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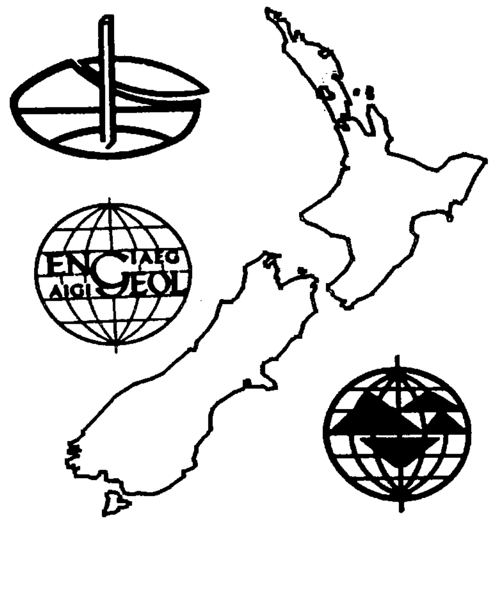

A NEWSLETTER OF THE N.Z. GEOTECHNICAL SOCIETY

NZ GEOMECHANICS NEWS

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NO. 56 DECEMBER 1998

CONTENTS

NOTES FOR CONTRIBUTORS

NZ Geomechanics News is a newsletter for which we seek contributions of any sort for future editions. The following comments are offered to assist contributors:

- Technical contributions can include any of the following:
  - technical papers which may, but need not necessarily be, of a standard which would be required by the international journals and conferences
  - technical notes
  - comments on papers published in Geomechanics News
  - descriptions of geotechnical projects of special interest.

- General articles for publication may include:
  - letters to the 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. Diagrams and tables should be of size and quality for direct reproduction. Photographs should be good contrast black and white gloss prints and of a suitable size for mounting to magazine format. *NZ Geomechanics News* is a magazine 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.

Stephen A Crawford
EDITOR

THIS IS A REGISTERED PUBLICATION

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. Authors must be consulted before papers are cited in other publications.

Persons interested in applying for **Membership of the Society** are invited to complete the application form at the back of the newsletter. The basic subscription rates are given on the information pages at the rear of this issue. These rates are supplemented according to which of the international societies, (namely Soil Mechanics, Rock Mechanics or Engineering Geology) the member wishes to be affiliated. Members of the Society are required to affiliate to at least one International Society.

Editor: S A Crawford
P O Box 5271
Wellesley Street
AUCKLAND

Phone: (09) 355 6054
Fax: (09) 307 0265
E-mail: scrawford@tonkin.nz

Advertising: D Fellows
6 Sylvan Valley Ave
Titirangi
AUCKLAND

Phone: (09) 817 7759
Fax: (09) 817 7035
Email: dfellows@xtra.co.nz

NEW EDITORIAL TEAM

There has been a very healthy response to the magazine this issue with a barrage of letters to the editor, technical notes and articles. Just as well there are more editors around to take the flak. The assistance of Doug Johnson and Tony Cowbourne has been invaluable and the 'more-than' secretarial help provided by Debbie Fellows has given this issue a life of its own. Keep those letters and articles rolling in.

Competitions

Two new competitions (with prizes!) are on offer – one for your best photo and the other a settlement prediction exercise. The latter especially should be a bit of fun and an opportunity to bid your cigarette packet calculation against the best the Universities will (hopefully) offer. The site is located in the North Island and is a chance to apply some recently presented relationships for CPT'S in New Zealand. We're still organising the load/site information and lab data, but register as soon as possible and we will forward a package to you. $100 will buy many lotto tickets.

New Fellows

Congratulations to the three new Fellows of IPENZ – Andy Olsen, Tim Sinclair and Dick Beetham. It's pleasing to see the years of dedicated service to the Geotechnical Society in New Zealand has been publicly acknowledged. This is also an opportunity to broadcast to members for nominations for more F.IPENZ, in particular those who have supported the geotechnical community and society. For that matter, why not make some suggestions for Life Members of the Society.

Web Sites

Two issues here. We have received some flak recently for the state and access to the Society web site. Deservedly so too. We now have freer access to make regular updates and add links to sites of geotechnical interest. If you have any suggestions please forward then to the Management Secretary or the Editor (refer Management Committee Contacts list toward the end of this issue).

Also the ISSMGE intends to replace their somewhat meagre newsletter with a web site sometime early in 1999. "So if you want come and get. Personally I have a problem with this attitude and suggest you join me in writing a note to our Vice Presidential Representative, Prof. Mark Randolph (**randolph@civil.uwa.edu.au**, tel. (618-9380 3075) or fax (618-9380 1044). to at least get continuation of the newsletter in parallel with the web site until at least 90% of all members have email.

That's all for now. Wishing you all a good Christmas read and a relaxing holiday.

Steve Crawford
EDITOR

SETTLEMENT PREDICTION COMPETITION (\$100 Prize)

NZ Geomechanics News is calling for registrations of interest for predictions of settlement for a site which is to be preloaded. The site is underlain by volcanic deposits.

The following data will be supplied to registrants:

- **Borehole Data**
- **CPT Data**
- **Pre-Load Dimensions**
- **Time Frame for Preload**
- **Lab Test Data**
  - **Consolidation Test Data**
  - **Triaxial Test Data**
  - **Atterberg Test Data**
  - **Foundation Soil Density Test Data**
  - **Soil Moisture Contents**

Predictions will comprise magnitude of maximum settlement at the end of the pre-load time frame and must be supported by brief calculations. Guesses will not be accepted. Entries are free, but restricted to individual Society members only. Entries will be judged by a panel of three and their decision will be final.

Please forward your registration of interest to

The Editor
NZ Geomechanics News
P O Box 5271
Auckland

MONITOR THE MONITOR!

Experiences in the monitoring of a number of civil engineering projects prompts me to make some comment on the pitfalls in this tedious, and sometimes abused, process. Why get involved with it at all?

Well, we may be just building up new information for future use. More often it is because there is a hazard and we need to confirm assumptions made in a design or to be warned of progress towards a malfunction which can be avoided.

At all events, what is required is systematic recording, interpretation and reporting of physical data over a period of time. A time when nothing dramatic may be happening and all concerned become bored and off-load as much of the chore as possible onto machines (e.g.: computers). Mechanical components can be directly linked to computers deputised (by humans) to exercise discretion and control, but structures, especially earth structures, need continuous human judgement and intervention.

The experiences prompting this discourse have been of lack of human attention and judgement.

In the case of two hydro-electric schemes where manual field observations had been passed through a computerised recording system, readings which should have caused some excitement, were un-noticed until a much later periodic review occurred. In one case benign field readings had been mis-entered into the system, a fact only cleared up by reference to the field officer's personal note book – which he happened to have kept! In other cases readings were inexplicably abnormal suggesting either mis-function of the detecting mechanism or a reduction in stability of the structure.

An interpretation of a series of groundwater observations in a slope stability investigation, in another case, relied upon a computer "massaged" plot which had embalmed a fundamentally impossible reading which nobody noticed until an independent review many months later. Nobody was monitoring the machine!

Assuming that an observation and reporting system has been carefully designed at the outset, the lessons to be learned from the above cases include: -

- In the early days of the monitoring, levels of normality, warning and alarm should be established by prompt and close attention to the field records.

- Those established levels should be continually under the eyes of the person making the field observations who will first notice an abnormality.

- After a sequence of un-alarming results has been recorded, establish a routine of regular periodic scanning of and comment on the results by a second, more senior person.

- At longer intervals (perhaps annually) review the observation schedule to establish new stations where necessary and eliminate those no longer significant so that the tedious processes are reduced to the essential minimum.

- Provide for regular, independent reviews (such as SEED reviews) by an unjaded eye.

In short, pay attention, do it properly or don't do it at all

Monitor the Monitor.

D K Taylor
Member

TAURANGA TLA EXPERIENCE

The following perspective is offered in reply to the well presented paper called "The Design of Permanent Slopes in New Zealand" and is a personal view rather than the operational policy of the Local Authority I am fortunate enough to work for. My thoughts will however draw on my experiences within the Tauranga District.
Putting aside the issues of Legislative requirements incumbent on both the Practitioner and the Local Authority, it is important that in assessing a site for its long-term stability/ permanence the practitioner needs, in my view, to ultimately ask the questions:

1. What is the expectation of the intended end user of the land?
2. What is the expectation of the developer of that land?

In a good majority of cases the answer to the first question is an expectation that any given slope/building platform is safe (in the long-term), will require no further investigation after purchase and is able to be developed in a non-specific design manner.

The answer to the second question is to maximise return while minimising development expenditure. In maximising return developers often recognise the need to adequately investigate the subject site and thus through prudent investigation and analysis remove or reduce any impediments that may dissuade a potential purchaser from buying his developed property.

Couple the answers of these questions with Ethic 1 of the IPENZ Code of Ethics and the need for a prescriptive style design code for slopes has diminished as the requirements of the investigation and design have ultimately been answered.

Once the above questions have been answered it is then up to the practitioner to resist the economic pressure that is likely to be exerted on him by the developer and so doing complete the investigations and analysis in a professional manner.

All practitioners will be aware that the greatest asset a good soils practitioner can employ in his daily activities is the intuition that he/she has for the soils structure and the topographical landform in his/her area. This intuition coupled with a careful research of the practitioners own records and that of the Local Authority and a considered site investigation will provide the soils practitioner with that best yet intangible indicator of permanence which is the level of comfort he/she feels after the site investigation and analysis process is complete.

Soils, apart from any other engineering discipline, is one that relies as heavily on judgement as it does on analytical computation and at the end of the day one can never say with 100% certainty that one has fully solved the soils problem as always with the soils problem is still relatively "unseen."

Therefore, within any guideline/code the emphasis must also go on the development of the practitioner's judgement as much as on what assessments should address as part of the investigative process.

With this regard Peer Reviewing of one's work is of immense benefit as it not only reinforces to the practitioner that he/she is maturing and developing good investigative and analytical skills but also that important ingredient of judgement.

The bonus, in my opinion, of Peer Review is the dissemination of information from a highly experienced practitioner, to perhaps a less experienced practitioner, of those experiences that have been learnt over time and which are not in text books, guidelines or will never be written down.

This generally ensures the less experienced usually become more proficient quickly and have adopted and developed better work ethics, attitudes and methods than may otherwise be the case.

This also reduces the chance of mistakes and therefore reduces the risk to the younger practitioner, the practitioner's employer and the community as a whole.

It is also the role of the Local Authority to ensure that it is aware of the level of risk involved with the various geotechnical issues contained within its boundaries and ensure that the right practitioner addresses any given subject site.

A combination of:

- Local Authority and practitioner expertise,
- a register, compiled and maintained by the Local Authority, of capable geotechnical specialists who exercise judgement and skill &
- the judicious use of Peer Review where necessary

will ensure that risk to the ultimate end user, the community at large, is significantly reduced or minimised. As a consequence, so also will the risk to the geotechnical practitioner and also the Local Authority.

Paul Baunton
Development Engineer
Tauranga District Council

MORE FROM TAURANGA EX HONG KONG

I wish to comment (on 'The Design of Permanent Slopes', *Geomechanics News* No. 55) as follows:

1. The Hong Kong (Slope) Manual Tables 5.1. to 5.4 have been superseded recently. The FoS's have not changed nor the principles but the word "risk" has been clarified as close scrutiny revealed there was confusion particularly in the eye of the public. I believe your Table 1 is (proceeding) along the right track but the text should be expanded to explain exactly what you mean by risk, hazard, consequence etc. I believe the definitions should be in line with internationally accepted definitions for risk assessments (you may have done this already but your reference to the now out of date Hong Kong work suggests not). For example your Table 2, what do you mean by risk, ie. Probability of failure, or the likelihood that a failure will kill or injure if a failure does occur?

2. I believe that long term monitoring of the ground water levels in slopes is not carried out much in NZ. A recent study in the (Tauranga) Minden for the Western Bay District Council is an exception. Hence there is significant uncertainty about the maximum design groundwater conditions a slope could expect. This issue could be further highlighted in Section 6.7. The use of Halcrow buckets, as a routine measure would help in this regard.

3. There is no discussion on slope displacements under earthquake loads in Section 6.8. I believe this approach is a legitimate way at addressing the implications of earthquakes on slopes rather than the FoS approach. The reality is that applying a factor of say 0.3 g in a stability analysis is a gross oversimplification and Bolton-Seed's Terzaghi paper and later research highlights this point. Factors such as wave amplification, type, direction, topography etc have to be considered - as do dynamic shear strength and pore pressure response. Even then I believe it is beyond the state-of-the-art. Probability matrices have been developed on expected slope displacements in California and this may be a better way forward on this issue.

4. My personal view is that a realistic assessment of the stability of natural slopes, that are not failing, is beyond the state-of-the-art. I think you make reference to that point somewhere in your paper. Therefore, the use of a number say FoS = 1.5 is inherently misleading. What we are really saying is that a slope has a lower probability of failure than say a steeper adjacent slope that has a calculated FoS = 1.2. This is because of the huge number of uncertainties associated with natural ground, soil properties, geology, man-made influences, weathering, relict joints, groundwater flow paths etc. My preference would be to stress this point some more as it is a better position to defend from a liability perspective, ie every slope has a probability of failing. The fact that it had a calculated FoS of say 1.5 does not remove this possibility (I think this is the Australian position). I therefore strongly support the empirical approach stated in Section 6.9 - where there is no evidence of large scale instability and the most likely failure mechanism is a slabbing off of material from the face of the slope resulting from water infiltration and weathering.

John Scott
Project Manager
Tauranga District Council

MARGIN FOR IGNORANCE

It is good to see another large step towards resolution of the profession's position in *Geomechanics News* No.55 ('The Design of Permanent Slopes'). I enclose my comments upon some of the matters, for you to publish. Referring to my comments in *News* No.48, 1995, I can see very little that I would change since then.

There has been a pre-occupation with the idea of a commercial Factor of Safety (Margin for Ignorance?) which can be applied only if there has been a quantitative analysis of the slope. Such an analysis is a useless myth if it is not based upon defined soil properties, site geometry and groundwater pressures.

In section 3.5 the statement is made that "often assumptions have to be made". Why? The only reason that I can think of is that it is "too expensive" to gather the necessary data. That is a decision for the client to make, not the engineer or the geologist.

Geometry and groundwater are specific to the site. Soil properties might be "borrowed" from other sites provided that index tests and informed

geological examination demonstrates identical soils at the two sites, and the source of the quoted soil properties is identified. Otherwise the soil properties are not "defined" (8.0- point 8, 4.0(c)).

If "full saturation" (6.7) means groundwater at the ground surface, then it is as well to remember that static water height is not the same thing as piezometric pressure, which may be artesian.

A distinction that could be made is that between an "analysis" which includes a quantitative analysis of the forces involved and an "assessment" which does not, but depends upon qualitative judgements.

Table 1
Bearing the foregoing in mind, I am of the opinion that Table 1 is appropriately general enough to cover most cases and should not be more detailed but there could be some amendments as follows:

- Evidence/Type of Instability:

  -Very High, and High Risks
  "within site or extending beyond site

- *Minimum* Extent of Investigation Req'd:
  -Very High, High Risks
  require a quantitative analysis.
  -Medium, Low, Very Low Risks
  require a qualitative assessment

Table 2
In leaping to a Peer Review are we ducking the question of who should be making the assessment or analysis which is to be reviewed?

I can't accept the idea that TLA's should carry no risk. The Resource Management Act, quite reasonably, make them responsible for reaching a decision on land stability. (To protect the public from the actions of vested interest). If the TLA's can't accept liability then why should anyone else, with lesser financial resources and insurance cover?

Certainly, the assessment may be delegated but the TLA cannot escape responsibility for evaluating the capability of the person making it. In this context, the review is not by a "Peer" but by an independent person (possibly with greater or more specialised experience) advising the TLA.

D K Taylor
(Personal comments)

A SCOTSMAN'S VIEWPOINT

I would like to congratulate the authors on the preparation of a paper ('The Design of Permanent Slopes') that contained such a large amount of interesting background factual information on slope stability problems and how they are handled both here and overseas. I believe that most of the recommendations made at the end of the paper will receive wide spread support from the profession but I couldn't help thinking that most readers of the article probably had some insight, understanding or belief in such recommendations prior to it's publication.

I was impressed with the manner in which the fundamental issues of stability assessments were identified in four simple questions: -

1. When is an assessment necessary?

2. Who is the appropriate person to carry out the assessment?

3. What should the assessment address?

4. Who should review and approve the assessment?

The authors only lay claim to resolving, or least addressing, the third question within the context of the paper. I think they do themselves a disservice and the answers are there for the reader who is prepared to think about these issues and interpret the findings of the authors' research.

For example, in answer to question 1, it is my opinion that when considering the design of a permanent slope for a residential development an assessment of the stability issues will be deemed necessary when the District Plan identifies the location as a Medium to Very High risk area.

The fact that Slope Hazard Maps for incorporation into District Plans at practicable scales have not yet been prepared is irrelevant. The legislation is in place that demands their production and the technology is available to prepare them. Provided the Hazard Maps are only ever referred to as a preliminary guide on slope risk and are simply an indicator of the need for a more detailed assessment then the questions on liability, economics, politics and cost are negated.

The answer to question 2 (and question 4 for that matter) is equally simple and contained within the responses to the NZGS Questionnaire *(refer NZ Geomechanics News Dec. 1994)*. Assessments have to be at least signed off by a Registered Engineer who is a recognised Geo-specialist. I would be a very strong supporter of any proposal to initiate a directory of geotechnical specialists along the lines of the BGS that would include a simple entry of individual members of the NZ Geotechnical Society, with their qualifications and employment history.

In regard to the role of the Peer Reviewer I would take issue with the authors over their statement that a peer review should not be regarded as a second opinion in disguise. The difference is rarely understood by clients who issue instructions to their Peer Reviewers. Their intent is often to receive a second opinion on an issue that has a critical impact on their property or finances. Provided the peer review role is performed professionally and openly then the trend towards a greater use of Peer Reviewers should be welcomed as recognition of the importance of our work.

Recognising the need for brevity the final point I would like to highlight (and I'll try and keep it short) is in regard to factors of safety. The authors recommend a minimum Fs = 1.5 for design conditions and 1.2 for earthquake conditions. I think this needs to be reconsidered since it is, in my opinion, not consistent with the philosophy advocated in the rest of the paper.

To undertake any numerical stability analysis demands that you make some assumptions about the slope and it's mechanical behaviour. In order to develop a moderately accurate analytical model demands a detailed knowledge of topography, geology, groundwater and associated characteristic properties coupled with some evidence that the mechanism and methodology adopted in the analysis bear some resemblance to probable or realistic events. Without such detailed knowledge it is quite feasible to analyse any given slope and, by adopting different combinations of possible parameters or loading conditions, arrive at any safety factor within an extremely wide range.

Considering Table 1 in the paper, the only sites which would be subjected to an investigation which was sufficiently rigorous to undertake a sensible numerical analysis would be those identified as Very High Risk. However, the described instability events for those sites classified as High or Medium Risk by definition imply at, or near, limit equilibrium conditions (Fs = 1.0). If the consenting authorities demand that such sites demonstrate a numerical Fs >1.5 then a significant increase in the extent of investigations must follow.

Since it may not always be practicable or possible to undertake the necessary detailed investigations I find it difficult to advocate the inclusion of a prescriptive safety factor if the consequence is an analysis which I cannot justify.

Also, congratulations also on the publication of an excellent checklist for stability assessments (very similar to the one I have developed here which suggests we have been plagiarising the same sources!!) and the risk classifications, which I think is open to some debate, but is a great start.
I would conclude by applauding the authors in their attempts to tackle such a broad and complex problem. In my opinion it is an excellent start to a process that will probably never reach a conclusion but will in time lead to a more consistent and professional engineering practice.

J Grant Murray
Geotechnical Engineering Manager
Kingston Morrison Ltd

FORMALISE INTUITIVE RISK-BASED APPROACH

The ('Design of Permanent Slopes') report is a comprehensive review of current NZ practice regarding stability assessments and the introduction of a risk based approach to stability is particularly welcome. The risk-based approach is used intuitively by many practitioners and the suggested risk classification will help formalise it. I note that a review of international practice for risk based approach to stability (Quantitative Risk Assessment) and its application in NZ practice is being carried out by Riddolls & Grocott Ltd and will be soon published in a BRANZ report.
Comments on particular aspects of the reports are:

Table 1 provides a useful draft framework for classifying risk. However, it needs to be reviewed for compatibility with the BRANZ report. The extent of investigations needs to be clearly distinguished from those required to assess foundations.

Table 2 usefully categorises peer review requirements. However, the scale of project does not necessarily relate to risk classification and could be misleading, and it would be better to omit it from the table.

The checklist for Stability Assessment is comprehensive but needs to include Quantitative Risk Assessment. Some explanatory notes on the anticipated use of the checklist by TLA's and practitioners is needed. We strongly endorse the recommendation that an engineering geological assessment should be made as a first step in a stability assessment.

In summary, the report provides a most useful base for development of a national set of stability guidelines. The comments and input of practitioners into the guidelines is essential. The views in this letter reflect those of Worley.

G B Farquhar
Geotechnical Engineering Manager
Worley Consultants Limited

QUANTIFY RISK ASSESSMENT

Thanks for your request for comments on the proposed stability guidelines. Due to the limited time available, we offer the following comments in respect of only one of the aspects you have solicited opinion on, namely that of risk classification and risk acceptance criteria with which we have some understanding.

As background information, we have recently completed a study on landslide quantitative risk assessment (QRA). This is still very much an emerging concept overseas, and has received very little attention in New Zealand to date. Our study has focused on the concepts and procedures of QRA, as well as a review of risk guidelines and risk acceptance criteria.

The trend overseas is very much towards couching landslide risk in quantitative rather than qualitative terms. The limitations of QRA are well recognised, principally that not all slopes can be evaluated quantitatively due to lack or insufficient data. However, some of these limitations can be partly overcome through the use of Monte Carlo simulation and other techniques to enable geologic uncertainty to be modelled in slope stability analysis.

With these comments as background, we would like to suggest that development of qualitative risk classifications such as is being proposed by way of Table 1 in EQC Project report 95/183 ('The Design of Permanent Slopes') is perhaps not reflecting the trends which are being established overseas. This is not to say that such qualitative risk classifications are not useful, but that perhaps more useful risk classifications are available when risk is expressed quantitatively. We believe the stability guidelines should include a summary of QRA concepts, procedures and risk acceptance criteria.

