SULLAGE	Usually requires pump-out or mains sewer systems; absorption trenches may be possible in some areas if risk is acceptable. Storage tanks should be water-tight and adequately founded.	Discharge sullage directly onto and into slopes. Use absorption trenches without consideration of landslide risk.
EROSION CONTROL & LANDSCAPING	Control erosion as this may lead to instability. Revegetate cleared area.	Failure to observe earthworks and drainage recommendations when landscaping.
DRAWINGS AND SITE VISITS DURING CONSTRUCTION		
DRAWINGS	Building Application drawings should be viewed by geotechnical consultant	
SITE VISITS	Site Visits by consultant may be appropriate during construction/	
INSPECTION AND MAINTENANCE BY OWNER		
OWNER'S RESPONSIBILITY	Clean drainage systems; repair broken joints in drains and leaks in supply pipes. Where structural distress is evident see advice. If seepage observed, determine causes or seek advice on consequences.	

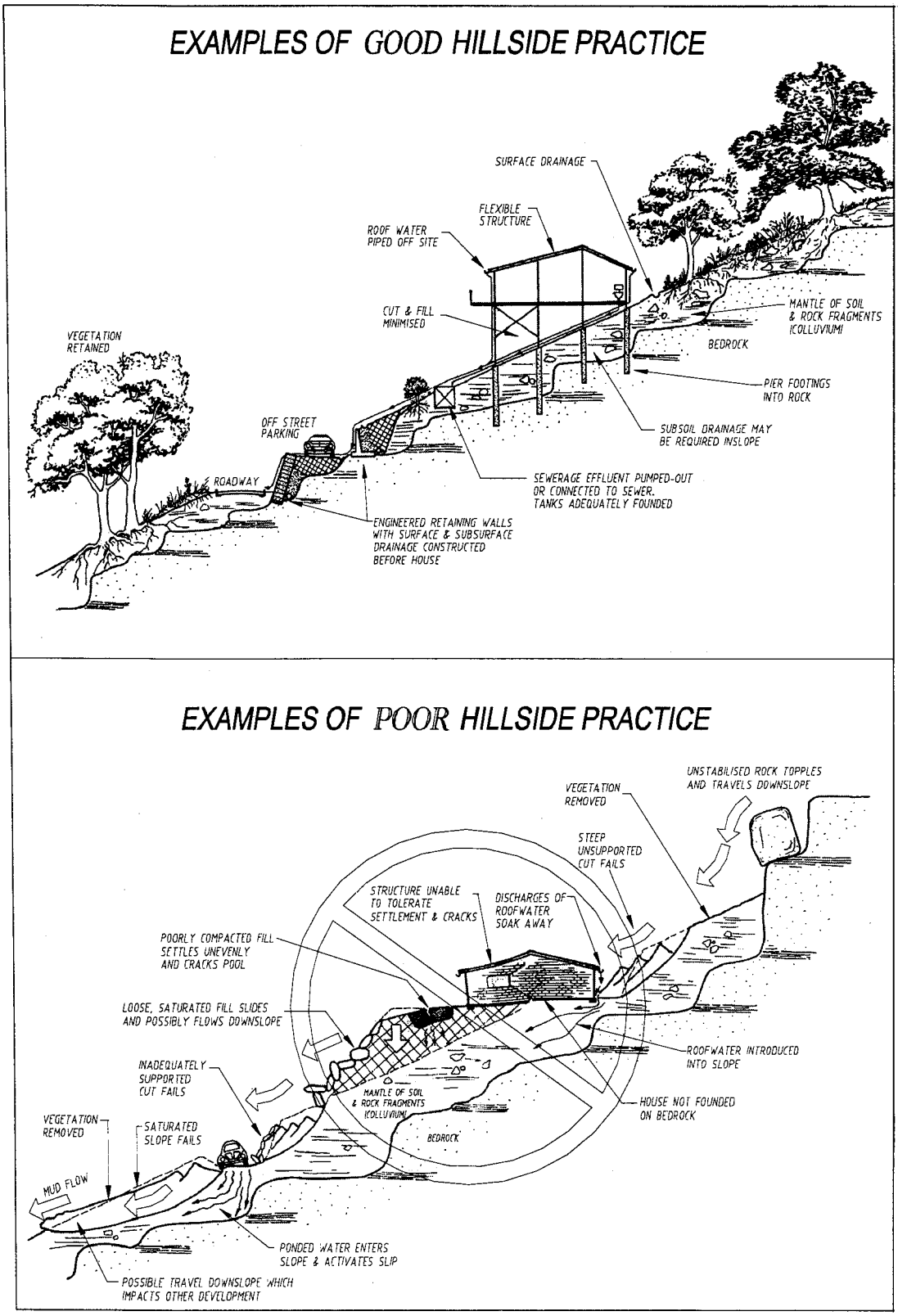


Figure J1 Illustrations of Good and Poor Hillside Practice

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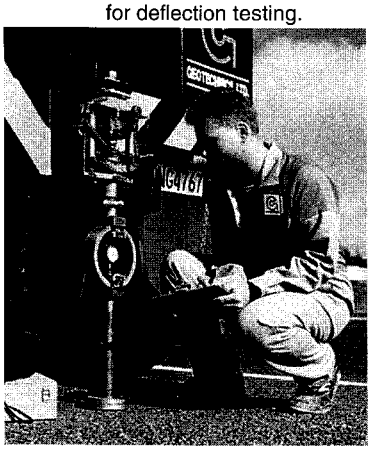

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The unit has also been designed to perform Californian Bearing Ratio and Plate Bearing Tests and has built in facilities and equipment for the performance of these tests.

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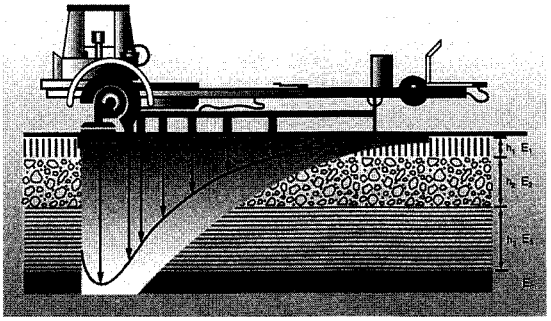

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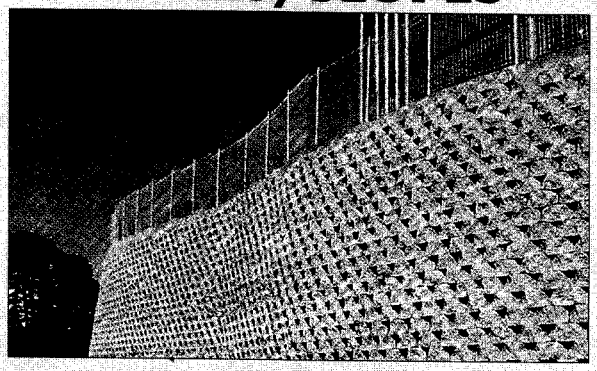
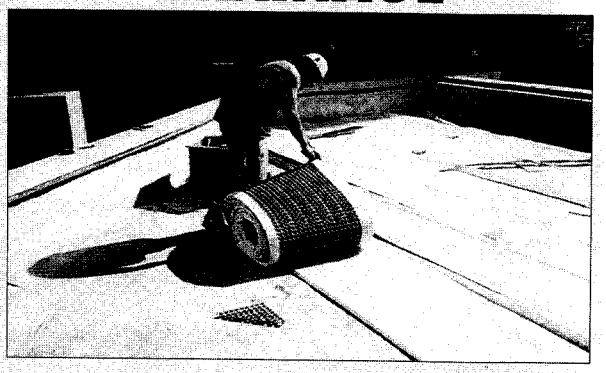


When the need is to hold the ground, we have a range of products for every situation from large scale hillside reinforcement to decorative retaining walls

DRAINAGE



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