Guy Grocott
Riddolls & Grocott Ltd

RISK CLASSIFICATION USEFUL

The ('Design of Permanent Slopes') paper is timely and important for (the) reason (that) it sets out useful and necessary objective suggestions for a difficult area of professional practice. The suggested table of Risk Classification is considered particularly useful as a common starting point in evaluations and as a catalyst for development in this area.

Similarly the table on levels of peer review is a good starting point for what probably will be a contentious area. There is a need to define what is meant by peer review that (perhaps) it is a review of methodology and not a blow by blow design check.

Marlborough District Council is concerned about the ability to be assured of the quality of inputs to the material that is presented for its acceptance. One area of concern is the resourcing of practitioners to, in fact, "... investigate to the extent necessary."

Any attempts to improve the standards of investigation and reporting must be applauded and supported particularly in the development of guidelines for investigation but also in the documentation of results for the end user.

Neil Morris
Marlborough District Council

BASE RISK ON VALUE

Generally it is easier to assess a likely range of soil strength parameters and to postulate a mechanism of failure than it is to predict when or how often a particular failure will occur. Sensitivity analyses can be carried out using the range of assumed parameters and conditions (which may range from moderately conservative to worst credible) to provide additional confidence as to the likely mechanism and the groundwater regime that may have been present at the time of failure. The calculated factor of safety determined by typical slope stability analyses does not however provide information that is readily understood by most clients. The use of an arbitrary factor of safety is not linked to the likelihood of occurrence nor the costs of repair nor the potential for loss of life. This approach does not generate data that is readily understood by lay people. Similarly variations in approach, parameters and analytical techniques which give rise to different FoS's tend to cause delays in approval by regulatory authorities as to whether a particular development can proceed. This type of analysis does not provide information in a form that will allow clients to make fully informed decisions. The use of risk assessment techniques can provide additional advice to land owners, potential landowners and regulatory authorities. This approach has merit in overcoming misunderstandings surrounding the selection of a "Factor of Safety" other than 1 which does not provide anyone with a clear idea as to whether the risk of slope failure is acceptable. From our experience, clients typically do not want to accept any risk until such an approach is quantified in terms of the resultant direct cost to themselves.

Identifying the consequences of failure is potentially a reasonably straightforward process. Direct analysis to reliably identify when a particular failure sequence will occur and or the size of a particular event is beyond the current ability of analytical techniques used by practitioners. It is possible to identify the likely ground water conditions present at the time of failure. Quantifying the rainfall required to create such groundwater conditions is however generally not possible although back analysis of slope movement following a known rainfall event can give some indication of groundwater response to rainfall. The actual rainfall at a particular site and the groundwater levels prior to a particular event are rarely known. Determining the return period for that rainfall event in that particular location is likely to be affected by the limited reliable rainfall data available in the area. Changes to the roles of public organisations previously responsible for nation wide collection of meteorological data also mean that data that was previously readily available is no longer collected. The rainfall data that can be obtained is not usually immediately available nor is it published. The inability of practitioners to obtain reliable information on or determine the return period of a particular rainfall event or a sequence or pattern of rainfall events will limit our ability to accurately assess the frequency of a particular event or weather pattern but not the potential effect of that weather pattern. The sensitivity of a client to a particular frequency of event can also be addressed. For general civil engineering design this is similar to considering flooding of a basement once every 5, 10, 25 or 50 years and the costs of addressing each assumed frequency to mitigate the effects. It is more likely that for slope movement the frequency will be less definite, say once or possibly twice in the life of a particular structure. Eg due to creep a retaining wall may need to be replaced or rebuilt once within the lifetime of the materials used to construct that wall. The consequences for a particular client could therefore be assessed. Whether a particular risk is considered by a client to be reasonable tends to be heavily influenced by the assessment of the return period. It could be argued that the long return period of large seismic events significantly influences the development choices made by most Wellingtonians. Few if any will have consciously assessed the potential direct cost to themselves.

The "Table 1: Risk Classification...." proposed by Crawford and Millar does not address this issue. In addition the implications for development as outlined are very limited and could well lead to an increase in the imposition of a greater number of Section 36(2)'s than is perhaps warranted by the site conditions. The Table as outlined does not include for comment as to the length of time over which failure may occur and therefore the opportunity to mitigate damage and prevent loss of life. We assume it is intended by the authors that such considerations be taken into account when considering the consequences of instability although users would not be directed to consider such factors. Events that have a probability of 1 (eg creep), that is they are part of a relatively slow but on-going process likely to be affecting an area much larger than that considered for a single lot development, are described in Table 1 as posing the same risk as those triggered by individual extreme/low frequency rainfall events. Similarly evidence of past instability (which could be of geological origin or historic and arising from conditions no longer influencing the site) automatically places a site in the high risk category. Given New Zealand's geological youth large parts of the country could be placed in this category. While the presence of such features needs to be identified an assessment as to whether movement has occurred and is likely to occur within the next 50 or 100 years is necessary before a broad classification of high risk can be imposed. Similarly this category needs to be linked to something that is readily understood. eg the likely return on a bet on the horses. The dollar input is understood and the potential low return/loss is understood. Few stop to consider the development of their single largest asset in such terms. Basing risk assessments on current land value and assuming total loss and or a similar approach to dwellings using typical values for the area under consideration may make the assessment more accessible to clients. The classification of low or high risk can then be made by the individual. A multi-millionaire may feel that for 10 to 15 years he/she is happy to accept that he may incur a loss of $200 to 300,000 in a 1 in 100 year event and possibly take out insurance against such an event as a moderate risk but feel that damage in the $50 to $100,000 range is acceptable and not worth insuring against although other individuals may consider such a risk to be extreme.

If we are to move to a risk based assessment the method needs to use comparative terms that can be understood both in terms of the frequency of occurrence and the cost of repair based on values at the time of assessment or the extent of anticipated damage categorised. Both could then be reassessed on an annual basis if necessary and used by insurers.

Malcolm Stapleton
Geotechnical Engineering Manager
Babbage Consultants Ltd

CONSEQUENCES OF FAILURE

This letter responds to the paper " The Design of Permanent Slopes for Residential Building Developments" by S.A. Crawford & P.J. Millar (EQC Ref. 95/183). I agree that there is a need for national guidelines and I generally concur with the suggestions put forward in the paper. Some specific suggestions are:

- Delete "consequences of instability" from Table 1 as it is not directly related to risk of instability. This would not prevent risk to life and property being taken account of in the stability assessment/extent of investigation undertaken.

- Excavation procedures and requirements for support of slopes during construction should

be included with Building Consent applications. This aspect could perhaps be included in the checklist for stability assessments.

Some general comments on the review process from a regulatory perspective are:

- A report by a suitably qualified professional should accompany any proposal for development of a site with medium or greater risk of instability. This person should always review the "final design" site works, preferably before a consent application is lodged. In some cases this would significantly reduce consent processing time and costs.
- A set of guidelines for peer review would be useful. Note that the peer review needs to go beyond the stability assessment and consider the relevant engineering works to be undertaken on the site.

Bill Vautier, M.IPENZ

(The above comments are those of the undersigned and do not necessarily represent the position of Auckland City Environments).

CLIFF TOPS & 50-YEAR LIFE

The utilisation of the guideline approach to slope stability is one that is preferred by (North Shore City) Council rather than a set of prescriptive guidelines. The current trend in building is certainly along these lines, with widespread use of the alternative solution guideline recommended by the New Zealand Building Code and supporting New Zealand standard codes.

From our experience, while the Court system places some reliance upon the expert witness, some value is found in reaching an understanding of the level of care and skill applied in establishing performance out of the solutions developed by the practitioner, over the intended

service life. For most new buildings, this is no less than 50 years.
By way of example, the Council has adopted guidelines for coastal resource consent applications in relation to various planning activities and geotechnical assessments, which must aim to meet the criteria set (copy follows).

A typical 50 year regression rate becomes a significant determinant when consideration is being given to ensuring that the intended development does nothing to increase the natural rate of erosion or create significant risk of accelerated erosion and/or instability of the site or adjoining land.

It follows that nothing is done to establish a building likely to be affected by that circumstance. Its performance is more assured and the insurability of the building is then more likely to be assured.

With regard to Table 1 (of 'The Design of Permanent Slopes' paper), I have only one concern. The table is not addressing performance factors out of the measures developed by the practitioner, commensurate with the risk levels listed. If the consequences of risk are high, so too should a guideline level of performance be given for the purpose of developing evidence to give some confidence to the practitioner(s), the Council(s), the owner(s) and the Courts. An appropriate level of post-construction monitoring / maintenance could be referred to in order to qualify your recommendation in item 8.0.

With regard to Table 2, I have no concern.

Thank you for the opportunity to comment.

Brian Gunson
Team Leader - Compliance
North Shore City Council

(A Copy of the NSCC Coastal Consent Criteria referred to above follows this letter)

Amended 15 October 1998 all previous copies should be discarded.

GUIDELINES FOR COASTAL RESOURCE CONSENT APPLICATIONS

The Coastal Conservation Area is defined by a line on the District Plan Maps and generally includes the first line of residential sites back from the coast and includes coastal and esplanade reserves.

<u>In the Coastal Conservation Area activities requiring a resource consent are as follows:</u>

1. **Permitted activity**

- Site works (any disturbance of the existing ground surface of any site, including the excavation or depositing of soil, spoil or other material) which expose up to 300m^2 surface area of bare earth, except where the works are located less than 5 metres from any cliff face. *Rule 9.4.1.1*
- Excavations no greater than 1.5m in depth. *Rule 9.4.1.1*
- Retaining walls which have the effect of raising the natural ground level by up to 0.5m located on any boundary or in any yard. *Rule 9.4.1.1*
- Site works authorised by a subdivision consent *Rule 9.4.1.1*

2. **Controlled activity**

- Buildings (as defined in Section 21 of the Proposed Plan, and includes decks, terraces and steps greater than 0.5m in height, retaining walls greater than 1.5m in height, fences and boundary walls greater than 2m in height, and swimming pools), and structures such as stormwater outlets, ,jetties, slipways, pumping stations, and raised manholes, all which are outside any yard requirements. ***Rule 8.4.1.1***
- Retaining walls which have the effect of raising the natural ground level by more than 0.5m and no more than 1.5m located on any boundary or in any yard. *Rule 9.4.1.2*

3. **Limited discretionary activity**

- Site works (excavation or deposition0 which expose greater than 300m2 of bare earth which are not discretionary activities in accordance with Rule 9.4.1.4 *Rule 9.4.1.3*
- Site works that encroach on a site of Geological Significance identified in Appendix 8B of the Proposed District Plan and shown on the District Plan Maps. *Rule 8.4.5.1*
- Excavation (not on a boundary or in any yard) greater than 1.5m in depth. *Rule 9.4.1.3*
- Retaining walls which have the effect of raising the natural ground level by more than 1.5m. *Rule 9.4.1.3*

4. **Discretionary activity**

- Site works (excavation or deposition of any material) located less than 5m from any cliff face *Rules 8.4.2.1 and 9.4.1.4*
- Site works on land comprising a Site of Special Wildlife Interest identified in Appendix 8A of the Proposed District Plan and shown on the District Plan Maps *Rule 9.4.1.4*
- Site works on land comprising a significant landscape feature as identified in the District Plan maps. *Rule 9.4.1.4*
- Site works involving retaining walls exceeding 1.5m in height *Rule 9.4.1.3*
- Site works where the subject land has an average gradient steeper than 1:4 *Rule 9.4.1.4*
- Any pruning, removal or alteration to soil around the roots of pohutukawa trees of 3m or more in height, or any native vegetation (of any size) in the foreshore yard. *Rule 8.4.6 1 (a) (i)*

The following guidelines advocate good land management practices and have been developed with the intention of protecting the coastline, particularly cliffs, and coastal properties from adverse effects of activity and development that may accelerate erosion or detract from the natural character of the coastal environment.

GUIDELINES

1 A geotechnical assessment will be required for any resource consent in the Coastal Conservation Area according to Rule 9.6.5 of the Proposed District Plan with the éxception of resource consents for tree work, and a geotechnical investigation and report will be required for any structure included in 1(a), or activity included in 3, or 4(a). Any geotechnical report must demonstrate that:

- The development does not detrimentally affect the appearance and natural qualities of any cliff or steep coastal terrain.
- Development does not increase the natural rate of erosion or create significant risk of accelerated erosion and/or instability of the site or adjoining land.
- The stability of any building or development is adequate to ensure that additional engineering works are unlikely within the intended life applied for in the building consent.

2 Any existing vegetation, particularly pohutukawa trees, should be retained along the cliff top edges and the cliff face wherever possible. The layered vegetation from ground cover through shrub layers to the tree canopy is important in retaining a stable soil-moisture regime and sheltering the friable soil from wind and rain. It is the alternating wetting and drying from rain, wave splash, wind and sun that accelerates the frittering and weathering of exposed cliff surfaces. Once vegetation is removed from cliff sites it is extremely difficult to re-establish because of the harsh growing conditions. It is the native coastal species that are pre-adapted to these conditions that are the easiest to work with for any landscaping or re-instatement required. These include species such as:

Ground covers

Coastal spleenwort (fern)	*Asplenium flaccidum haurakiensis*
Bush flax	*Astelia banksii*
Coastal tussock (grass)	*Chionocloa bromoides*
Coastal toetoe (grass)	*Cortaderia splendens*
Native toetoe (grass)	*Cortaderia toetoe*
Mercury Bay weed	*Dichondra repens*
Fern	*Doodia media*
Pohuehue (vine)	*Muehlenbeckia complexa*
Mountain flax	*Phormium cookianum*
Flax	*Phormium tenax*
Poor Knight lily	*Xeronema callistemon*

Native Shrubs & trees

Coastal daisy bush	*Brachyglottis greyii*
	Cassinia fulvida
Coprosma species	*Coprosma prostrata*
Karamu	*Coprosma lucida*
Taupata	*Coprosma repens*
	Coprosma macrocarpa
Cabbage tree	*Cordyline australis*
Three Kings cabbage	*Cordyline kasper*
Akeake	*Dodonaea viscosa*
Purple akeake	*Dodonaea viscosa purpurea*
Puka	*Griselinia lucida*
Broadleaf	*Griselinia littoralis*
Hebe species	*Hebe speciosa*
	Hebe macrocarpa
	Hebe parviflora
	Hebe obtusata
Kanuka	*Kunzea ericoides*
Manuka	*Leptospermum scoparium*
Mahoe	*Melicytus ramiflorus*
Wharangi	*Melicytus ternata*
	Melicytus novae-zelandiae
Pohutukawa	*Metrosideros excelsa*
Ngaio	*Myoporum laetum*
Karo	*Pittosporum crassifolium*
Golden tainui	*Pomaderris kumeraho*
Fivefinger	*Pseudopanax laetus*
Houpara	*Pseudopanax lessonii*

3. Stormwater disposal.

- Where possible: stormwater run-off must be directed back towards the road to join the piped drainage system.

- Where this is not possible:

  a) Where hard surface areas, including roof areas, exceed 250m^2 and are directed towards the cliff the stormwater run-off must be collected, either in a community or individual system, and discharged through a thrust-bored drainage system emptying to MHWS level and fitted with an energy dissipation device to a velocity less than 1m/sec. to prevent scour.

b) Where hard surface areas, including roof areas, total less than 250m^2 stormwater must be directed to a safe discharge point down as identified within a geotechnical report.

The safe disposal of stormwater run-off from cliff-top properties is a highly technical issue for which suitable advice must be sought. Most mechanisms of cliff retreat are accelerated by saturation.

4. A resource consent is required for any alteration to natural ground level along the cliff top and within 5m of the cliff face. The reason is that the addition of material, such as compost, grass clippings or garden rubbish, adds extra weight to the cliff top and can increase the risk of overburden failure, causing slippage.

5. The visual effects of the development (building, deck, retaining wall, steps etc.) should be assessed from adjacent public areas - beach, reserve and water's edge, as well as from surrounding neighbours. The development must be assessed with regard to the effects on the natural character of the coast, and whether the natural character of the site and surrounds can absorb further built elements.

   Effects to be avoided include:

   - the appearance of the development should not break the continuity of the coastal landscape by striking a strong contrast rather than blending in with surrounding features.

   - the appearance of the development should not interrupt nor intrude upon the legibility of the immediate coastal landscape. The development should not disrupt the clear line of cliff, nor a significant stand of vegetation, for instance.

   - the development should not adversely affect a significant landmark, cultural feature or significant stand of vegetation.

   - the development should not visually dominate the coastal landscape.

Management Committee

The new committee, which took office in July, comprises:

Geoffrey Farquhar	Chairperson
Debbie Fellows	Management Secretary
Grant Murray	Treasurer
Ian McPherson	
Jaimie Bevin	
Stephen Crawford	Co-opted, Editor - Geomechanics News, Chairman - Stability Guidelines Subcommittee
Guy Grocott	Co-opted, Symposium Organiser
Warwick Prebble	Ex officio, Australasian Vice-President IAEG
Colin Newton	Ex officio, Immediate Past Chairman

In particular I welcome Debbie Fellows who was selected by the new committee to fill the role of Management Secretary. I wish to thank the following members, who stepped down from the previous committee, for their service to the Society.

Chris Freer	Alexei Murashev	James Burr
P Brabhaharan	Neil Crampton	Mick Pender

Warwick Prebble

The Society expresses its gratitude to Warwick who at the end of this year steps down from his 4 year role as Australasian Vice-president of the International Society for Engineering Geology and the Environment. He has kept Australasia at the forefront of the affairs of the IAEG and we are fortunate in having another New Zealander, Bruce Riddolls, to take over from him.

Roading Geotechnics '98

The symposium held in July was successful in all respects - the number of sponsors, the number and quality of technical papers, the number of registrants and the quality of the field trip all exceeded expectations. Thanks to all who participated.

Geomechanics Lecture

Tim Sinclair of Tonkin & Taylor Ltd has been awarded the honour of presenting the 10th NZ Geomechanics Lecture. The lecture was established to honour individuals who have made a notable contribution to NZ Geotechnical Engineering. Tim will first present his lecture at the 8th Australia New Zealand Conference on Geomechanics to be held in Hobart in February 1999. Thereafter it will be presented in major centres in NZ.

Registration of Engineering Geologists

In past years it was taken that engineering geologists could not become registered engineers. The Society together with the universities is presently pursuing accreditation of current Engineering Geological degrees as a basis for corporate membership of IPENZ and registration. IPENZ is also currently pursuing the government for a new Engineers Registration Act. Meanwhile the society is aware of engineering geologists with more than 10 years experience, including engineering design and responsibility, who are applying for registration. We will keep members posted on the outcome of these matters.

After the branch and technical group forum held on 6th October, Warwick Bishop (IPENZ CEO issued a position paper on 21st October. A copy of this paper follows 'Chairman's Corner' for the benefit of Society members.

AGM

The last AGM was held in July at the symposium. We will revert to our normal time of February next year.

Education

A motion was passed at the last AGM for the society to subsidise educational activities that will benefit members. The committee is working on matters and the first event is scheduled to be a two day workshop on Quantitative Risk Assessment of slope stability. *(see notice later in this issue).*

We are also currently reviewing alternatives for a Pavement Design Workshop for members to become more familiar with the use of the AUSTROADS design manual.

Best wishes for Christmas and the New Year.

Geoffrey Farquhar
Chairman

PROPOSED NEW ENGINEERS REGISTRATION ACT

The Institution of Professional Engineers New Zealand (Inc) - IPENZ - is a **voluntary body** comprising some 8,000 professional engineers and technologists in both New Zealand and around the globe. We have been in existence for 84 years and speak for about 40% of the practicing engineers in the country.

IPENZ sets the **standards** for professional engineering in New Zealand. It does this through **accreditation** of undergraduate engineering degrees and **assessing practice competency** through **professional peer review**. By participating in **international professional engineering agreements** IPENZ ensures that New Zealand professional engineering qualifications are recognised throughout the world.

The Institution's **Code of Ethics** imposes standards of conduct covering professionalism and integrity, society and community wellbeing, sustainable management and promotion of engineering knowledge. The Code, which is backed up by a **disciplinary process,** gives assurance to the public and clients that members will act in a competent and professional manner.

IPENZ believes there should be a statutory backed register of professional engineers who have been assessed by the profession as competent to practice safely in New Zealand.

Purpose

The main purpose of the register is the **protection of public health, wealth and safety** by providing a mechanism to identify professional engineers who have been assessed by their peers as meeting a minimum standard of competence.

Complete reliance on market-driven rules to control the quality of professional engineering services is extremely difficult and potentially dangerous. By their very nature output controls are reactive, responding to either a foreseen risk or an actual failure. This may be theoretically sound, but does not take into account the human and social costs of a disaster or failure and nor is sufficient feedback given to prevent similar events re-occurring. A system that identifies a minimum standard of performance for professional engineers is a necessary complement to the 'output' controls if public health, wealth and safety, the environment and the interest of the consumer are to be protected.

When the majority of engineering services were provided by either a government department or agency these organisations had quite rigorous processes in place to ensure the major infrastructure investment decisions were technically sound. This role now falls back onto either elected or nominated Boards. The Commerce Act requires Directors of these Boards to make prudent decisions that would necessitate Boards obtaining expert advice. Most Boards of Directors are composed of people with backgrounds that give them comfort in requesting expert investment, tax, legal and marketing advice. Engineering, however, is an area of technology where expert advice often needs to be sought by Boards. A registration system provides a benchmark standard for individuals who are capable of providing this expert advice.

The privatisation process that is occurring in the development of New Zealand's infrastructure will increasingly provide overseas investment into this country. These investors often bring their own technological knowledge with them. It is important, therefore, to have some process that identifies local individuals who have met a minimum professional engineering standard that includes an understanding of the special requirements which exist within the New Zealand engineering working environment. Organisations which enter into contractual relationships with these overseas investors can then be assured that the technological support provided by the investors is sound particularly if the individuals providing this advice have met registration requirements in New Zealand.

Because engineering is increasingly becoming a globalised profession, NZ needs a registration system that is compatible with internationally accepted standards and registration/licensing systems. It is acknowledged that many other countries in which NZ engineers work have a statutory backed registration or licensing systems for professional engineers. IPENZ is presently involved in the development of two international registers of experienced professional engineering:

one, known as the Engineers Mobility Forum, is comprised of engineering institutions and professional engineering licensing bodies in USA, Canada, Hong Kong-China, Australia, South Africa, United Kingdom and Australia. There is another engineer register proposed within the APEC region.

It is anticipated that the proposed new register of professional engineers would form the basis of these international agreements. By having a statutory backed register for professional engineers this gives New Zealand some bargaining power to ensure other countries free up their licensing or registration system so that New Zealand registered engineers have the ability to gain entry onto the registers in other countries after meeting local requirement conditions. With our major consulting firms now souring significant volumes of their work internationally a New Zealand register of professional engineers, backed by statute, is required to ensure that New Zealand engineers working abroad are not disadvantaged.

Principles

- A suitable title (eg Chartered Engineer) needs to be protected in statute. This would be a quality brand, which enabled consumers to have sufficient confidence that those who had obtained C Eng status were able to adequately perform the task for which they have been employed.
- The minimum entry standard should be benchmarked to the M.IPENZ entry standard.
- There should be some system, which ensures engineers on the register are maintaining, or enhancing their competence and continuing to practise in their self-certified areas of competence.
- Those on the NZ register would need to have an understanding of NZ codes and practices relevant to their area of practice. It is implied therefore that those engineers educated and trained overseas would need a period of adaptation working in NZ as an engineer before gaining entry to the NZ register.
- Those on the register would be bound by a common code of ethics and disciplinary system. Statutory backing of the discipline system ensures that there is some "teeth" to the process thereby ensuring registered engineers are accountable for their performance to their peers.

Requirements

- Duplication of systems should be avoided. IPENZ should perform the administration and assessment tasks associated with the NZ register of professional engineers as opposed to parallel systems being developed.
- The new Act should be a light-handed regulation which is not prescriptive.
- Entry to the register should *not* be restricted to only M.IPENZ engineers. Those that have gained corporate membership status within other recognised engineering institutions deemed to be equivalent in standing to M.IPENZ would be eligible to gain entry to the NZ register. (ie Washington Accord countries - USA, Canada, UK, Ireland, South Africa, Australia, and Hong Kong-China) as would other engineers who had been assessed by IPENZ as having reached the M.IPENZ competency standard but choose not to belong to any recognised professional engineering institution.

Preferred Model for the new Act

IPENZ is of the view that an Act, similar to the Society of Chartered Accountants Act 1996, where the Act is wrapped around the professional body, is appropriate for the engineering profession. It is expected, however, that a special register of professional engineers would be set up for those that met the registration requirements. It is also expected that there would be a separate Board to oversee the registration system whose membership, although mainly professional engineers would also include consumer representation.

Warwick T Bishop
Chief Executive - IPENZ
BE (Elec.) F.IPENZ FIEAust FIEE

Highway construction/ Ground insulation

Styropor foam as a lightweight construction material for road base-courses

1 General

The main consideration when constructing roads on poor load-bearing subsoil is that every load deforms the soft soil layers; and the greater the load, the greater the deformation. This deformation process continues over years, depending on the thickness of the soil layers. The low shear resistance of poor loadbearing subsoils means that concentrated loads should be avoided as far as possible, otherwise these layers will give at the sides. Compensating for this form of subsidence by laying new material leads to further settlement due to the additional burden.

The conventional techniques of subsoil improvement by complete or partial replacement of the soil are often time consuming and therefore costly. By employing lightweight materials, the weight of the road embankment – and with it the load on the subsoil – is reduced considerably.

A largely subsidence-free method of construction is thus obtained when practically no additional loads are brought to bear – ie, by using extremely lightweight materials in the embankment such as blocks of Styropor foam (see figs. 1 and 2).

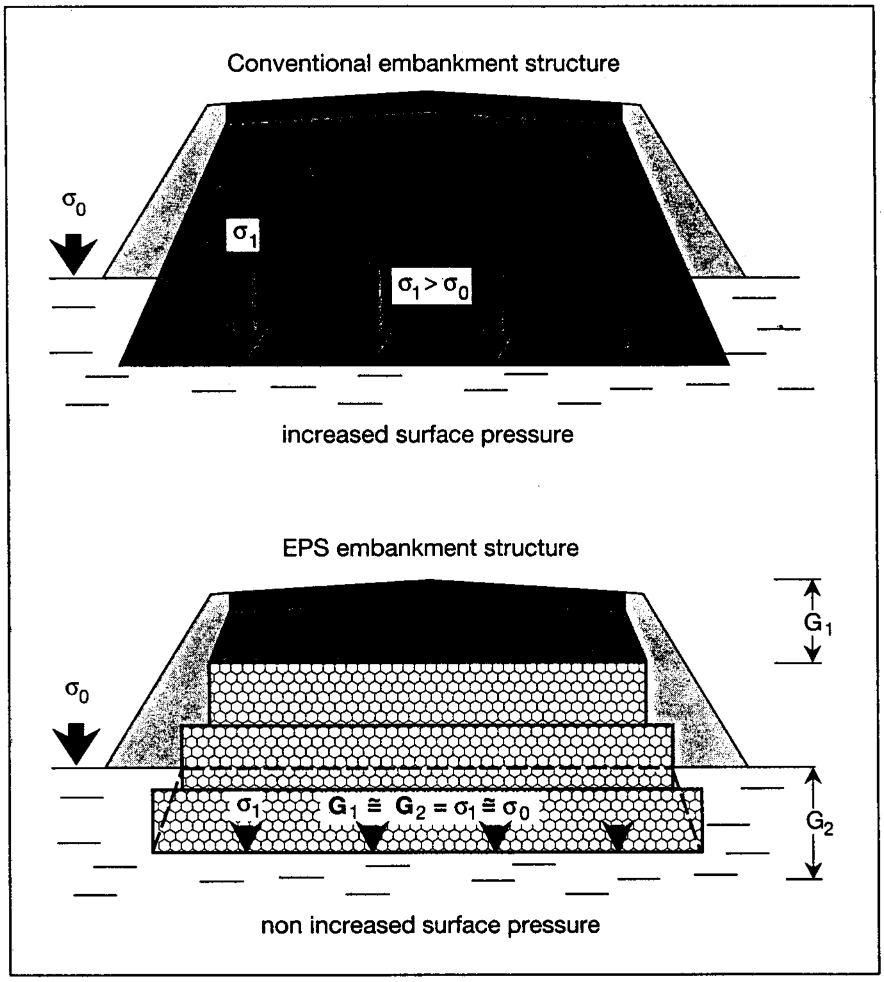


Fig. 1 Comparison of conventional and EPS embankment structures.

New Members

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

N Al Alusi	P Baunton	M Bindra	S Christensen
M Dunphy	P Geddes	B Hill	B Hinson
K McManus	D McGuigan	M McSaveney	P Mohi
P Norfolk	W Scott	A Suchanski	A Swain

All new members now receive a 2-page information sheet on the Society from the Management Secretary, including the name and contact of their local branch coordinator.

L J Wong has tendered his resignation from the Society. The Society membership is now 405. This includes 15 student members.

New Fellows of IPENZ

Three members of the Society have recently been accepted as Fellows of IPENZ for their valued services to the Society and geotechnical practice in New Zealand. Congratulations to Dick Beetham, Tim Sinclair and Andy Olsen.

Should you feel that there are other members of the Society that would make worthy recipients of FIPENZ please let a member of the committee know.

Changes of Address or Personal Details

The Society is often plagued by returned post because members have moved. If you have changed your postal address, email or phone numbers recently, please let me know. We will be publishing an updated membership booklet in February 1999, so please ensure your current details are correct by checking the 1997 Membership booklet or sending an update to me (as below).

Debbie Fellows
Management Secretary

UPDATE: MEMBER'S CONTACT DETAILS

Member's Name: ____________________
Postal Address: ____________________
____
Daytime Telephone No. ____________________
Daytime Fax No. ____________________
E-mail Address: ____________________

***PRIVACY CONDITIONS:** Under the provisions of the Privacy Act 1993, an applicants 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...../....../.......

ISSMGE ACTIVITIES: APRIL – SEPTEMBER 1998

A Board meeting of ISSMGE was held in Atlanta in April 1998, and the following issues were discussed.

- **XV ICSMGE:** Professor Togrol, attending the meeting by invitation, gave an update on the XV ICSMGE, to be held in Istanbul starting 27 August, 2001. The first bulletin for this conference will be published in November 1998.
- **Secretary-General to ISSMGE:** The President, Prof. Kenji Ishihara, announced that Professor Neil Taylor, City University, UK, would be taking over from Dr Dick Parry starting in January 1999, with the hand-over being completed by the Council meeting in Amsterdam in June 1999.
- **Heritage Museum Task Force:** A task force has been appointed to explore the feasibility of setting up a Heritage Museum to provide a show-case for early papers and scientific equipment of the ISSMFE. Prof. Brandl, Vice-President for Europe, is currently chairing the task force.
- **Technical Committees:** The final list was presented, and overlaps (noted) between TC9 and TC17, and TC10 and TC16 (see attached list).
- **GeoEng 2000:** An update was provided on the international joint-society conference, GeoEng 2000 (see attachment). Planning for this conference is progressing well, and anyone who is able to help market the conference overseas is asked to contact the Chair of the organising committee, Max Ervin, in order to obtain flyers.
- **1st International YGPC:** The first international Young Geotechnical Engineers Conference will be held in Southampton, UK, in September 2000. One or two delegates from each National Society will be invited to the conference. The 4th ANZ Young Geotechnical Professionals Conference (YGPC) will be held in Perth in February 2000, and will provide a forum for choosing representatives from Australia and New Zealand for the international conference.
- **Corporate Membership Task Force:** A task force was formed, chaired by Mr Springall, VP North America, to prepare an action document on Corporate Membership of the ISSMGE.
- **SGI Geotechnical Database:** A formal Memorandum of Agreement has been signed between the ISSMGE and the Swedish Geotechnical Institute (SGI), over the geotechnical database SGI-line. Free access to the database over the Internet (public.sgi.geotek.se) has been extended until the end Nov. 1998, after which individuals will be charged US$250 per annum. Any feedback on deficiencies of the database should be provided to Mark Randolph, UWA.
- **Web site for ISSMGE:** A provisional web-site for the ISSMGE has been established **(www.eng.cam.ac.uk/teaching/issmge),** but is likely to be overhauled in 1999. Please provide constructive comments on the web-site to Mark Randolph, UWA.
- **"STOP PRESS" – WEB SITE DISPLACES NEWSLETTER:** In the future, possibly from as early as the start of 1999, the printed version of ISSMGE news would be replaced by a downloadable file from the ISSMGE web-site. ***(! see note below - Editor)***
- **New ISSMGE Awards:** Two proposals for ISSMGE awards have been received. The first proposal, from the Japanese Geotechnical Society, is for young members, to be presented at each international conference; the basis for these awards will be papers presented to the proceeding regional conference and the (current) international conference. The second proposal is from the Swedish Geotechnical Society, for the establishment of a Fellenius Award and Lecture. The details of both these awards are currently under review.

The next Board meeting will be in Paris, in March 1999.

Mark Randolph
VICE-PRESIDENT (AUSTRALASIA)
INTERNATIONAL SOCIETY FOR SOIL MECHANICS & GEOTECHNICAL ENGINEERING

(Note: If you have any comments to forward to your representative on the ISSMGE Board, Mark Randolph, please contact him via email (**randolph@civil.uwa.edu.au**), tel. (618-9380 3075) or fax (618-9380 1044).

TECHNICAL COMMITTEE	SMS	Chairman	Australian or NZ Representative	Fax No:
TC-1 Instrumentation for Geotechnical Monitoring	Turkey	Dr. T. Durgunoglu		
TC-2 Centrifuge Testing	Japan	Prof. T. Kimura	Prof. M. Randolph	+61 8 9380 1044
TC-4 Earthquake Geotechnical Engineering	Portugal	Dr.P. Seco E. Pinto	**Prof. M. Pender (NZ)**	+64 9 3737 462
TC-5 Environmental Geotechnics	Germany	Prof. J.L. Katzenbach	Mr. R. Parker Dr M. Bouazza	+61 3 9818 7990 +61 3 9905 4944
TC-6 Unsaturated Soils	Canada	Prof. D. Fredlund	Dr. B. Richards	+61 7 3378 2078
TC-7 Tailing Dams	Chile	Prof. J.H. Troncoso	A/Prof. M. Fahey	+61 8 9380 1044
TC-8 Frost	Finland	Prof.E. Slunga		
TC-9 Geotextiles and Geosynthetics	France	Dr. J.P. Gourc	Prof. M. Hausman	+61 2 9330 2633
TC-10 Geophysical Site Characterisation	USA	Dr.R.D. Woods	Mr B. Whiteley	+61 2 9888 9977
TC-11 Landslides	Canada	Dr. D.M. Cruden	Prof. R. Fell	+61 2 9385 6139
TC-12 Validation of Computer Simulations	Australia	Prof. J. Carter	Prof. J. Carter	+61 2 9351 3343
TC-13 Mechanics of Granular Materials	Japan	Prof. M. Oda		
TC-14 Offshore Geotechnical Engineering	Norway	Dr. S. Lacasse	Prof. M. Randolph	+61 9 380 1044
TC-15 Peat	Netherlands	Mr. R.J. Termaat		
TC-16 Ground Property Characterisation from In Situ Testing	Canada	Prof. P.K. Robertson	A/Prof. M. Fahey	+61 8 9380 1044
TC-17 Ground Improvement	USA	Prof. I. Juran	Mr. G. Mostyn	+61 2 9385 6139
TC-18 Pile Foundation	Belgium	Prof. W. van Impe	Prof. H. Poulos	+61 2 9888 9977
TC-19 Preservation of Historic Sites	Italy	Prof. C. Viggiani		
TC-20 Professional Practice	India	Prof. V.V.S. Rao	Mr. D. Starr **Mr. M. Stapleton NZ)**	+61 7 3832 1687 +64 9 377 1170
TC-22 Indurated Soils and Soft Rocks	France	Mr. J.L. Durville	Prof. I. Johnston	+61 8 9639 0138
TC-23 Limit State Design in Geotechnical. Engineering	S. Africa	Prof. P. Day	Mr. G. Mostyn	+61 2 9385 6139
TC-24 Soil Sampling	UK	Dr. D. Hight		
TC-25 Tropical and Residual Soils	Brazil	?	Dr. J. Simmons **Dr L. Wesley (NZ)**	+61 7 3278 1004 +64 9 373 7462
TC-26 Calcareous Sediments	Australia	Prof. R.J. Jewell	Dr. M. Khorshid	+61 8 9367 7576
TC-28 Underground Construction in Soft Ground	UK	Dr. R. Mair		
TC-29 Stress-Strain Testing Geomaterials in the Lab	Japan	Prof. F. Tatsouka	Dr. D. Airey	+61 2 9351 3343
TC-30 Coastal Geotechnical Engineering	Japan	Prof. A. Nakase		
TC-31 Education in Geotechnical Engineering	France	Prof J.P. Magnan	Prof. H.G. Poulos	+61 2 9888 9977
TC-32 Risk Assessment And Management	USA	Dr E. van Marcke	Prof. R. Fell	+61 2 9385 6139
TC-33 Scour of Foundations	USA	Prof J.L. Briaud	Dr L. Cheng **Mr B. Melville (NZ)**	+61 8 9380 1018 64 9 ....
TC-34 Deformation of Earth Materials	Greece	Prof Vardoulakis	Dr Hans Muhlhaus	+61 8 9389 1906

REPORT ON TC2 ACTIVITIES: April - September, 1998

The main 4-yearly milestone of the Technical Committee TC2 on Centrifuge Modelling has just passed, with the holding of the international conference on centrifuge modelling in Tokyo in September. Keynote speakers included Professors Andrew Schofield, Bruce Kutter, and Tom Kimura. The conference was very successful, with over 80 overseas participants and a total of 147 papers in Volume 1 of the proceedings. The second day of the three-day conference was devoted to four parallel workshops, held at different centrifuge facilities in the Tokyo area. It is interesting to note that Japan now has 37 separate centrifuges, with 50 % in universities, and the remaining 50% split between national research institutes and industry.
Other initiatives of the Committee have been:

- preparation of a manual for conducting centrifuge modelling, including sample preparation, soil characterisation and procedures for common modelling, such as for shallow foundations; a draft of this manual was distributed at the conference, with a final version to be published in Volume 2 of the proceedings;
- assembly of a CD-Rom collection of photographs illustrating equipment, techniques, visual results and so forth (distributed at the conference, and available through the Tokyo Institute of Technology web-page: geotech.cv.titech.ac.jp/~cen-98/Library/ MAIN.HTM);
- instigation of a collaborative project among different centrifuge facilities in South-East Asia and Australia for comparison of soil preparation and modelling techniques for (a) sand, and (b) clay; a summary report on the sand collaborative project was presented at the conference, with a fuller review to be included in Volume 2 of the proceedings.
- The next centrifuge conference will be held in St Johns, Newfoundland, in July 2002.
- A new journal has been proposed, entitled Journal of Physical Modelling in Geotechnics, to be edited by Professor Osamu Kusakabe, with co-editor Professor Mark Randolph. Negotiations are still proceeding with two publishers, but it is hoped to publish the first issue in the second half of 1999. Anyone interested in submitting an article should contact the undersigned.

Mark Randolph

IAEG ACTIVITIES: June-November 1998:

The Executive Committee and Council meetings held in Vancouver on the 17th and 18th of October 1998, were the last of the current Executive Committee and bring to completion the Action Plan of the last four years of the IAEG. The plan was initiated in 1995 and was directed mainly towards the following matters:

Environmental concerns

Close involvement of IAEG members with environmental quality and assessment, including health and safety issues, together with the large area of common ground between engineering geology and environmental geology persuaded the IAEG executive to alter the name of the association so as to include the word "environment". There was overwhelming support for the change to our current title of :- The Association for Engineering Geology and The Environment.

Most, in fact nearly all Council members, feel that this more accurately reflects the activities of the Association and that it presents a better and more realistic image to the public of what the Association does and what it stands for.

The name is embodied in the statutes and by-laws of the IAEG.

We have a considerable involvement in co-sponsoring of environmental symposia, of which the Athens 1997 meeting was perhaps the best example. Most of our commissions have an environmental purpose, either directly or indirectly.

Related to this increased image on matters environmental is the increased interaction between the IAEG and our sister societies such as the ISRM, ISSMGE, IAH, ITA and the IUGS and it's commission, Co-geoenvironment.

New By-laws

The By-laws were revised thoroughly and have been enacted.

Encouragement of New National Groups

In 1997 three new national groups were welcomed to the IAEG, Georgia, Lithuania and Mongolia. In 1998, at Vancouver, we were able to welcome five more;- Iran, Pakistan, Estonia, Singapore and Indonesia. The latter country was in fact welcomed back after an absence from the activities of the association for some years. It is very encouraging to see this marked expansion of our international influence.

The IAEG has followed a policy over the last four years of fostering new and small national groups. To that end we have kept the fees stable since 1994. There will be a slight rise in 1999 but this will not be applied to countries designated by World Bank criteria to be not of high income.
Our total of national groups has risen to 71.

Development of the Bulletin

The Executive Committee decided to keep improving the Bulletin, both in content and volume. The aim was to produce three and perhaps four issues each year, an increase from the current two issues.

It was decided to separate the roles of Secretary – General and Editor in Chief because of the anticipated extra workload, and to approach a publisher.

Brian Hawkins of the U.K. is now the new Editor in Chief with Roger Cojean of France as the new co-editor, particularly for articles written in French. There will be four issues per year with 1998 being a transition year of three issues. The new title is Bulletin of Engineering Geology and the Environment. Members are encouraged to produce papers and send them to the editors.

Springer has been chosen to publish the new format. Volume 57, No 1, was published in June 1998. We are expecting a wider and greater distribution with these new arrangements.

Routine matters, including Finances

As a result of the Executive Committee's work in 1995, and in particular the efforts of our Treasurer and secretariat, the finances are now in a sound and healthy position.

The IAEG has no debts. Finances have been spent on the Bulletin, the Newsletter and in support of some symposia. The Executive Committee has not incurred any expenses for attendance at meetings. These have carried by National Groups and individuals.

The main meetings of the committee and council were Copenhagen (1995), Beijing (1996), Athens (1997) and Vancouver in 1998.

Membership overall is steady and certainly not in decline. In fact there is a slight increase even when we exclude those who persistently do not pay their fees.

Council unanimously approved an increase in fees for 1999 of 32 US dollars per year for members with the Bulletin to cover the increase in costs.

Secretary – General

The current Secretary, Dr Louis Primel will retire in 1999 and so was not available for re-election in Vancouver. He will be replaced by the current Treasurer Michel Deveughele. The long and very valuable service which Louis has given to the IAEG has been of immense benefit to the Association. He will be remembered as a great shaping force in the history of the IAEG.

New Executive Committee 1999 –2002

The following executive committee was confirmed after elections for the positions of President and Vice – Presidents for Europe.-

President :	Wang Sijing (China)	
Vice Presidents:	Africa:	Rodney Maub (Sth Africa)
	North America :	Richard Gray (USA)
	South America :	Paolo Teixeira Cruz (Brazil)
	Asia :	Ibrahim Komoo (Malaysia)
	Australasia:	Bruce Riddolls (NZ)
	Europe:	Antonio Coelho (Portugal)
		Nieck Rengers (Netherlands)
Secretary –General :	Michel Deveughele (France)	
Treasurer :	Pierre Potherat (France)	

Hans Cloos Medal

This, the highest honour bestowed by the IAEG, was awarded to Owen White of Canada. Owen is a former President of the IAEG and an engineering geologist of international repute. It was particularly appropriate that the medal was presented in Vancouver on the occasion of our 8th Congress.

Richard Wolters Prize

The Prize was presented to Prof. Qing Siqing, of China. Prof. Qing Siqing is a distinguished young Chinese engineering geologist.

The deadline for the next nominations for these two prestigious awards of the IAEG will be June 30th 1999.

Geo Eng 2000

The Geo Eng 2000 Conference was promoted by myself and our President Paul Marinos at every reasonable opportunity at both the meetings and the Congress. The time is now opportune, I believe, to publish and widely distribute the First Circular. Already there are a number of circulars being distributed for other events on the year 2000 geotechnical calendar. There is keen interest in Geo Eng 2000 and I have encouraged as many people as possible in the IAEG to make plans to attend.

Finances

An overall result of + 28,000 FF at 31 December 1997 is reported in the 1997 financial report. There is a forecast of a net loss of 120,000 FF at the end of 1998 as a result of expenses incurred in the change over to the new arrangements for the Bulletin and the Newsletter and the change over to the new secretariat. This will lead to a decrease of 20% in the assets of the Association but should be offset from 1999 onwards by a positive result. The 1999 forecast is for a surplus of around 30,000 FF, or 6,000 USD.

I will be reporting again on other matters in the near future, such as the Vancouver Congress itself which was a most enjoyable event. It was especially pleasing to see other New Zealanders and Australians at the Congress contributing to the Scientific and Technical programme by way of papers in the proceedings and also as presentations including Keynote addresses.

For the meantime, my sincere appreciation to the New Zealand Geotechnical Society for supporting me to these meetings and the Congress. I am very grateful for the consistent and full level of support I have received over the last four years.

Warwick Prebble
VICE PRESIDENT, AUSTRALIASIA, 1995-1998
INTERNATIONAL ASSOCIATION FOR ENGINEERING GEOLOGY & THE ENVIRONMENT

(Note: If you have any comments to forward to your representative on the IAEG Board, Bruce Riddolls (as VP for 1999 – 2002), please contact him via email (Riddolls-Grocott@mactropolis.co.nz), tel. (643- 377 5696) or fax (643- 377 9944).

A GLOBAL VIEW FROM THE PACIFIC RIM

VANCOUVER, B.C., CANADA . SEPTEMBER 21-25 1998.

The 8th Congress of the IAEG was hosted by the Canadian Geotechnical Society and held in conjunction with the 15th Canadian Tunnelling Conference which was organised by the Tunnelling Association of Canada. The 41st Annual meeting of the Association of Engineering Geologists (AEG) was held in Seattle, Washington, USA during the week after the Congress.

This planned coordination of the three major meetings allowed for delegates to participate in more than one if they wished to. It also led to the running of some joint IAEG – AEG field trips in between those two conferences.

The Canadian Tunnelling Conference was held in parallel with the last two days of the IAEG Congress and continued on Saturday 26th September. Delegates to both were able to move between the various sessions and select from the wide variety of papers being presented. The combination of the different organisations and the opportunity for both formal and informal interaction created a conference atmosphere that was stimulating and rewarding.

Several hundred delegates from many different countries attended the Congress, including three New Zealanders and seven Australians. Most of the Australians and New Zealanders either presented papers and /or contributed them to the Congress Proceedings. Some of us also chaired Sessions. It was very pleasing to see such a high level of commitment and participation from AGS and NZGS members.

A keynote address was presented by John Read of Australia on "Deformation of high rock slopes in open pit mines", as the introduction to the first session of the theme on "New developments and case histories in surface workings".

Congress Themes

The seven themes each commenced with a keynote address and covered the following topics; -

- Site Investigations
- Natural Hazards
- The Environment
- Construction Materials
- Surface Workings
- Underground Excavations
- Coastal and Offshore Engineering

The Scientific Programme began with a special session on the engineering geology of the Canadian Cordillera. Poster sessions were also held throughout the week with authors being present at specific times. The six volume conference proceedings are available from Balkema. The reference is Moore, D and Hunger, O 1998; Engineering geology and the environment. Proceedings of the eighth international congress of IAEG, Vancouver, 21-25 September, 1998. c5000 pp.

Field trips

There were several technical tours, some lasting five to seven days to places such as Alaska and the Rockies. During the Congress a number of one –day technical tours visited a diverse range of sites such as Howe Sound, Fraser Delta, Downtown Vancouver, landfills, Coast Mountains forestry and the Fraser Valley. The last mentioned tour took delegates to the famous Hope Slide and also the Cheam Slide, Katz Slide and the Fraser Canyon.

Social and general matters

The Canadian Geotechnical Society hosted and organised an excellent social programme. Before the Congress, the IAEG Executive and Council held their annual meetings and the election of the new Executive Committee. These meetings have been reported on in another article.

The Congress was very stimulating and enjoyable, with much technical and scientific content of direct relevance to other Pacific Rim countries.

I am sure that all who were present would compliment the Canadians on running a very successful Congress.

Warwick Prebble
Vice President IAEG for Australasia.

ISRM ACTIVITIES: April to October 1998

Introduction

This report will provide an overview of ISRM matters over the period of April to October 1998. The Board and Council met at NARMS98 in Cancun Mexico in early June and thus there is much to report. AGS and NZGS action is required in the matters of Interest Groups and Language.

1999 Congress - Paris

Organisation for the Paris Congress is advancing well. The Congress will be held at the Palais des Congress between Wednesday 25th and Saturday 28th August 1999.

The Australian papers have been selected. Authors and titles are shown on the attached table, authors have been advised. In addition, I understand that the NZ contribution is also decided and I have included details on the same table. ***Full papers, abstracts and translations and internet abstracts are required by 31st December 1998.***

AUTHOR(S)	TITLE
Baczynski	STEPSIM4: Step-path based method for assessing slope stability risks
Barr & Jupe	The Kaiser effect for samples pre-stressed at 820m and 2.4 km with stress tensor results
Beck & Brady	A moment tensor density method for prediction of mining induced seismic risk
Duplancic & Brady	Characterisation of caving mechanisms by analysis of mine seismicity and rock stress
Duran	Do slopes designed with empirical rock mass strength criteria stand up?
Enever, Gunning, Gale & Reynolds	A statistical study of the variability of the measured horizontal rock stress field in a sedimentary basin
Fama, Shen, Craig, Follington & Leisemann	Layout design and case studies for highwall mining
Galvin & Hebblewhite	Geomechanics developments in mining coal under strong roof and weak floor conditions
Glastonbury, Mostyn & Fell	Analysis and prediction of pre and post failure deformation of rock slopes
Indraratna & Ranjith	Deformation and permeability characteristics of rock with interconnected fractures
Kelly, Gale, Luo, Hatherly, Balus & LeBlanc-Smith	Ground behaviour about longwall faces and its effect on mining
Lee & Mikula	Estimation of the large scale insitu shear strengths of geologic structures at Mt Charlotte Mine, Kalgoorlie, WA
Tan, Choi & Richards	Coupled Physico-chemical interaction and thermoporoelasticity mechanisms in shale
Read, Richards & Perrin New Zealand contribution	**Applicability of the Hoek-Brown failure criterion to New Zealand greywacke rocks**
Windsor	Systematic design and reliability assessment of rock reinforcement and support for jointed rock masses
Wold, Choi, Wood, George & Williams	Longwall mining beneath a gorge: Its impact on rock stress, fracture and subsidence, and the release of reservoired gases
Wu	Computer aided stress and strain paths test and their effects on mechanical behaviour of reservoir rocks

Muller Award

Further to my last report, nominations had closed for the Muller Award and, therefore, I did not pursue the nomination of Prof Brown. I would like to suggest that we should make a very strong nomination for Prof Brown for the Fourth Award, decided in 2002. I hope the incoming Vice President will champion this with support from the regional National Groups. Prof Herbert Einstein of MIT was selected as the recipient of the Third Muller Award.

Rocha Medal

The Rocha Medal was considered and Dr Arno Daehnke of CSIR, South Africa, was awarded the Medal for his thesis entitled *"Stress Wave and Fracture Propagation in Rock"*. The regional nomination, the thesis entitled *"The modelling of flexural toppling of foliated rock slopes"* completed by Dr Deepak P Adhikary at UWA, was very highly regarded and has been included in next year's consideration (the only such nomination to be so considered).

Interest Groups

The VP South Africa, Dr Nielen van der Merwe, has prepared a detailed proposal for "Interest Groups" to exist in parallel with National Groups within the Society. These will be self funding and mainly for communications. The governance of the Society will still reside with the Board and Council as currently constituted. The proposal for changes to the Statutes and By-Laws has been sent to each National Group for a postal vote. I recommend that the AGS and NZGS favourably consider this proposal and lodge their vote by the required date.

Language

Prof Herbert Einstein (VP North America) and myself have drafted suggested changes to the language clauses in our statutes, by-laws and guidelines. These were considered at the Board and Council meetings. The Board considers that it is time to change the statutes to reflect the fact that the Society has moved from a Eurocentric one to a truly world wide society. The Board agreed on a proposal that preserves the status of French and German as official languages but does not require simultaneous translation at Congress nor for abstracts to be translated as at present. But it does allow for papers at Congress to be written in any of the three official languages. The AGS and NZGS should now have received these proposals for comment and then, after modification, for voting at the Paris Congress. Each National Group should consider this matter carefully. Some National Groups have indicated that they would support a proposal that removed any reference to French or German and had English as the only official language. It is likely that such a proposal would gain the necessary two thirds majority. *(This proposal was supported by the NZGS - Editor).*

Other matters

GeoEng2000 has been selected as the International Symposium of the ISRM for the year 2000. The Board and Council meetings will be held during the Symposium.

The Guidelines for surcharges due to the Society for Regional and International Symposia were revised. ISRM International Symposia would have a 5% surcharge, while joint meetings and regional symposia would be negotiated by the regional vice president.

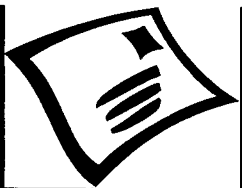
Several new National Groups, including Nepal, have been admitted to the ISRM in the last year. Recently a group in Singapore has applied for admission. The ISRM has to determine its position with regard to this matter as Singapore is currently considered to be part of the South East Asian

Geotechnical Society. I would be interested in hearing the views of the AGS, NZGS and the VPs of the ISSMGE and IAEG on this matter.

As always, I encourage all members to consider whether they have anything that they would like to write up for publication in the ISRM News Journal, this is a very good chance to show off both research and practice with newsy type articles. Please advise your local groups regarding this and encourage them to prepare material. I would be very happy to assist potential contributors if required. Each VP is responsible for one issue of the ISRM Journal and I am looking for contributions on rock slopes for Vol 6 No 3.

Garry Mostyn
VICE PRESIDENT FOR AUSTRALASIA
INTERNATIONAL SOCIETY FOR ROCK MECHANICS

If you have any comments to forward to your representative on the ISRM Board, Garry Mostyn, please contact him via email (g.mostyn@unsw.edu.au) tel. (61-2-9385 5021) or fax (61-2-9385 6139).



Have You Changed Your Personal Details??

If you have moved, changed employer, phone number or e- mail address, please let us know by contacting the Management Secretary.

Contact Details:-
Debbie Fellows
Tel:- 09 8177759
Fax:- 09 8177035
Email :- dfellows@xtra.co.nz

Société Internationale de Mécanique des Roches

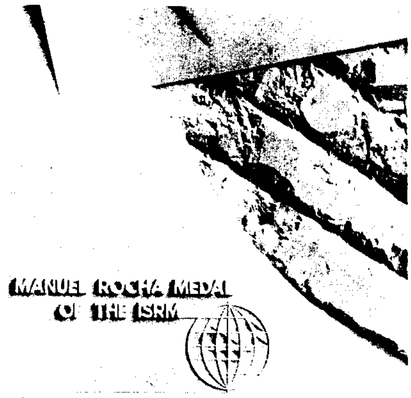
International Society for Rock Mechanics

Internationale Gesellschaft für Felsmechanik

ROCHA MEDAL 2000

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

AN INVITATION IS NOW EXTENDED TO THE ROCK MECHANICS COMMUNITY, AND SPECIALLY TO FACULTY MEMBERS, FOR NOMINATIONS FOR THE ROCHA MEDAL 2000.



Past Recipients

1982 – A. P. Cunha, Portugal; 1983 – S. Bandis, Greece; 1984 – B. Amadei, France; 1985 – P. M. Dight, Australia; 1986 – W. Purrer, Austria; 1987 – D. Elsworth, UK; 1988 – S. Gentier, France; 1989 – B. Fröhlich, Germany; 1990 – R. K. Brummer, South Africa; 1991 – T. H. Kleine, Australia; 1992 – A. Ghosh, India; 1993 – O. Reyes W., Philippines; 1994 – S. Akutagawa, Japan; 1995 – C. Derek Martin, Canada; 1996 – Mark P. Board, USA; 1997 – Martin Brudy, Germany; 1998 – Fiona MacGregor, Australia; 1999 – Arno Daehnke, Republic of South Africa.

APPLICATION

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

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

A thesis summary in one of the languages of the Society, preferably in English, of about 5 000 words, detailed enough to convey the full impact of the thesis and accompanied by selected tables and figures, with headings and captions also presented in English. In the interest of the candidates it is recommended that this summary be prepared at the level of a refereed paper;

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

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

Application Deadline

The nomination shall reach the ISRM Vice-President concerned by registered mail not later than 19981231.

*** Vice-Presidents**

For Africa:
Dr. Nielen van der Merwe, Itasca Africa (Pty) Ltd., PO Box 38425, Booysens 2016, Republic of South Africa
For Asia:
Dr. Ou Chin-Der, Public Construction Commission, 9th F1., No. 4, Chung Hsiao W-Rd., Sec. 1, Taipei 100, Taiwan, Republic of China
For Australasia:
Mr. Garry R. Mostyn, School of Civil Engineering, University of New South Wales, Sydney, NSW, 2052, Australia
For Europe:
Prof. Giovanni Barla, Technical University of Turin (Politecnico), Structural Engineering Department, Corso Duca degli Abruzzi, 24, 10129 Torino, Italy
For North America:
Prof. Herbert H. Einstein, Massachusetts Institute of Technology, 77 Massachusetts Avenue, Room 1-324, Cambridge, MA 02139, USA
For South America:
Prof. Michel L. Van Sint Jan F., Pontificia Universidad Católica de Chile, Depto. Ingenieria Estructural y Geotécnica (441), Casilla 306, Santiago 22, Chile

AUCKLAND BRANCH ACTIVITIES

The Auckland branch of the NZ Geotechnical Society has held eight meetings since May 1998.

The Northern Region Student Prize presentations were held in mid-November. Four presentations were made for the Student Prize. This year the presentation evening was held at the University of Waikato and was attended by some 30 people. Congratulations to the prizewinner, Darren Wise. *(Synopses of the presentations are published in the following article, 1998 Student Prize - Editor).*

All meetings were well attended and the presenters' efforts were well received by the audiences. Many thanks to our sponsors, Ground Engineering Ltd and Maccaferri NZ Ltd, for their continuing support. The programme is presented below.

DATE	TOPIC	PRESENTERS
Thurs 4 June	Viaduct Basin Development	Dr David Carter Daniel MacKereth
Thurs 25 June	Metropolis Tower Foundation Design Sun Alliance Foundation Design	Chris Freer, Andrew Langbein, Chris Bauld
Thurs 30 July	Quay Park Ground Remediation	Walter Starke, J. Grant Murray
Thurs 27 August	Mercury Energy CBD Tunnel	Keith Dickson Tony Pink Bernard Hegan
Thurs 24 September	Landfill Liner Design	Malcolm Stapledon, Tony Kortegast
Thurs 29 October	Deep Basement Construction. Dynamic Testing Of Piles	J. Grant Murray Maurice Fraser
Mon 16 November	Northern Region Student Prize	Christine Simpson, Darren Wise Jonathon Sickling, David Dravitiski
Thurs 26 November	Geotechnical Properties Of Volcanic Soils	Mick Pender, Laurie Wesley, Tam Larkin, Graeme Duske

Neil Watson
Auckland Branch Coordinator
(Contact details: Tel. (09) 379 1200, Fax (09) 3791210, E-mail nwatson.worley.co.nz)

WELLINGTON BRANCH ACTIVITIES

The Wellington Branch had a quiet year in 1998 with four meetings as follows:

- Mike Isaac and Colin Mazengarb on geological maps for the 21st century and the QMAP series currently being prepared by the Institute of Geological and Nuclear Science,
- Stewart Palmer and Dr Julian Seidel from Beca Carter on construction of the Westpac Stadium with particular focus on ground improvement and piling,
- Dr Julian Maund from Opus on the construction of deep cut-off wall for a 5-storey underground carpark for the Diagonal Mar shopping complex in Barcelona, Spain.
- Dr Graham Ramsay from Beca Carter on foundations for the new Otira Viaduct currently under construction on SH73 near Arthur's Pass

We are currently compiling our programme for 1999 with a goal of a talker every 6 to 8 weeks. A rough programme subject to change and confirmation of some speakers is:

8 December - Dr Mick Pender on pumice soils and the CPT

2 February - Russ Van Dissen on recent research on faulting in and around New Zealand

16 March - Ian McPherson on either The Abbotsford Landslide or Mistakes that I have seen other people make

27 April - Mike Crozier with the topic to be announced.

Hopefully Mick Pender will give us another talk and Tim Sinclair will be delivering the Geotechnical Society lecture later in the year. We are planning on a full year's programme and look forward to good support from the members.

Ian McPherson
Wellington Branch Coordinator
(Contact details: Tel. (04) 472 9589, Fax (04) 472 9922, E-mail idm@wel.conwag.co.nz)

CHRISTCHURCH BRANCH ACTIVITIES

The Southern Zone section of the New Zealand Geotechnical Society student prize was held on Friday 9 October 1998. This year three students presented short talks on their respective post graduate work. *(Synopses of the presentations are published in the following article, "1998 Student Prize" - Editor).*

- Paul Ollet, Department of Natural Resources Engineering, Lincoln University. *"Landslide Dam Formation in the Callery River".*
- Justin Harrison, Department of Geology, University of Canterbury. *"Filtration of Port Hills Loess for Retaining Wall Situations".*

- Brian Adams, Department of Civil Engineering, University of Canterbury *Two-Dimensional Response of the Lower Hutt Valley in the Event of a Major Rupture of the Wellington Fault".*

The judging panel comprising Messrs Don MacFarlane and Marton Sinclair awarded the prize to Justin Harrison. All talks were of a particularly high standard and considerable effort had gone in to their respective presentations.

Guy Grocott
Christchurch Branch Co-ordinator
(Contact details: Tel: (03) 377 5696, Fax:(03) 377 9944, E-mail: Riddols_grocott@ mactropolis.co.nz)

OTAGO BRANCH ACTIVITIES

The Otago Branch has had a quiet year, with no scheduled meetings. Phil Glassey has passed the role of Branch Co-ordinator to Tim Browne of Opus International Consultants. Tim is ably supported in the role by the geotechnical team in Dunedin, including Ian Walsh and Graham Brown.

We will be running a fieldtrip to the Horseshoe Bend Dam during the last week in January during placement of the RCC (Roller Compacted Concrete) dam. This (as are all our meetings) will be open to all IPENZ members.

We will have our next meeting in February, when Ian Walsh and Tim Browne will talk on "Geotechnical Aspects of the Homer Tunnel, Fiordland", and David Stewart will talk on mine subsidence under the proposed Fairfield Bypass, Dunedin.

On a general note, the Otago Branch meetings are well supported by the wider engineering community in Otago and Southland. If any potential speakers are coming through Dunedin, we would welcome the opportunity to hear them talk about current geotechnical issues or projects."

Tim Browne
Otago/Southland Branch Co-ordinator
(Contact details: Tel: (03) 474 8899, Fax:(03) 474 8995, E-mail: tim.browne@opus.co.nz)

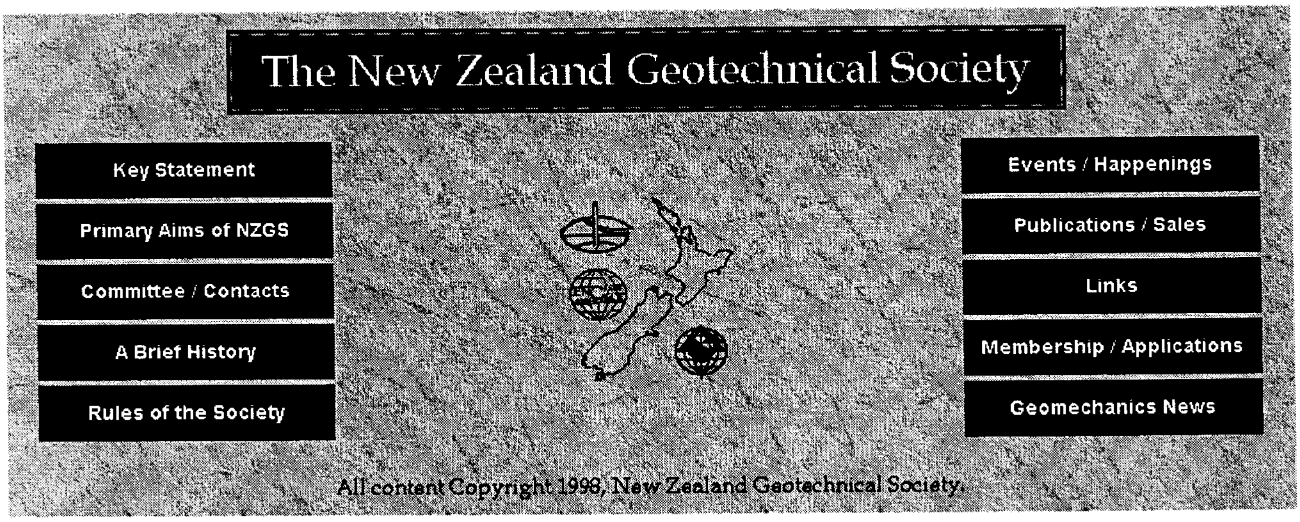
Note changes to the NZGS Soil & Rock Description Guidelines -
see page 60

THE NZ GEOTECHNICAL SOCIETY WEB SITE

http://www.ipenz.org.nz/geotech

The Society's web site was created earlier this year. The pilot version is currently undergoing a major overhaul, expanding to give more information on the society and provide links to other geotechnical sites.

It will become an increasingly important communication medium, both within the Society and with the wider international geotechnical community.



The 1999 NZ Geomechanics Lecture

Geotechnical Analyses: Fundamentals to Fractals

T. J. E. Sinclair, MA, MSc, DIC, MIPENZ, MICE, C.Eng
Director, Tonkin & Taylor Ltd, Auckland

The Geomechanics Lecture was established to honour individuals who have made a notable contribution to New Zealand Geomechanics. There have been nine lectures since 1974 and it remains a prestigious award for the Geotechnical Society.

The Society is proud to announce that Tim Sinclair will be presenting the 1999 Geomechanics Lecture at the 8th ANZ Geomechanics Conference in Hobart early next year. Tim will also tour selected New Zealand centres in the first half of 1999, with dates and venues to be announced closer to the time.

ABSTRACT

The purpose of the paper is to examine some of the methods of analysis in geotechnical engineering and their relevance to the real world. The first part of the paper considers numerical modelling as an analytical tool, using some historical examples to illustrate the range of applications, the need for such methods and the failings. The second part returns to fundamentals, with a case history to demonstrate how the very simple basic concepts can provide insight to complex problems. The conclusion looks forward to how developing techniques might eventually add to the practising engineer's capabilities.

The New Zealand Geotechnical Society Student Prize

INTRODUCTION

The New Zealand Geotechnical Society wishes to recognise and encourage student participation in the fields of soil mechanics, rock mechanics, and engineering geology. It therefore presents annually, two merit awards, known as the **"New Zealand Geotechnical Society Student Prize"**.

The award is made to the bona-fide full-time student of a recognised Tertiary Institute in New Zealand who makes the adjudged best presentation on any aspect or topic in the field of geomechanics to the designated Local Branch Meeting in either Auckland or Christchurch. Students are normally required to speak for 20 minutes followed by 5 minutes of questions.

The prize is awarded in each region to the student who is judged to have made the best presentation in terms of clarity, and who is considered to have dealt with questions most competently. The composition of the judging panel is a matter of the Local Branch convenor, and the judges' decision is final.

The following synopses are a summary of the presentations made by each student.

LANDSLIDE DAM FORMATION IN THE CALLERY RIVER

PAUL OLLETT
Department of Natural Resources Engineering, Lincoln University

In 1930, Franz Josef (FJ), Westland, was threatened by flooding in the adjacent Waiho River. This flooding was initiated in the Callery River, which is a tributary to the Waiho River. Here, a landslide completely blocked the Callery River for one day. The resulting flood was extremely high, and debris from these slips was carried downstream. Such occurrences are called landslide dam failures, and until recently, their threat to the FJ residents and facilities had been mainly forgotten. Information is required to assess the extent that landslide dam failures may contribute to flooding of FJ.

The most crucial component of this assessment is the selection of historic and future potential landslide sites in the Callery gorge. Past studies (by examining aerial photographs) have identified major landslide sites, and from these seven were selected. It is also important to determine how a schist landslide dam will form in the Callery River. The dimensions of a landslide dam have a significant affect on the breaching process and subsequent outflow. This was accomplished by building a physical model to represent the valley shape, carrying out a series of small scale landslide tests using a suitable material, then physically measuring the formed dam dimensions.

Charts were constructed from the test data, and the likely dam face slopes of the seven selected sites were determined. Then, by taking advantage of the obvious symmetry in the Callery River's gorge, equations were derived to calculate dam and impounded lake dimensions of the seven sites. Formed dam and lake dimensions are to be supplied to a computer model (Mike 11) to simulate the effect of landslide dam failures in the Callery River.

FILTRATION OF PORT HILLS LOESS FOR RETAINING WALL SITUATIONS

JUSTIN HARRISON
Dept of Geology, University of Canterbury

The talk will be a precis of my thesis, entitled "Filtration of Port Hills Loess for Retaining Wall Situations". This has been a laboratory based research project, built around a three-tiered testing programme, and focuses on geotextiles as filtration media.

The Stage 1 test is designed to be a simple test, and provide comparative information on permeability and retention between filtration options. It is also hoped that the relative data between filtration options will be comparable to the standard test also used (ASTM D5101-90 – see later). This test has provided good initial results. Observation of samples subjected to this test have been conducted using a Scanning Electron Microscope (SEM), and have indicated the formation of a "vault structure" as opposed to the traditionally thought of bridging filter.

The ASTM test provides data on permeability and gradient ratio (thought to be a direct indicator of the efficiency of the soil/geotextile system).

Finally, the Stage 2 test is a laboratory-based simulation of retaining wall situations, and has the ability to investigate the effects of a number of variables.

TWO-DIMENSIONAL RESPONSE OF THE LOWER HUTT VALLEY IN THE EVENT OF A MAJOR RUPTURE OF THE WELLINGTON FAULT

BRIAN ADAMS, POSTGRADUATE
Department of Civil Engineering, University of Canterbury

Alluvial valleys bounded by stiff bedrock tend to trap seismic energy, creating an amplified and longer duration response. Lower Hutt is situated on one such alluvial valley, bounded by the potentially active Wellington Fault along its western edge. The fault represents a major seismic hazard to the Wellington region, with a characteristic earthquake of magnitude 7.1-7.8 rupture expected within the next few hundred years. Here we use a 2-D elastic finite element analysis to simulate the propagation of seismic waves through the valley. Several geological cross-sections are studied in order to gauge the overall response of the valley under earthquake excitation.

The response is found to be characterised by the propagation and interaction of surface waves generated at the valley edges. The deep wedge of soft alluvium is excited into 2-D modes of resonance by the surface waves as they reflect back and forth across the valley. At certain locations, displacements are amplified up to 18 times that of the nearby outcropping bedrock. Amplifications are also highly frequency dependent, with certain eigernmodes in the range 0.5-2.5 hertz giving the greatest response. Surface waves also cause localised effects close to the valley sides. The aptly named basin-edge effect[1] generates large displacements as incoming body waves construct with the horizontally propagating surface waves. It appears to act as a broad-band amplifier along a strip of land 70-200 m away from the fault trace on the valley's western edge.

Post-processing of simulation results in the frequency domain allow us to estimate the intensity and spatial distribution of likely ground motions during a major rupture of the Wellington Fault. Estimated response spectra may be calculated at specific locations for use in structural design, while a general picture of the shaking intensity distribution within the valley may be shown on spatially varying plots of Fourier amplitude spectra.

[1] Kawase, H. (1996): The cause of the Damage Belt in Kobe: "The Basin-Edge Effect", Constructive Interference of the Direct S-Wave with the Basin Induced Diffracted/Rayleigh Waves. *Seismol. Research Lett. 67(5):25-34*

LANDSLIDE SUSCEPTIBILITY MAPPING FOR THAMES AND KAUAERANGA VALLEY

CHRISTINE SIMPSON,
Dept of Earth Sciences, University of Waikato

A landslide susceptibility map of the upper slopes of Thames and the northern slopes of the Kauaeranga Valley is presented. This region is characterised by well-jointed andesites that range from slightly weathered to completely weathered. This material therefore has the potential to fail along discontinuities as well as at shallow levels in weathered andesite.

Landslide susceptibility mapping depicts division of a land surface into zones of varying degrees of stability. Factors recognised as being important in controlling stability are lithology, geomorphology, landuse, drainage, lineations, strength, durability, and plasticity. An initial desk study using aerial photographs identified large-scale movements, while fieldwork was undertaken to identify remaining geomorphology, as well as geology, drainage and landuse. A laboratory program was designed to determine particle size, durability, mineralogy, plasticity, strength and density.

Overlying maps of lithology, geomorphology, lineations, drainage and landuse showed small failures occur in weathered andesite or colluvial material where land is sparsely vegetated. These failures showed dominantly slide-flow mechanisms, with the main triggering factor appearing to be rainfall. Field tests showed the strength of these materials to lie between the rock/soil boundary, having very low Schmidt hammer readings and high shear vane and penetrometer readings. Particle size analysis using a Malvern Lasersizer identified soils as silty clays, with clay contents ranging from 10 - 50%. The weathered andesites are characterised by low to very low durability index values, with low plasticity indices in the range of 10 to 45%. The liquid and plastic limits fall within the ranges of 50 - 115% and 7 - 65% respectively, with the higher values corresponding to more intensely weathered horizons. These completely weathered andesites are highly jointed through weathering and relict andesite joints. These influences the stability of the slope on a larger scale.

Large deep-seated failures identified from field work and aerial photo examination are structurally controlled as wedge and planar type failures. Stereonet analyses show dominant joint sets to be oriented in directions detrimental to the safety of the slope. In general these failures are unaffected by landuse or drainage patterns, with geology and discontinuities being the dominant controlling factors. Field observations identified a large number of slickensided joints that have at least 2 mm of clay infill. Strength tests of this infill, and the strength of the weathered andesite in this area is currently under investigation so that the stability of these slopes may be evaluated.

AN INVESTIGATION OF THE WAIROA NORTH FAULT, SOUTH AUCKLAND

DARREN WISE
Dept of Geology, University of Auckland

The Wairoa North Fault dissecting the Hunua Ranges, South Auckland is of Quaternary age and may be potentially active. It downfaults Mesozoic basement greywacke to the West and the resultant fault angle depression has been infilled with Pleistocene and recent sediments. The Paparimu segment of the Wairoa North Fault is composed of a number of smaller segments offset along prominent East to North-east trending defect sets. A series of coalescing depositional fans of early Pleistocene age (the Paparimu Formation) conceals the fault plane in various locations. This formation consists of compact stiff iron-stained grey clays overlying friable greywacke derived gravels with a clayey silty matrix. Exposures of the greywacke basement display a highly to completely weathered greyish brown weak (R2) to very weak (R1) rock mass with closely spaced fractures, and considered a hard (S6) soil mass in some places.

Distinctive breaks in slope occur on the surface of the fans near the base of the greywacke range and large well developed sag ponds exist on the downthrown block. Several streams flow parallel to the range front probably along fault plane and gouge zones. Range-front parallel "notch" gullies are evident in the Pleistocene deposits. A large occurrence of ground collapse pits in the Paparimu Formation indicate this deposit is prone to active subsurface tunnel gully erosion. The fault plane may be capturing the subsurface drainage system and controlling the distribution and termination of the collapse pits.

Gravity surveying has been used to accurately locate the position of the fault at various locations and to delineate the subsurface form of the fault plane. Results indicate the fault plane to be dipping 60° W, with a throw of at least 70m. Large zones of extremely weathered greywacke have also been interpreted from the gravity data. Electrical resistivity profiling has resulted in two dimensional images which clearly show the sharp contrast between the unconsolidated alluvial sediments and the indurated greywacke representing the fault. The electrical resistivity method could not distinguish any displacements in the overlying soils. A shallow high resolution seismic reflection survey identified a prominent reflecting horizon at 45m depth in the basin sediments which apparently terminates at the fault plane.

Engineering geological and geomorphological mapping correlates well with geophysical results to indicate the existence of a 4.5m fault scarp in the Paparimu Formation and hence demonstrating a Pleistocene age. However there is no evidence of deformation in Holocene stream deposits immediately north of the fault scarp suggesting no Holocene movement on the fault. It is intended to excavate and log a trench across the fault scarp and possibly drill boreholes in the surrounding area to reveal the late Quaternary activity of this fault.

SHEAR STRENGTH OF A FAILED SLOPE, EAST TAMAKI, SOUTH AUCKLAND

JONATHON SICKLING
University of Auckland School of Engineering,

The "Southern Landslide Zone" is a large area of identified slope instability occurring in Waitemata Group material in the Brookby-Alfriston-East Tamaki area. The project site is dominated by a large blockslide measuring over 500m from head scarp to toe. Investigations for a residential subdivision near the toe of the slope has identified extensive slope debris. Examination of the slope debris shows the material comprises angular fragments of low or unweathered sandstone and mudstone in a silty/clayey matrix. Frequent slickensiding is observed in this material. Weathering of the top 2m of the slope debris has formed a residual soil, with shear surface structures preserved from the underlying slope debris.

Within the slope debris a number of small secondary rotational slumps occur near the block slide toe, with slope angles prior to failure estimated to be as low as 7° in some cases. This is of obvious concern if cuttings steeper than 7° are to be made in this material. The slope debris is inferred to be a homogeneous soil containing multiple shear surfaces attributed to the large displacements involved during the primary block slide failure.

Undisturbed sampling included two samples taken from different vertical horizons in one of the slumps. Laboratory testing involved consolidated undrained triaxial testing to determine the peak effective stress strength parameters c' and f', and ring shear tests on remoulded samples to determine the residual effective stress strength parameters c'_R and f'_R. Atterberg limit tests were carried out to determine the plasticity index for each sample.

Several different methods of back analysis were performed to compare with the laboratory test results. Assumptions included zero effective cohesion, circular or log-spiral failure surface, water table at the ground surface, bulk unit weight and slope angle prior to failure (based on adjacent unfailed slopes).

It was found that back analysis generally agreed well with the residual strength parameters obtained from the ring shear tests. Allowing for lateral variation in material properties over such a large site (sample locations are over 200m apart), results from the ring shear tests indicate f'_R ranges between 15.4° to 21° while back analysis gave f'_R ranging between 14° and 24°. Peak strength parameters from the triaxial tests gave c' between 0-17 kPa and f' = 29.2°, which do not agree with the back analysis. The higher values of peak strength for the failed soil are attributed to the secondary failures probably occurring on existing failure surfaces (slickensides) at the residual state, while triaxial test samples did not necessarily contain these failure surfaces.

Therefore back analysis is a cheap and reliable method of predicting f'_R for rotational failures in slope debris or colluvium in Auckland's "Southern Landslide Zone". Back analysis of large block slides is far more difficult, as estimates of slope angles prior to failure for prehistoric slides must be subject to a large degree of uncertainty. In these cases samples must be taken from the failure surface (eg weak clay seam) to accurately determine f'_R by ring shear testing. Ring shear tests on thin clay seams from similar block slides in the region have measured f'_R to be as low as 9°.

A GEOMECHANICAL INVESTIGATION OF LAKE SEDIMENTS WITHIN PARADISE VALLEY, ROTORUA

DAVID DRAVITZKI,
Dept of Earth Sciences, University of Waikato

Paradise Valley is located directly to the west of Rotorua City and Mt Ngongotaha and has been known to have erosion and minor slips occur along Paradise Valley Road. Lake sediments deposited c. 22-80 ka by previous high stands of Lake Rotorua water levels were investigated to determine strength parameters and erosional mechanisms as part of a MSc study for the University of Waikato with the involvement of Environment BOP.

Terraces in the basin are heavily dissected and are comprised of three packages of diatomaceous lake sediments separated by two stratigraphically distinct peat bands dated at 24-33 ka and >65 ka respectively. The terrace deposits are capped by five metres of loess, tephra and soil horizons. Three sites (eastern, western and road) were chosen to investigate the loess cap and three layers of lake sediment in order to characterise the range of geotechnical properties in both horizontal and vertical distribution throughout the basin.

Field observations indicated that the lake sediments are highly erodible and may be potentially sensitive. The lake sediments consist predominantly of silt sized (80%) material with 10% clay minerals. Atterberg limits indicate two types of behaviour present: medium-high plasticity with plastic limit 40-60%, liquid limit 50-70%, plasticity index 9-18%; and high plasticity with plastic limit 75-80%, liquid limit 110-120%, plasticity index 35-45%.

The high plasticity behaviour has only been observed in the western site and is believed to be influenced by a local source of clay mineral, possibly a zeolite mineral. The high plasticity of the lake sediments is also supported by a high activity, ranging from 1.05 - 4.73 (average 1.99) although this property is influenced by the low clay fraction in the sediments. The lake sediments are characterised by an intermediate - high dispensability (predominantly D1, but range down to ND3 and ND4) according to the pinhole tests, which is evidenced by field observations of piping.

To investigate the possibility of sensitive behaviour, direct shear and ring shear tests were carried out in order to obtain intact and remoulded strengths of the lake sediments. The direct shear testing has shown that there is no statistically significant variation in the strength properties of the lake sediments within the basin. Friction angles and cohesion values were typical of silts with friction angles ranging from 31-37° and cohesion from 0.5 kN m^{-2}.

The ring shear tests have provided residual cohesion values of 5-7 kN m^{-2} and residual friction angles of 4-13° for the same range of normal stresses as the direct shear testing. The very low residual friction angles are unable to be accounted for at this time and a rigorous analysis of testing technique and procedures is currently under way to establish the credibility of the results. Comparative data for silts are scarce in the literature, yet it is still surprising that such behaviour has not been observed before. The results as they stand would indicate that the lake sediments exhibit a moderate to sensitive behaviour that increases with normal stress/depth.

At this point the lake sediments have been deemed to have a high colloidal erodibility, activity, and sensitivity which contributes to the siltation problems evident in the field. Slumping is the only failure mechanism observed. These indicators pose problems for land use in the area (predominantly lifestyle blocks) and may aid land management should the area become developed.

**CONGRATULATIONS
TO
1998 NZ GEOTECHNICAL
SOCIETY STUDENT
PRIZEWINNERS**

JUSTIN HARRISON
(SOUTHERN REGION)

&

DARREN WISE
(NORTHERN REGION)

COURSES OF INTEREST TO GEOTECHNICAL PROFESSIONALS, 1999

The University of Waikato offers courses of interest to geotechnical professionals predominantly within the Department of Earth Sciences as part of B.Sc. or M.Sc.(Hons) degree programmes, or towards a PGDipSc. The Department acknowledges the fundamental importance of the Earth's physical environment and its resources - landscapes, rocks, sediments, water, oceans and climate - for New Zealand's development, and this philosophy is maintained as a central and integrating theme of both the teaching and research of the Department. Students are encouraged to undertake courses covering a wide range of Earth Sciences in order to develop a general appreciation of the environment, with more advanced training in some particular branches of the discipline included at higher levels. The aim is to produce versatile graduates with the ability to integrate ideas across a wide range of the sciences associated with the natural physical environment.

Two courses are particularly aimed at careers in the geotechnical field:

- **0772.351Y Engineering Geology** - the nature of soil and rock strength; means of determining strength in the field and laboratory; processes of erosion on slopes and the mechanics of slope instability; stability analysis; site investigation techniques; hillslope development and means of managing hillslope processes.
- **0772.551Y Rock and Soil Mechanics** - the principles and practice of rock mechanics, especially the strength and rheology of intact rock, the development of fractures, and analysis of the behaviour of discontinuous rock masses; the principles of soil mechanics; case studies of site investigation projects, specific materials, and distinctive environments.

Other courses of potential interest to geotechnical professionals include:

- Hydrology papers **(0772.341Y and 0772.541Y)** deal with both surface hydrological processes and groundwater, including groundwater modeling, pumping test theory, runoff generation, river flow modeling, and aspects of water pollution.
- A variety of papers in coastal studies and environmental issues::

  0772.343A Coastal Geomorphology and Management - geomorphic evolution of coastal features; assessment of coastal hazards; impacts of sea level rise; dredge spoil disposal; methods of coastal protection.

  0772.344Y Coastal Oceanography and Engineering - methodologies for quantifying coastal processes are developed and applied to coastal management issues.

  0772.543Y Coastal Sedimentation and Environment - principles behind monitoring programmes imposed under RMA 1991; coastal hazard analysis and planning; dredge spoil monitoring; beach renourishment; issues relating to port design and sea walls.

  0772.544Y Numerical Modeling of Coastal Processes - the development and use of numerical models in the investigation of coastal processes, including field data collection, model calibration, modeling techniques and applications.

  0772.545Y Marine Instrumentation and Data Analysis - introduction to a variety of state-of-the-art marine instruments and associated data analysis.

  0085.521Y Environmental Evaluation - an interdisciplinary course based on a waste management theme considering the technical, economic and political considerations of waste management, and developing skills involved in research related to the environment.

  0777.541Y Environmental Technology - Water and Wastewater - an interdisciplinary course which integrates physical, biological, and chemical aspects of wastewater processing technology and discharges.

ENQUIRES

Vicki Moon
Department of Earth Science, University of Waikato
Private Bag 3105, Hamilton
Fax (64-7) 856 0115 ; Tel. (64-7) 838 4239

GRADUATE STUDY IN GEOTECHNICAL ENGINEERING

The following geotechnical engineering papers are available for graduate degrees in the Department of Civil and Resource Engineering at the University of Auckland. The degrees available are Diploma in Engineering, Master of Engineering Studies, and Master of Engineering. Both the Diploma and Master of Engineering Studies degrees require 14 points for completion, the Master of Engineering is 21 points of which 14 are for the thesis. A full time course of study for one year is equivalent to 14 points, part time study for the Diploma and Master of Engineering Studies is possible.

Students may choose papers from the graduate papers in the School of Engineering and from the list of undergraduate final year electives (it is also possible to take papers from other departments in the University). Brief descriptions of the geotechnical engineering papers available are as follows (not all the papers are taught each year):

Slope Engineering (1 point) (Final year elective)
Geological appraisal of slope behaviour and site investigation techniques for stability assessment. Further work on the shear strength of soil and rock masses. Mechanisms of slope failure and the influence of groundwater. Evaluation of slope stability and assessment of the risk of failure. Techniques for the remediation of unstable slopes and for enhancing existing stability. Lessons from slope engineering case histories. Instrumentation for monitoring slope performance.

Foundation Engineering (1 point) (Final year elective)
Engineering geological inputs to foundation engineering. Common procedures for site investigation. Shallow and deep foundations: selection, ultimate and serviceability limit state design, construction, and performance. Design of retaining structures. Introduction to the effects of earthquakes on foundations.

Groundwater Hydrology (1 point) (Final year elective)
Theory and application of the principles governing groundwater movement. Ground water modelling techniques. Well hydraulics and contaminant transport. Groundwater quality and the management of groundwater resources.

Geomechanics 3 (2 points) (Final year elective)
(Geomechanics 1 and 2 are core papers in the second and third years of the undergraduate degree).
Triaxial testing with the measurement of pore water pressure. Effective and total stress paths during drained and undrained tests in laboratory apparatus and field loading processes. Dynamic soil behaviour, site response during earthquakes, dynamic response of a foundation on an elastic soil. Seepage for cases with an unconfined flow boundary. Rates of consolidation for one and two dimensional flow paths; radial consolidation with application to sand drains; finite difference calculations for the consolidation of layered deposits. Earth dam design: flow net construction; piping resistance and filters; stability analysis for the end of construction, steady state seepage, and rapid drawdown conditions; behaviour during earthquakes; instrumentation for performance monitoring.

Excavation Engineering and Design (2 points) (Final year elective)
Theoretical, practical, and environmental aspects of ground excavation: rock cutting, ripping, drilling, and other excavation techniques. Properties of commercial explosives, mechanics of blasting, and blast design. Surface and underground excavation design: stress analysis, stability and support requirements.

Applied Geomechanics (2 points) (Graduate paper)
Application of the principles of soil mechanics, rock mechanics, and engineering geology to practical problems in civil engineering. In recent years the material taught in this paper has centred around techniques for the earthquake resistant design of foundations.

Geomechanics Seminar (2 points) (Graduate paper)
Engineering properties of residual soils. Seepage including the method of fragments; groundwater flow; dewatering techniques. Ground improvement techniques; reinforced earth; use of geosynthetics. Statistical and probability applications in geotechnical engineering.

Earthquake Engineering (2 points) (Graduate paper)
Fundamentals of seismology including earthquake waves, magnitudes and felt intensities. The damaging effects of earthquakes upon land and human constructions. Study of some relevant historical earthquakes.

Strong motion earthquakes and the response of land and buildings. Fundamentals of earthquake resistant design of engineering structures.

Advanced Rock Mechanics (2 points) (Graduate paper)
Theoretical and applied treatment of the constitutive behaviour of rock masses with particular attention to failure analysis and numerical modelling. The paper has a number of project-based modules as well as a laboratory/field project in some area of rock measurement.

Projects (2 and 4 points) (Graduate papers)
Project work may comprise up to 6 points for the Diploma and Master of Engineering Studies degrees. The student prepares a written report on the outcomes of independent work. Aspects of the design, analysis, performance, and /or evaluation of a geotechnical problem or process may be studied.

ENGINEERING GEOLOGY COURSES AT UNIVERSITY OF AUCKLAND

The following courses are offered in the Department of Geology at the University of Auckland, in the field of Engineering Geology.

These are all one semester 2 point papers, but they are each of different format and timing. However, they are all of the same overall time commitment. One is on a regular series of one hour long timetabled slots 5 times a week each week of the 2nd semester (372). Another is once a week for 2 to 3 hours each week of the 1st semester (771). The other paper is entirely field based during the last 2 weeks of February and involves 6 or 7 days of fieldwork with 3 or more days of writing up and reporting (701).

435.372 Case Histories in Engineering Geology
Examples of the application of geological principles and practice to a range of engineering sites especially in weak rock, volcanics and active regions. Practical exercises in standard techniques use actual localities in Auckland. (Final year BSc and BE 2 pt 2nd semester paper).

435.771 Engineering Geology
Seminars, workshops and a field and lab. exercise on a selection of topics from the literature and local experience. (First year MSc 2pt 1st semester paper).

435.701 Special Topic in Engineering Geological Mapping.
For Graduates, final year BSc and BE students, and Geotechnical Professionals. Commencing date ; Friday 12th February 1999. A 2 week long field based course in the 2nd half of February. Auckland examples in a range of rocks and soils with noted geotechnical problems will be studied, mapped and reported upon using current techniques.

This course is designed to be a hands-on experience in practical methods of mapping outcrops and geomorphology for geotechnical purposes. There is a 2 point paper and will consist of;-

- 6 to 7 days of fieldwork, some of which can be done at times that may vary to suit the different schedules of people enrolled in the paper. Geotechnical mapping and field characterisation of rocks, soils and defects. A range of lithologies will be studied, including soft weak rock, residual soils, Quaternary deposits and volcanics (including rhyolitic soil masses) in the Auckland region
    - Landslide mapping.
    - Photogeology.
    - Construction of maps and sections.

Assessment will be based entirely on the mapping assignments.

Fieldwork can be completed by February 26th, but assignments will be submitted shortly after that date. Transport is to be arranged amongst the group or by individuals enrolled in the paper. All localities are in easy reach of Auckland, from 30 minutes to one and a half hour's drive.

This paper is designed to give experience in the art of observation and recording of the engineering geological characteristics of a variety of rocks and soils of the Auckland region and their geomorphic expression. We will examine the potential of these deposits to contribute to hazardous situations and challenging geotechnical conditions.

For enrolment forms please contact the following; -

Admissions and Enrolment Office
The University of Auckland
Private Bag 92019
Auckland
Phone; 3737599 x 5013 or 5025
e-mail ;enrol@auckland.ac.NZ

Course Coordinator and Instructor; -
Warwick Prebble, Department of Geology
University of Auckland
Private Bag 92019
Auckland
Phone; 3737599 x 7591
Fax, 3737435
e-mail ; w.prebble@auckland.ac.nz

BY THE 1st of DECEMBER 1998.

A small additional fee will be charged for late enrolments

UNIVERSITY OF CANTERBURY

GRADUATE STUDY IN GEOTECHNICAL ENGINEERING

Refer to Kevin McManus, School of Engineering, Civil Department, Tel. (03) 366 7001, E-mail: k.mcmanus@canterbury.ac.nz

GRADUATE STUDY IN ENGINEERING GEOLOGY

Refer to Dr David Bell, Geology Department, Tel. (03) 366 7001, E-mail: d.bell@canterbury.ac.nz

Have You Changed Your Personal Details??

If you have moved, changed employer, phone number or e-mail address, please contact the Management Secretary,

Debbie Fellows
Tel: - 09 8177759
Fax: - 09 8177035
Email: - dfellows@xtra.co.nz

Photo Competition
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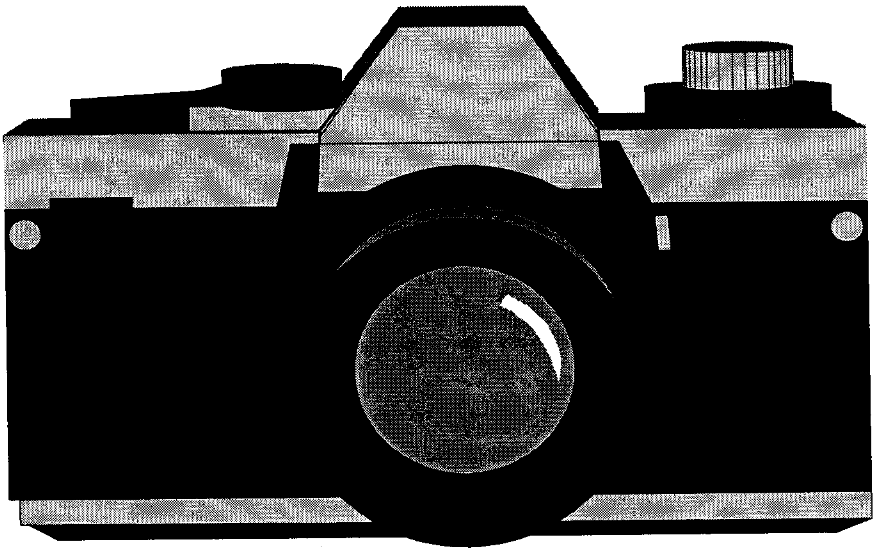


We want: - Photos that highlight the art and science of geotechnical practice in NZ.

The photo can demonstrate any aspect of the profession, from important geologic or geomorphologic phenomena, through to major structures which have significant geotechnical features.

Have your winning photo printed in Colour in the next edition of the Geomechanics News and put on the Society Web Page.

How to enter:-
Send your entry by 31 March 1999 to Management Secretary, NZ Geotechnical Society, P O Box 12 241, Wellington. Clearly mark your entry with your name and caption.

Conditions of Entry: -

1. Only amateur photographers may enter.
2. Photos must be taken by the entrant.
3. No computer generated pictures.
4. Photos will not be returned.
5. Any Photos received may be published in subsequent Society publications.
6. Winning entries will be final and no correspondence will be entered into
7. NZ Geotechnical Society members only may enter

ADVANCE NOTICE OF WORKSHOP

The New Zealand Geotechnical Society will be sponsoring a 2 day workshop on Quantitative Risk Assessment of Soil and Rock Slopes.

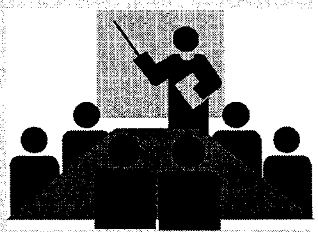
The workshop will be run by Prof. Robin Fell and Garry Mostyn of the University of New South Wales.

The date and location are yet to be confirmed but it is expected to be early 1999 in Auckland. Places for the workshop will be limited and are likely to available on a first come first served basis.

If you are interested in attending please register your interest with the society Management Secretary.

Contact Details:-

Debbie Fellows
Tel:- 09 8177759
Fax :- 09 8177035
Email:- dfellows@xtra.co.nz



ERRATA : NZ GEOTECHNICAL SOCIETY GUIDELINES FOR SOIL & ROCK DESCRIPTION

The following two tables are suggested replacements for those currently included in the Society Guidelines for the Description of Soil & Rock.

TABLE 2.8 : SOIL COHESIVE STRENGTH

TERM (COHESIVE)	DIAGNOSTIC FEATURES	UNDRAINED SHEAR STRENGTH, Su (kPa)	UNDRAINED COMPRESSIVE STRENGTH. q_u (kP
Very soft	Exudes between fingers when squeezed	< 12	< 25
Soft	Easily indented by fingers	12 – 25	25 – 50
Firm	Intended only by strong finger pressure	25 – 50	50 – 100
Stiff	Indented by thumb pressure	50 – 100	100 – 200
Very stiff	Indented by thumbnail	100 – 200	200 – 400
Hard	Difficult to indent by thumbnail	200 - 500	400 – 1000

TABLE 2.10: SPT & SCALA PENETROMETER RESULTS

TERM (COHESIONLESS)	SPT'N' VALUE (NO. OF BLOWS/ 300 MM)	SCALA PENETROMETER (NO. BLOWS/ 100 MM)	SCALA PENETROMETER (NO. BLOWS/ 100 MM)
Very dense	> 50	> 17	> 11
Dense	30 – 50	7 – 17	6.5 – 11
Medium dense	10 – 30	3 – 7	2 – 6.5
Loose	4 – 10	1 – 3	1 – 2
Very loose	0 – 4	0 – 2	0 - 1

A comprehensive review of these guidelines is due to be undertaken next year. Please address any comments or suggestions for this upgrading to:

The Secretary of the Society
Debbie Fellows
email: dfellows@xtra.co.nz
tel: (09) 817 7759
fax: (09) 817 7035

FUNDING SOUGHT TO UPDATE CODE FOR URBAN LAND SUBDIVISION

One of the issues under discussion at a recent meeting of the Territorial Authorities/Civil Engineering Industry Advisory Group was whether or not to fund a revision of the ***Code of Practice for Urban Land Subdivision.***

Standards New Zealand (SNZ) put forward a proposal to revise NZS 4404:1981 – *Code of Practice for Urban Land Subdivision*, on the basis that important legislative changes needed to be incorporated into the standard. Since the last publication of the standard in 1981 there have been a number of important legislative developments including the implementation of the Resource Management Act and the Building Act.

It was agreed it was appropriate to revise NZS 4404, subject to funding being found. Over the next month a number of different bodies will be approached, including Local Government, the Ministry for the Environment and Transit New Zealand, for funding of the standard. If adequate funding is not found, SNZ will have to notify the withdrawal of the standard to all interested parties.

DOWN TO EARTH BUILDINGS

Earth building was once perceived to be the province of cranks and eccentrics, according to one of New Zealand's leading practitioners in the field, Warkworth architect Graeme North. Now, with the help of three earth-building standards, newly out from Standards New Zealand (SNZ), earth building seems set to join the building mainstream.

Graeme North was one of nine members of a technical committee facilitated by SNZ, some of whom have worked together since 1991 to bring the standards into being –" the gestation was a long one but giving birth to the standards was worth the time and effort involved", he says.

The standards are the first of their kind in the world, in that they are performance-based rather than merely prescribing the materials to use, which was previously the case. Earth building has been around for centuries in all parts of the world but until now there were no comprehensive guidelines available. Architects, builders, enthusiastic amateurs, or anyone who has nursed a secret ambition to build an earth-house, will now find it a much easier task to do so. One reason, in particular, is that they are designed specifically to comply with the New Zealand Building Code.

Previously when building an earth house the architect or builder often found that obtaining a building consent was not an easy task, due to general lack of familiarity with the building processes and materials involved. Consequently when clients made enquiries to their builder or architect about building an earth house, they were often gently discouraged, North says. These standards, however, are intended to be cited as a means of compliance with the performance based Building Code. If a design complies with a cited document gaining a building consent for it becomes a lot easier, North says.

The three documents consist of *NZS 4298 Materials and Workmanship for Earth Buildings*, *NZS 4297 Engineering Design of Earth Buildings* and *NZS 4299 Earth Buildings Not Requiring Specific Design*. The first of these documents, NZS4298 details materials and workmanship requirements for the use of unfired earth in the form of adobe, pressed earth brick, rammed earth or poured earth, and is intended to be used in combination with either of the other two standards.

Adobe brick is perhaps the building technique most commonly associated with earth houses and traditionally involved tamping together a mix of subsoil (containing a relatively high clay content), and fibre (straw, grass roots) and forming a brick using a mould. These would then be allowed to air dry and then formed into walls using a mortar of the same material. Nowadays builders often opt for more sophisticated methods and introduce modern materials such as cement, asphalt and use mechanical compressors to make the bricks – or even buy them ready-made. In recent years several New Zealand companies have sprung up selling ready-made earth bricks in the wake of the ever-increasing popularity of earth buildings.

Graeme North designed his first earth house in 1971 on the tail end of the hippy era and for the next twelve years or so demand for earth built houses was relatively quiet. However, for the last fifteen years demand has increased year by year and current production of earth-built houses is approximately forty a year. That number is likely to increase exponentially with the publication of these standards he says. Especially, as the standards encapsulate the knowledge collected by the New Zealand Earth Building Association over 25 years, of which North used to be chair.

So why do people chose to live in earth houses? Clearly the chief resource required – earth – is in plentiful supply and is therefore an environmentally sound option. The actual building technique requires thick walls of approximately 300 mm thick. The thermal mass combined with appropriate design, ensures heat is retained in winter and in summer keeps them cool, providing a cost-effective means of maintaining an even indoor temperature. The materials used have a relatively low level of toxicity in comparison to many modern materials and are fire-resistant. Many people are under the misconception that earth-built houses are cheap, which is not necessarily the case, North says. 'The thickness of the walls and high labour content can push up the price, but walls are only 10-15% of the overall building cost so quotes for earth-built houses are generally quite competitive. You are also buying a quality material". Most of the buildings over a hundred years old in New Zealand which are still standing are earth buildings, ranging from Pompalier House in Russell to St Bathan's pub in Otago.

Earth walls are not as strong as timber or steel and are usually used for one storey high dwellings only. However, these standards have been drafted keeping earthquake considerations in mind. Some of the best earthquake engineers in the world are in this country, North says. Therefore there is no reason why public buildings should not be made out of earth, citing the tradition of earth-built mosques in the Middle East, adobe buildings in New Mexico, multi storey apartment buildings in Europe and resort hotels in Australia.

Another member of the committee engineer Richard Walker, supports North's views on the earthquake risk of earth houses. "These standards are very similar to ones for masonry and concrete. As long as you take certain precautions, such as having concrete foundations, reinforcement in the walls, good timber bond beams and ceiling diaphragms (diagonal boards tying the others together) you shouldn't have a problem – after all some of the oldest buildings in New Zealand are earth houses."

Having lived in an earth house for the last twelve years Walker says he wouldn't live in any other kind. He points out that the fact that the building is made of natural materials means that once they have completed their purpose they can be allowed to biodegrade naturally, thus completing the cycle.

For further information contact Standards Development Consultant Ian Brewer: 04 495 0911, e-mail ianb@standards.synet.net.nz

SOIL STRENGTH REDUCTION FACTOR, Φ_G

The strong turnout for Mick Pender's Auckland Branch presentation earlier this year on limit state factors for soils, and some recent personal communication with a few structural engineers, suggests that there is uncertainty with this issue. Such uncertainty is ably assisted by various New Zealand codes. The New Zealand loadings code is at variance with other (NZ) Codes as outlined in Table 1 below.

Table 1: Variation of Strength Reduction Factor for NZ Foundation Design

Code	NZS 4203	NZBC	TNZ Bridge Code	AS 2159
Strength Reduction Factor, Φ_G	0.6	0.5	0.40 to 0.45	0.40 to 0.55

Notes:
1. NZ4203 = NZ Loadings Code (1994) clause 6.6.3
2. NZBC = NZ Building Code (1992) Verification Method B1/VM4, Table 2
3. TNZ Bridge Code = Transit NZ Bridge Code (1994), Table 4.1
4. AS2159 = Australian Piling Code (1995), Table 4.1
5. Φ_G based on assessment from borelog data

Starting from an agreed point, i.e. working stress design, $\text{FOS} = \dfrac{\text{Ultimate Capacity, } Q_U}{\text{Working Load, } W_L} \geq 3$

and considering this working stress relationship in terms of "pseudo limit state factors", we get

Average Load Factor, $(DL+LL)_{GW} = 1.0$
Strength Reduction Factor, $\Phi_{GW} \geq 3.0$

where G denotes geotechnical/soil and W denotes working stress condition.

For 'true' Limit State design, and where DL $\approx$ LL (a common condition), the

$$\text{'Average' Load Factor} = [1.2\,(DL) + 1.6\,(LL)] \times \tfrac{1}{2} = 1.4$$

$$\text{'Average' Limit State Load} = 1.4 \times W_L$$

Therefore, 'Average' (True) Limit State Strength Reduction Factor $\Phi_G = \dfrac{1.4\,W_L}{3.0\,W_L} = 0.46$

As can be seen from Table 1 above, $\Phi_G = 0.46$ compares well with all the codes mentioned, except for NZS 4203.

Similarly, for a seismic foundation load case, where a FOS $\geq$ 2.0 is normally adopted for working stress conditions, and DL, LL and EQ load factors are all 1.0, the

'Average' (True) Limit State Strength Reduction Factor, $\Phi_{GEQ} = \dfrac{1}{2.0} = 0.50$

For 'average' foundation conditions based on borehole data and where LL $\approx$ DL, it is concluded that a Φ_G reduction factor $\not>$ 0.5 is appropriate.

However, this approach does not allow for flexibility when greater confidence can be gained from better soils or foundation investigation and testing. If greater confidence is obtained then the strength reduction Φ factor could be increased. This is the approach used in the Australian Piling Code (see ϕ_g in AS 2159, Table 4.1 extract below). A similar approach is suggested for revisions to NZBC and NZS 4203.

Stephen Crawford
Geotechnical Engineer

TABLE 4.1

RANGE OF VALUES FOR GEOTECHNICAL STRENGTH REDUCTION FACTOR ϕ_g

Method of assessment of ultimate geotechnical strength	Range of values of ϕ_g
Static load testing to failure	0.70–0.90
Static proof (not to failure) load testing (NOTE 1)	0.7–0.90
* Dynamic load testing to failure supported by signal matching (NOTE 2)	0.65–0.85
Dynamic load testing to failure not supported by signal matching	0.50–0.70
Dynamic proof (not to failure) load testing supported by signal matching (NOTES 1 and 2)	0.65–0.85
Dynamic proof (not to failure) load testing not supported by signal matching (NOTE 1)	0.50–0.70
Static analysis using CPT data	0.45–0.65
Static analysis using SPT data in cohesionless soils	0.40–0.55 ← BH data
Static analysis using laboratory data for cohesive soils	0.45–0.55
Dynamic analysis using wave equation method	0.45–0.55
Dynamic analysis using driving formulae for piles in rock	0.50–0.65
Dynamic analysis using driving formulae for piles in sand	0.45–0.55
Dynamic analysis using driving formulae for piles in clay	Note 2
Measurement during installation of proprietary displacement piles, using well established in-house formulae	0.50–0.65

NOTES:

1 ϕ_g should be applied to the maximum load applied.

2 Signal matching of the recorded data obtained from dynamic load testing should be undertaken on representative test piles using a full wave signal matching process.

3 Caution should be exercised in the sole use of dynamic formulae (e.g. Hiley) for the determination of the ultimate geotechnical strength of piles in clays. In particular, the dynamic measurements will not measure the 'set-up' which occurs after completion of driving. It is preferable that assessment be first made by other methods, with correlation then made with dynamic methods on a site-specific basis if these latter are to be used for site driving control.

4 For cases not covered in Table 4.1, values of ϕ_g should be chosen using the stated values as a guide.

TABLE 4.2

GUIDE FOR ASSESSMENT OF GEOTECHNICAL STRENGTH REDUCTION FACTOR (ϕ_g)

Circumstances in which lower end of range may be appropriate	Circumstances in which upper end of range may be appropriate
Limited site investigation	Comprehensive site investigation
Simple method of calculation	More sophisticated design method
Average geotechnical properties used	Geotechnical properties chosen conservatively
Use of published correlations for design parameters	Use of site-specific correlations for design parameters
Limited construction control	Careful construction control
Less than 3% piles dynamically tested	15% or more piles dynamically tested
Less than 1% piles statically tested	3% or more piles statically tested

Ultimate limit state design of shallow foundations – load factors and strength reduction factors

M J Pender

The ultimate limit state design of foundations requires us to move away from the traditional factor of safety approach. There is no great difference in principle, one simply has to accommodate oneself to some different terminology. In the factor of safety approach the designer usually exercises judgement in deciding on the appropriate value for the factor of safety. At first glance the ultimate limit state approach to foundation design seems more prescriptive, even though a range of values has been presented for the strength reduction factor. This note compares the required foundation width for the factor of safety design against bearing capacity failure and that obtained using the ultimate limit state approach.

In the past in geotechnical work we have not made a formal distinction between dead and live load. The NZ Loadings Standard (NZS4203:1992) makes a distinction though, and prescribes the following factoring of loads:

Dead load only (D):	1.4D
Combinations of dead (D) and live load (Q):	1.2D + 1.6Q.

Ultimate limit state design of foundations replaces the factor of safety with a strength reduction factor, Φ. Thus in designing a foundation against bearing failure the following inequality must be satisfied for a design to be acceptable:

$$\Phi \times \text{(ultimate bearing strength of the foundation)} \geq \text{sum of the factored loads}$$

(Note that the units of the quantities on both sides of this inequality are forces. Thus the ultimate bearing strength is the ultimate bearing pressure times the effective area of the foundation).

The revision of the Building Industry Authority approved Document B1/VM4 presents a range of values for the strength reduction factor for ultimate bearing capacity of shallow foundations. The range given is 0.45 to 0.6, it was put forth after noting some Australian suggestions about limit state design. In this note the required width of a strip foundation on sand calculated using the ultimate limit state approach is compared with that given using a bearing capacity factor of safety of 3.0.

The required of a strip foundation on the surface of a dry sand deposit was calculated for friction angles between 20° and 40°. The calculations were done (i) using unfactored loads and a factor of safety of 3.0, (ii) using factored loads and a strength reduction factor of 0.45, and (iii) using factored loads and a strength reduction factor of 0.60. The results are plotted in Figs. 1 and 2.

The data plotted in Fig. 1 was calculated for the range of friction angles and a fixed split between dead and live load (D = 50 kN and Q = 100 kN). The plot shows that for the range of friction angles the required width for a factor of safety of 3.0 lies between the curves for the two values of the strength reduction factor. Thus foundation widths calculated for Φ equal to 0.45 and 0.60 are bounds on the width for a factor of safety equal to 3.0. Furthermore the differences between the three widths are not great. Figure 2 presents results for a fixed friction angle (30°), a total load of 150 kN, and differing splits between dead load to live load. This graph shows that when dead load dominates a strength reduction value of 0.45 gives a foundation width closest to that with a factor of safety of 3.0, and when live load dominates a strength reduction factor of 0.60 gives a foundation width closest to that for a factor of safety of 3.0. There is a kink in both the strength reduction factor lines as the live load ratio approaches zero. This reflects the change in the factor for dead load – 1.2 when associated with live load and 1.4 when there is no live load.

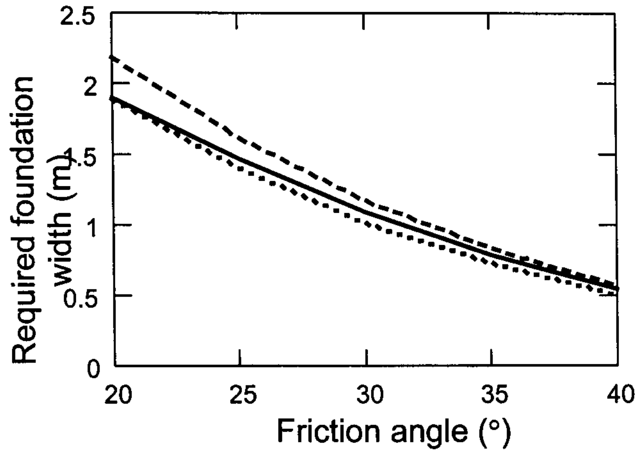


Figure 1: Required foundation width with various ultimate limit state requirements. Solid line: bearing capacity factor of safety = 3.0, upper dotted line strength reduction factor = 0.45, bottom dotted line strength reduction factor = 0.6. (For all cases the dead load is 50 kN and the live load is 100 kN.)

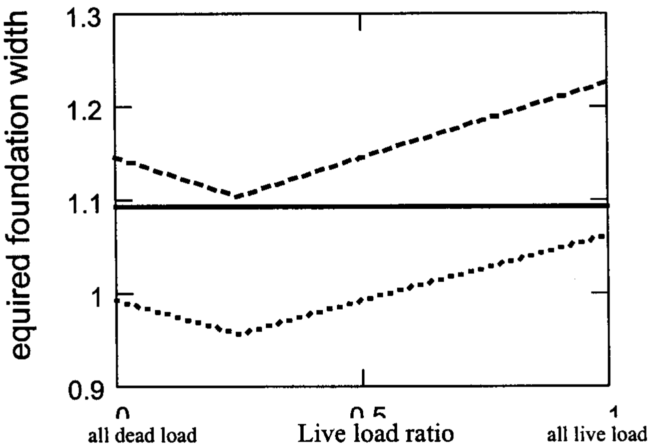


Figure 2: Required foundation width with various ultimate limit state requirements. Solid horizontal line: bearing capacity factor of safety = 3.0, upper dotted line strength reduction factor = 0.45, bottom dotted line strength reduction factor = 0.6. (For all cases ϕ = 30°, and the sum of the dead load live load is 150 kN.)

In conclusion then, these calculations illustrate:

- that when bearing capacity calculations are done using the ultimate limit state approach with factored loads, the required foundation width is not much different from that obtained using unfactored loads and the traditional bearing capacity factor of safety of 3.0.
- that the strength reduction factor of 0.45 can be associated with cases where dead load dominates, whilst the value of 0.60 gives the result closest to the factor of safety of 3.0 when live load is dominant.

RETRO-FITTED LANDFILL GAS WELLS

The Redvale Landfill north of Auckland now contains more than 1,000,000 tonnes of waste and has been open for more than 5 years. Gas well installation has progressed according to the designer's original plan, which comprised vertical gas wells 100 m apart. Recently, site management has added a number of features to the system e.g. additional vertical and horizontal gas wells to increase gas collection efficiency and reduce odours. Retro-fitting of additional vertical gas wells was undertaken in a completed area of the landfill where landfill gas odour was often detected. In the affected area, the original design showed wells more than 100 m apart.

Retro-fitting of vertical gas wells involved bottom-driving a steel casing down into the waste to create a void for the gas well. This method was selected to minimise the exposure of old waste and thereby to avoid any odour event. Odour control is one of the most important issues at the landfill. In this regard, deep auger drilling was considered unacceptable. The chosen method was also safer, quicker and quieter.

The casing was 18 m long, 0.3 m diameter, with a 1.2 m gravel plug. The casing was bottom-driven (internally) with a ram of approximately 1.1 tonnes falling 8 m. Starter holes were pre-augered 0.45 m diameter to 2.5 m, with odour neutralising sprays operating from hand-held wands. Driving was expected to be straightforward, as the waste was known to be mostly municipal waste. Large items such as a tree stumps or car bodies larger than 1 m generally were not taken into the landfill. It was found that the driving became more difficult below 15 m and the vibro struggled to extract the casing. This was judged to be due the pile "veering off" the initial path line with depth and tightening due to its curvature. After extracting the casing, pre-assembled slotted PVC pipe 150 mm diameter was lowered into the hole, which remained open to the full depth. The riser pipe string had a collar at 6 m below ground level that retained bentonite pellets as a seal up to ground surface. The annular void below 6 m was not filled, which allowed the compressed waste around the hole to partly relax and allow the gas to flow.

Based on the amount of gas produced and the reduction in landfill gas odour, the wells have been considered successful.

Bruce Horide
REDVALE LANDFILL ENGINEER

LANDSLIDES, INVESTIGATION AND MITIGATION

Special Report 247
Transportation Research Board
Editors: Turner and Schuster, 1996
ISBN 0-309-06151-2, ISBN 0-309-06280-X (pbk)

Landslides, investigation and mitigation, is the latest in a series publications by the US Transportation Research Board (TRB) on the topic of landslides.

In the geotechnical literature there are many references to slope stability and landslides, ranging from identification, investigation analysis to remedial control works. However, there have been few publications that bring together this wealth of information in a single source. This lack of a single source publications on the wide spectrum of landslides was first addressed by the TRB in 1958 with the publication of Special Report 29, Landslide and Engineering Practice. In 1978 the TRB published Special Report 176 – Landslides, Analysis and Control. Special Report 176 was reprinted a number times and translated in several languages becoming a reference textbook on many bookshelves.

In 1989 the TRB polled a number of its members within geotechnical and geological disciplines with regard to revising Special Report 176. A number of changes and new additions were identified to address the large amount of new technical information available. Under the Chairmanship of Keith Turner of the Colorado School of Mines, a new committee was set up to revise chapter outlines and identify authors, and compile Special Report 247 – Landslides, Investigation and Mitigation. The resulting publication containing 25 chapters compiled by 30 authors from USA, Canada and the Netherlands. The list of authors includes many notable names including Robert Schuster, David Varnes and Philip Lambe.

The new look TRB landslides publication is set out in five parts as follows:

Part 1: Principles, Definitions and Assessment

- Introduction
- Socio-economic Significance of Landslides
- Landslides Types and Processes
- Landslide Triggering Mechanisms
- Principles of Landslide Hazard Reduction
- Landslide hazard and Risk Assessment

Part 2: Investigation

- Organisation on Investigation Process
- Slope Instability Recognition, Analysis &Zonation
- Surface Observation and Geologic Mapping
- Subsurface Exploration
- Field Instrumentation

Part 3: Strength and Stability Analysis

- Soil Properties and Their Measurement
- Soil Slope Stability Analysis
- Rock Strength Properties and their Measurement
- Rock Slope Stability Analysis

Part 4: Mitigation

- Important Considerations in Slope Design
- Stabilisation of Soil Slopes
- Stabilisation of Rock Slopes

Part 5: Special Cases and Materials

- Residual Soils
- Colluvium and Talus
- Shales and Other Degradable Materials
- Hydraulic Tailings
- Loess
- Soft Sensitive Clays
- Permafrost

A broad spectrum of topics related to Landslides is covered with a large number of case studies used to illustrate various points, including reference to the Clutha Valley landslides. The subjects discussed are dealt with concisely, but in sufficient detail to enable a good understanding of the principles and methodology for landslide identification, investigation, assessment and remedial design. For those in search of greater detail, references are made to a wide range of literature in support of the topics covered. In fact combining all of the reference lists from each of the chapters makes this book a valuable source of geotechnical literature on landslides.

This publication on landslides is, in my option, one of the most informative and readable geotechnical books available. I am not aware of any other textbook that deals with the topic as comprehensively. This book, I believe, should have wide appeal providing an invaluable text book for students and an often fingered reference book on any well-respected engineering geologist's or geotechnical engineer's book shelf.

Doug Johnson
ASSISTANT EDITOR

SWEDISH GEOTECHNICAL INSTITUTE WEBSITE & GEOTECHNICAL LITERATURE DATABASE

A Brief Review

In the June 1998 edition of ISSMGE News, the Swedish Geotechnical Institute (SGI) Geotechnical Literature Database was advertised to ISSMGE

members. The literature database can be accessed via the SGI Internet web site (www.sgi.geotek). To promote use of the database, access was free of charge up until the end of November 1998. So, one lunch hour we 'paid' a visit to the SGI.

The SGI web site home page is in Swedish but an English version is available at the press of the button. The home page provides links to many pages detailing the activities, information and news about the SGI including the Service. Access to the Literature Service requires a user name and pass word. Once inside, a range of services is available including:

- Listings of the latest geotechnical literature in the SGI library
- Literature search of the SGI and companion external databases
- Assistance with loan and copies
- Enquires on geotechnical literature
- Sales of publications of the SGI

To register a user name and password and access the SGI literature database will now cost ISSMGE members $US250 about $NZ500 and non-members $US400 (about $NZ800 per annum).

The literature database has access to over 50,000 references within SGI's library as well as access to over 90,000 geotechnical references available via the Swedish central library. The database works using a number of search criteria defined by key words combined with Boolean operators *and*, *or* & *not*. Searches can be made using keywords for subject, geographical references, author, publication or year.

The database contents are distributed as follows
- Foundation and Reinforcement (26%)
- Soil and Rock Mechanics (25%)
- Properties of soils and rock (20%)
- Site investigations (7%)
- Environmental geotechnics (7%)
- Geology (7%)
- Energy, Snow and Ice Mechanics etc (8%)

The majority of references listed in the database (69%) are available in English.

The database search page is well set out and easy to follow. Output from a search is listed in a series of pages. Each page lists a separate publication title, author, publication title, keywords and an abstract. Publications are listed in date order, the most recent being first.

Search's on general topics (e.g. clays), as expected, get a large number of hits. In scanning over the output, all of the main geotechnical publications (e.g. Geotechnique, Canadian Geotechnical Journal) and conference proceedings were listed. Narrowing down the keyword selections, enabled a quick listing of selected publications on a range of topics entered ranging from slope stability analysis, retaining wall design, probabilistic analysis and horizontal drains. Specific searches on less main-stream subjects, such a cliff top stability, proved less successful.

Based on our limited use of the site and comparing the output of our searches with the results from other commercial databases, the SGI Geotechnical Literature Database appears to provide a comprehensive database of geotechnical literature. The database is easy to access, use and is relativity quick. The database output is comprehensive, albeit a little cumbersome having to move through the output page by page.

For ISSMGE members who undertake regular literature searches, the SGI Geotechnical Literature Database is, in our opinion, worth considering. To contact the SGI about use of the database, e-mail info@Geotek.se.

Doug Johnson
ASSISTANT EDITOR

- **A History of the Development ofAggregates for Roading Purposes**

  F G Bartley

- **Field and Laboratory testing of volcanically derived soils**

  M J Pender, V M Meyer, T J Larkin, L D Wesley, G C Duske

- **Correlations Between Cone Penetration Resistance and Standard Penetration of some New Zealand Volcanic and Alluvial Deposits**

  A S Ahmed-Zeki & Peter B C Bosselmann

- **Probability and Consequences of the Next Alpine Fault Earthquake**

  M D Yetton, A Wells, N J Traylen

A HISTORY OF THE DEVELOPMENT OF AGGREGATE FOR ROADING PURPOSES

F G Bartley BE FIPENZ

1 INTRODUCTION

This paper reviews the developments that have occurred over the last forty or so years in the use of unbound granular aggregates for the construction of road pavements.

2 EARLY BASECOURSE SPECIFICATIONS

Prior to the Second World War, most pavements were unsealed and a dense graded, clay rich aggregate was used to form a dense pavement with a relatively smooth top surface. A similar material was used as a basecourse for sealed pavements.

The Main Highways Board grading envelope for a particle size distribution curve of a basecourse aggregate published in 1938, is shown in Fig 1. (Bartley, Ferry and Major 1988).

For the purposes of this paper the nature of the particle size distribution has been assessed on the basis of the grading integer "n" which is the slope of the particle size distribution plotted to a log-log scale. An open graded aggregate has a steep slope while a dense material has a relatively shallow slope. The theoretical particle size distribution that will result in maximum density has a slope for which n = 0.5. In Figure 1, and in subsequent grading diagrams, lines representing n = 0.5 and n = 0.3 are used to illustrate the shift that has occurred in gradings over the years.

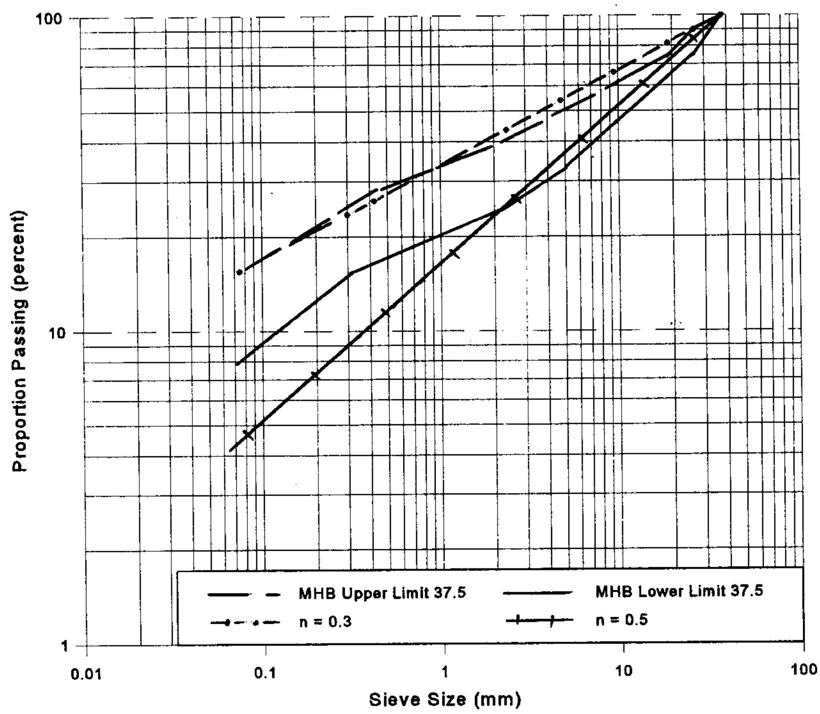


Figure 1 Basecourse Specification - MHB 1938

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Basecourse manufactured to this Specification had a relatively high optimum water content, was easy to lay, but produced a very clayey surface during final rolling. It had to be left as long as possible to dry out before sealing otherwise wheel ruts formed quickly. The sealing practice involved a tar primer to penetrate the clayey surface and provide the required adhesion for the seal coat. The basecourse layer was dense, had a low permeability and adequate stability once it had dried out.

The first major change in aggregate production occurred in 1954 when the NRB B/2 Specification was released. This had a slightly coarser grading (refer Figure 2) than the pre-war specification and required a harder and less plastic aggregate.

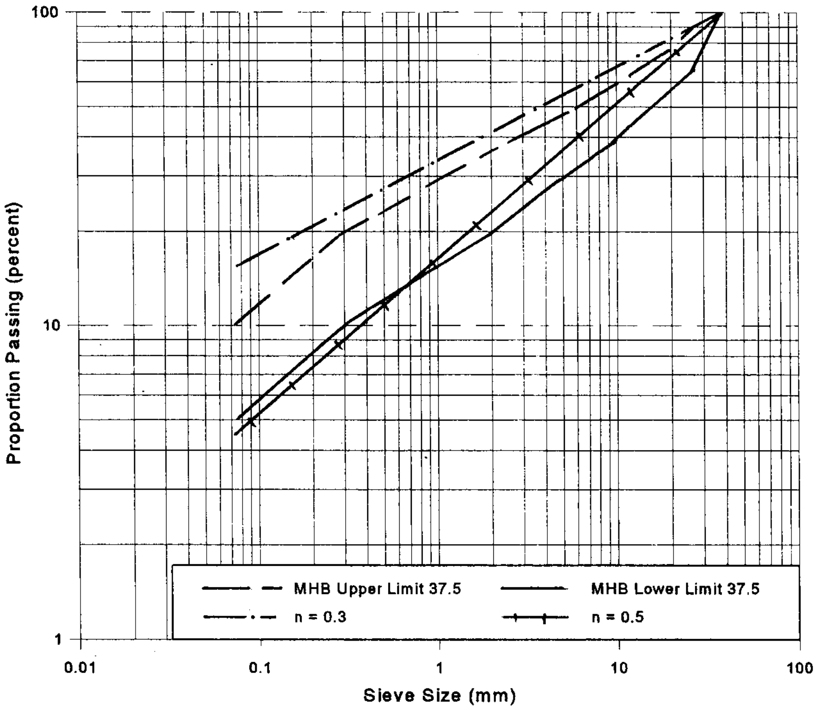

Figure 2 Basecourse Specification - NRB B/2 1954

The objective of the change was to produce an aggregate that would dry out more quickly, was less sensitive to water and would provide a surface suitable for a first coat seal without a primer. This Specification was used on the initial sections of the Southern Motorway from Ellerslie south.

3 NRB M/4 SPECIFICATION

In 1958, the first NRB M/4 Specification was produced, refer Figure 3. This was devised by drawing an envelope that encompassed the particle size distribution of a number of high performance aggregates from throughout New Zealand. It is suspected that the basis for this specification may not have been nationwide, but more confined to the hard greywacke river gravels found mainly in the South Island. This is because the specification provided for a further increase in hardness, a further reduction in plasticity and an envelope widened at the lower end to encourage the use of material that lacked fines.

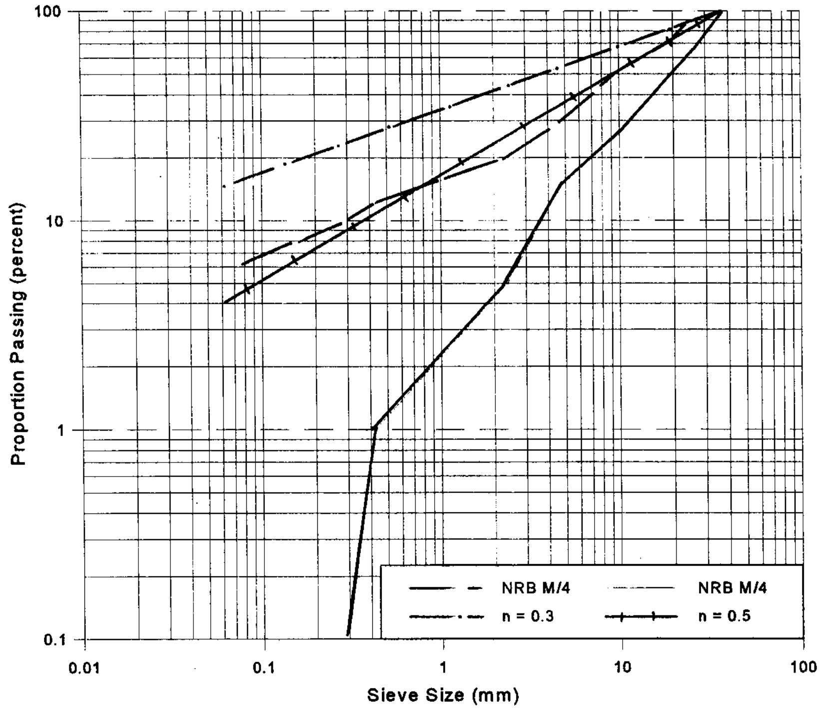

Figure 3 Basecourse Specification - NRB M/4 1958

Clay provides cohesion, a very important part of the strength of soils. By moving the grading well over to the open graded side of the spectrum and minimising the clay content the shear strength of basecourse was reduced to that developed by interparticle friction alone. They also effectively increased the total void space and also the permeability.

4 A FAILURE ON THE SOUTHERN MOTORWAY

In the mid 60's a failure developed in all four lanes of the Southern motorway just north of Redoubt Rd. This section of road had been built in 1955, using an argillaceous greywacke aggregate.

The results of an investigation into the failure carried out by Roading Division of the Ministry of Works, caused a major shift in our attitude to roading aggregate. Up until this time the pavement engineering issues primarily related to determining the correct thickness of a pavement and how to construct it. Stability had not been a concern since the move away from clay rich aggregates. Now here was a large scale failure believed to be due to instability in the basecourse.

The Roading Division investigation found evidence of degradation in the greywacke aggregate as the prime cause of the failure and the possibility that positive pore water pressures had also contributed (Buckland 1957). Bearing in mind that quality assurance had not been thought of in those days and that routine testing of aggregates was not practised, it could be said that the investigation was based on poor quality data and quite a lot of supposition. However, the report was accepted by the roading fraternity and as a consequence attitudes hardened against clay fines in basecourse aggregate and the use of aggregates that could produce clay fines. The possibility that positive pore water pressure could develop in a pavement also became of major concern.

Concerns about clay fines in aggregates became so dominant that many engineers specified M/4 quality aggregate regardless of the situation and some excluded any aggregate that had the potential to produce clay particles. As a result we had the ludicrous situation where M/4 aggregate was used as maintenance metal on loose metal roads and for the construction of car parks and tennis courts. Much of the best quality rock in the Auckland area has been used to build lightly loaded residential streets and supermarket car parks while a lot of lesser quality rock has been virtually dumped to waste.

In addition a lot of attention was paid to the drainage characteristics of the aggregate layers, although this was subsequently confined to ensuring that the subbase layer had a permeability much higher than that of the basecourse layer.

5 FURTHER CHANGES TO THE SPECIFICATIONS

5.1 NRB M/4 - 1973

In 1973 the grading envelope for M/4 basecourse was modified once again. This time to the limits shown in Figure 4. This shows that the pendulum had started to move back towards more dense gradings, although the clay content was still tightly controlled by way of the Sand Equivalent test. At the same time a grading shape control was introduced to ensure that well graded aggregates were produced.

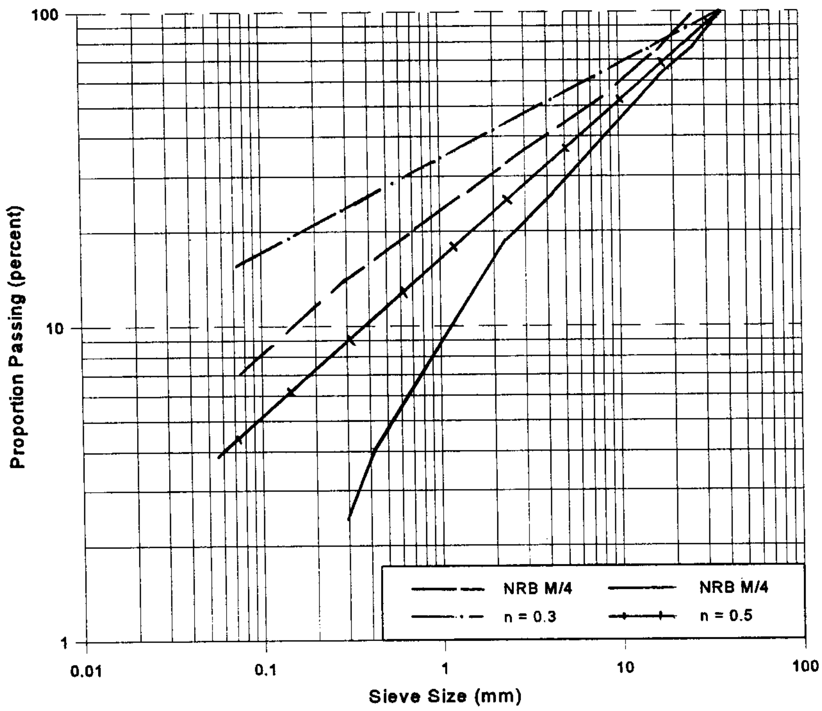

Figure 4 Basecourse Specification - NRB M/4 1973

From the time that the first M/4 specification was produced, changes were being progressively made to tests predominantly designed to limit the production and the plasticity of fines. The Atterberg Limit Test that was used initially was subsequently replaced by the Sand Equivalent test. This was because it was found that determination of a low value of Plasticity Index was prone to operator error and therefore the test was not suitable for universal application. The Los Angeles Abrasion Test which was used in the old MB specification, was replaced by the Crushing Resistance test and the MOW Weathering Resistance test was also introduced.

5.2 Local Derivatives

While the M/4 Specification was supposed to be used nationwide it was found that in some areas, particularly Napier, Gisborne and Rotorua, M/4 basecourse was very expensive to produce. To get round that problem the regional M/5 Specification was born. In general the M/5 relaxed the grading requirements mainly with respect to top size. However the decision to permit modifications to the M/4 specification for whatever reason called into question the justification for such an uncompromising national specification. Many quarries struggled to meet the M/4 Specification, particularly the Sand Equivalent requirement and there was then, and still is, a real need for slight relaxation of the SE value.

6 AGGREGATE RESEARCH

6.1 Road Research Unit

Another result of the Motorway Failure was that political pressure was applied to have research into roading matters, previously the preserve of Roading Division, opened up to include the private sector. This lead to the formation of four Technical Committees, including one on Pavements, set up under the overall guidance of the Road Research Unit of the NRB.

Each committee had representatives from the contractors, consultants, local authorities and universities as well as from the MWD. They were responsible for identifying suitable topics for research and getting it carried out. The money was provided through the Road Research Unit and Roading Division provided the support staff to each Committee.

6.2 Pavements Committee Projects

For virtually all of the 25 or so years of it's existence the Pavements Committee spent a significant part of its budget on aggregate research, in an effort to identify the key factors in the basecourse stability problem. In all nearly 60 projects were carried out in this topic area. The results of projects briefly described in the following sections provided only a few of the pieces that go to make up the jigsaw puzzle of aggregate performance.

6.2.1 Project BC2

One of the Committee's first projects, BC 2 (Bartley 1971), was a comparison of sound with supposedly failing pavements.

The concept was quite straight forward. Compare the properties of the aggregate in each type of pavement and identify those that control stability. All the possible factors that could correlate with failure including loading, density, water content, levels of saturation, grading and plasticity were measured.

The results of the project were less satisfactory than expected. They showed that:

- the concept of the project was not realistic because of the lack of historical construction and loading data,
- pavement failure was difficult to define,
- the measurable properties were not directly related to stability,
- there were one or two aggregates of supposedly poor quality that were performing well,
- dense graded, clayey, aggregate could carry heavy traffic loads.
- the results of Atterburg Limits, Weathering Resistance and the Sand Equivalent tests on their own, did not correlate with performance.
- mature roads have a total voids content in the range 5 - 20 percent with a mean value around 10 percent.

Some of the pavements that were assessed as "failing" had developed ruts and small areas of shear failure. At that time, pavements that had developed ruts deeper than 25mm were considered to be in the process of failing. This was because it was believed that water could pond in the ruts and flow into the basecourse through cracks in the seal. However, it is now recognised that ruts can also be associated with densification of the basecourse layer and are not necessarily a sign that failure is imminent.

Two interesting aspects were:

- Three supposedly sound pavements had greywacke basecourse aggregate with a very low Sand Equivalent value (11, 20 and 25).
- A pavement which was estimated to have carried more load than all the ten others tested, had been built from clay rich greywacke quarry strippings.

6.2.2 BC 16 Series at Quarry Rd

In order to overcome the problem of the lack of start and traffic data, Pavements Committee decided to mount a series of test strips on Quarry Rd in the Franklin District. At that time Quarry Rd carried all the traffic in and out of the Drury Quarry and all loads were recorded at the quarry weighbridge.

The BC 16 study (Tonkin & Taylor 1984) was carried out over a period of nearly 10 years during which the performance of local basecourse aggregates laid as overlays to the existing pavement were compared. During this study the best of the trial pavements carried 1.3 x 10^6 EDA which was probably of a similar order to the load on the Southern Motorway at that time.

The unbound aggregates that were trialed included greywacke, basalt and andesite from various quarries around the Auckland, Waitakere, Manukau, Franklin and Hauraki regions. Other materials trialed including aggregate mixed with foamed bitumen, with lime and with bitumen emulsion.

The results of this BC16 were probably the most interesting of the Committees work. Failures occurred in the basecourse layers of some pavements. Instability occurred in four unbound granular pavements immediately after construction and these had to be reconditioned and resealed. Other pavements deteriorated slowly over a period of 5 to 7 years, while others were in good condition at the end of the project.

However, while it was difficult to precisely identify the cause of failure it was possible to compare the properties of the materials and determine some of the critical parameters.

This series showed the :

- importance of the water content of the fines fraction;
- pavements were most vulnerable immediately after they were opened to traffic;
- coarse graded aggregates tended to rut more than dense graded aggregate;
- aggregate degradation was related to the initial grading;
- small quantities of lime could enhance the performance of clayey aggregate;
- basecourse layers densify under traffic until the total voids content is reduced to 15 - 20 percent.

With regard to the first point. The material finer than the 425 sieve was defined as the "mortar" fraction, being the fine material that surrounds the larger pieces of stone. It was found that if mortar fraction constituted more than 10 percent of the total aggregate and if the water content of this fraction was greater than the Plastic Limit, then the aggregate was unstable.

Figure 5 is a plot of the "n" value times the ratio of the water content to the plastic index versus the proportion of aggregate finer than the 425 micron sieve for all the aggregates used in the B2 and B16 test series. Also indicated on the plot is an arbitrary boundary between the stable and unstable aggregates. Most of the aggregates used in the test series had an "n" value approximately equal to 5.

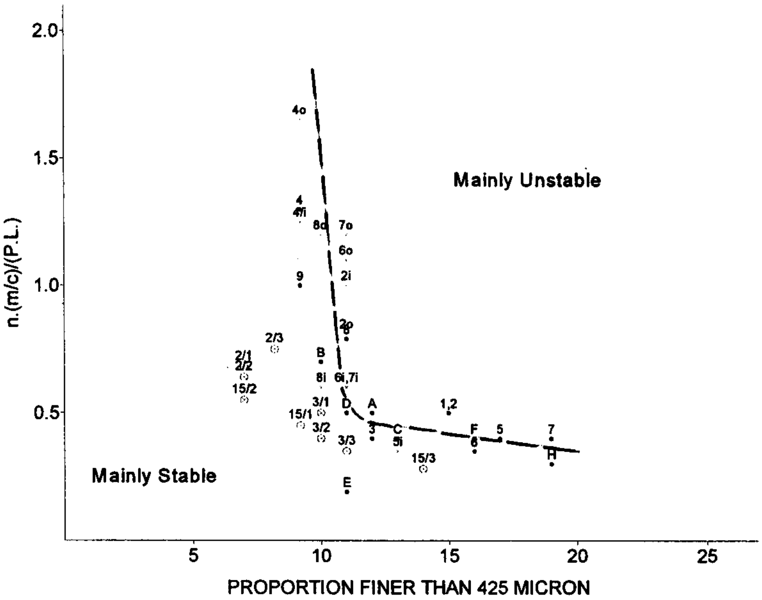

Figure 5 Properties of the Aggregate Mortar (after Tonkin & Taylor 1984)

The water content of the mortar fraction was estimated as follows:

- the water absorbed in the large stone (determined during the solid density test) was deducted from the total water content.
- the remainder was distributed between the mortar fraction and the remainder of the aggregate in terms of the mass of each material.

The importance of getting the grading right prior to laying the aggregate was apparent from the results of these studies. Open graded aggregates were shown to develop deeper ruts and to degrade more than dense graded material.

6.2.3 Swelling Clay Minerals

During the period 1975 - 1985, Prof Sameshima working in the Geology Department at Auckland University, carried out a detailed study of the clay minerals in the Auckland aggregates. This work was organised by the Pavements Committee.

Sodium smectite were found in the older west coast igneous rocks that came in contact with sea water during deposition, but not in the younger central area basalts. Smectites were also found as alteration products in some of the Greywacke group rocks. (Sameshima & Black 1980). Samples of aggregate from the Motorway Failure were found to contain iron rich Saponite another form of Smectite.

Sameshima developed the Clay Index test to quantify the swelling clay mineral content of an aggregate. In this test, samples of either natural fines or crushed rock material passing 75 microns are titrated with methylene blue. A Clay Index result greater than 3 indicates that the quantity of swelling clay mineral present may be excessive.

If high plasticity clay fines are released during rolling and they absorb water then they act as a "lubricant" between the stone particles. If enough water is added the whole basecourse can become very unstable and weave. This happened on one of the BC 16 test pavements that failed immediately after construction. However, if the pavement can be left to dry out (mortar W/C < PL) the aggregate layer will stabilise. The layer should remain stable from then on unless the seal is ruptured and enough water gets back into the aggregate to raise the W/C of the mortar above the PL.

The effect of high plasticity clays can be mitigated by modification with lime or KOBM. In this way the sodium ions in the smectite are replaced by calcium ions by ion exchange and a more stable type of mineral is formed. In addition some strong chemical bonds are established (Sameshima & Black 1982).

7 DEGRADATION

Degradation is the breakdown of larger aggregate particles into smaller ones. Large particles can break into two or more fragments or sharp edges can be eroded as two particles move against one another. Most degradation occurs at the surface of an aggregate layer during to the high level of stress applied by steel wheeled rollers, but some can also occur within the layer. Degradation is an unavoidable part of the compaction process.

Degradation is not necessarily a bad thing. It is one way of altering the grading to achieve a stable, dense arrangement of particles. Basecourse manufactured from the basalt rock in the Auckland region has a reputation for being a simple material to lay. This aggregate has a relatively soft stone that readily breaks down under the rollers but the fines are non-plastic. Less care is needed during laying because any segregated area can be rolled until enough fines are produced to fill the voids. Degradation becomes significant when the fines that are produced are high plasticity clays.

The results of the BC 16 Projects illustrated the influence that the particle size distribution has on degradation. Aggregate that has a preponderance of coarse particles with only a small quantity of fines to fill the void space will be more likely to degrade than one that has a lot of fine particles.

8 PORE WATER EFFECTS

No quantifiable evidence was found in any of the BC16 test sections to support the theory that pavements fail as a result of positive pore water pressures. However, Martin & Toan (Martin 1975) carried out repeated load triaxial tests on unsaturated and saturated samples of basecourse. They found that the resilient modulus was less and permanent strain greater, when the aggregate samples were saturated than when they were tested in the unsaturated condition.

On the other hand there is evidence that negative pore water pressures are important in providing an effective confining pressure within an aggregate layer.

Generally it is agreed that a saturation level of 80% is required before instability can develop.

9 CURRENT M/4 SPECIFICATION

Transit New Zealand made further modifications to the M/4 Specification in 1995. The change relaxed the Sand Equivalent requirement by providing that aggregates with SE < 40 could be accepted provided that:

- PI was not greater than 5%
- CI was not greater than 3

It also included a soaked CBR requirement (CBR>80%).

The grading envelope for the 40m aggregate (refer Figure 7), is the same as that used in the 1973 edition but a new envelope for for 20mm material has been added.

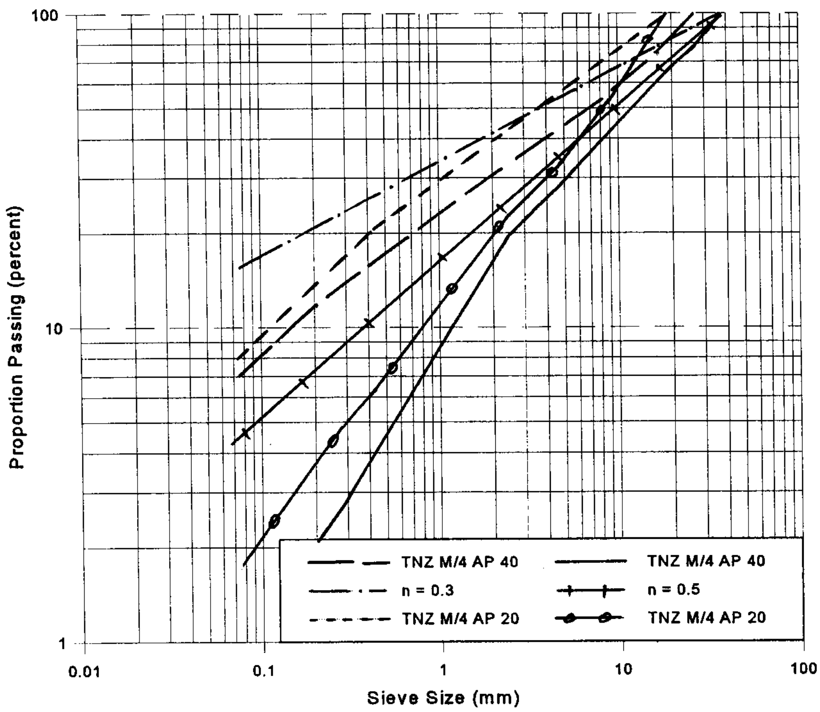

Figure 7 Basecourse Specification - NRB M/4 1995

It is not clear where 20mm envelope came from but it looks as though it would be a relatively open graded material.

The other notable addition is the requirement for source property and production property tests. Of all the changes, this is probably the most important, since it encourages the quarry operator to establish it's own quality assurance system. Included in the source properties is a suggestion that a petrographic examination be carried out. This also is important because a greater knowledge and understanding of the geological nature of the rock will help the operator in the production of quality aggregate. The results of the examination will also alert prospective purchasers to the presence of any deleterious minerals.

Note that the M/4 Specification now requires the contractor to show proof of compliance before the aggregate is supplied. This indicates that the requirements of the specification are not intended to be applied to material excavated from the road after construction has been completed or after trafficking.

Also note that the Clay Index test is applied to the natural fines. It is possible that an aggregate that contains smectite within the stone, could meet the M/4 Specification but that significant active clay particles could be released during compaction.

10 THE COMPACTION OF AGGREGATE

10.1 Introduction

Compaction is an essential part of the pavement construction process. It provides the level of density necessary to enable the aggregate layer to carry the traffic loads with the minimum level of plastic strain.

TNZ B/2 Specification, which describes the construction of aggregate layers, has been changed a couple of times over the last 15 years or so. In the main the changes have concerned the compaction of aggregates. Before 1987 there was no precise end point to compaction and no specific density had to be achieved. Instead there was a requirement to produce "a stone mosaic surface" ready for sealing. There was logic in that requirement to the extent that the basecourse had to be well compacted in order to produce the mosaic surface. However it was all rather subjective.

10.2 Plateau Density

In 1984 Ministry of Works and Development (Auckland) carried out research into aggregate compaction using various types of rollers (Cornwell & Sommerville 1984). They found that there was a level of density reached after a certain number of roller passes which could not be exceeded, regardless of the number of additional roller passes that were applied.

It appears that the concept of plateau density was seen by Roading Division as a solution to the degradation problem. They incorporated it into the first modification to B/2 published in 1987 along with the requirement that vibrating rollers should be used for the first stage of compaction. The idea was to limit degradation by restricting compaction to the minimum number of passes necessary to reach plateau density. However, before the pavement was sealed it was to be opened to traffic so that further densification could occur under rubber tyres. While the concept had merit so long as there was traffic to provide the additional compaction, it wasn't satisfactory for major deviations that could not be opened to traffic prior to sealing. It wasn't long before there were a number of new pavements around the country with quite deep ruts down the wheel paths.

The problem was not the compaction method so much as the grading of the aggregate. In BC 16 it was found that open graded aggregates densify under traffic. Hence getting the grading right before starting compaction is essential. A dense graded aggregate will not degrade and can be compacted with a minimum number of roller passes. On the other hand a relatively weak open graded aggregate will degrade to some extent during compaction, the more compaction the greater the degradation. If the aggregate is open graded and hard the degradation during compaction will be relatively minor. If the total voids content is high when the pavement is sealed, it is highly likely that further compaction will occur once the road is opened to heavy traffic.

10.3 TNZ B/2 Specification 1996

Transit New Zealand's answer to the rutting was to change the B/2 specification once more. This time they have followed the Australian practice and required that the density of basecourse prior to sealing be not less than 98% of MDD achieved using the vibrating hammer test (NZS 4402 : 1986 Test 4.1.3). However this change has not overcome the rutting problem with open graded aggregates.

The maximum density achieved in the laboratory test is a function of the grading of the aggregate and the efficiency of the vibrating hammer. If the aggregate is open graded and hard the density achieved in the laboratory will be relatively low say 20-25% total voids. If the vibrating hammer is old then the lab density will be even lower.

The reference to a laboratory test is just another impediment in the way of the compaction process. Regardless of how aggregates are compacted once the heavy trucks get onto the road, some additional compaction is bound to occur. Measurements taken during BC 2 indicate a mean minimum value of 10 % is the end result. Similar measurements from BC 16 show that younger pavements may have higher total voids (15 - 25%). It is surely preferable to construct a pavement to a low total voids level rather than run the risk of damage to the surface of a new pavement

A summary of measurements taken during BC2 and BC16 are presented in Table 1.

Table 1 Total Voids Content of Aggregate

Road	Aggregate	Mean Dry Density (kg/m3)	Specific Gravity[1]	Total Void (%)
BC 2				
Waimauku SH16	Basalt (Auckland)	2430	3.03	20
Royal Rd SH16	Basalt (Auckland)	2569	3.04	15
Russell's Flat. SH1	Greywacke (Northland)	2473	2.78	11
Maungatapere SH14	Greywacke (Northland)	2337	2.75	15
Braigh SH1	Greywacke (Northland)	2390	2.78	14
Matakohe SH12	Basalt (Northland)	2167	2.78	22
Waiwera SH1	Greywacke (Northland)	2355	2.81	16
Redoubt Rd SH1	Basalt (Auckland)	2619	3.05	14
Takanini SH1	Drury Greywacke	2310	2.76	16
Ballard Rd[2]	Drury Greywacke	2326	2.72	14
BC 16				
Quarry Rd	Waitakere Fine	2003	2.82	29
Quarry Rd	Waitakere Coarse	1977	2.80	29
Quarry Rd	Drury Greywacke Fine	2182	2.72	20
Quarry Rd	Drury Greywacke Coarse	2177	2.71	20
Quarry Rd	Hallewells Greywacke Fine	2330	2.73	14
Quarry Rd	Hallewells Greywacke Coarse	2368	2.75	14

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Quarry Rd	Whitford Greywacke	2322	2.73	15
Quarry Rd	Maramarua Greywacke Fine	2294	2.75	16
Quarry Rd	Maramarua Greywacke Coarse	2360	2.73	13
Quarry Rd	Drury Greywacke (General)	2279	2.73	17
Quarry Rd	Drury Greywacke M4	2300	2.92	21
Quarry Rd	Hallewells Argillite (Bitumen)	2389	2.68	11
Quarry Rd	Wainui Conglomerate	2271	2.79	18
Quarry Rd	Hallewells Argillite (Lime)	2330	2.75	15
Quarry Rd	37E Basalt	2319	3.00	23
Quarry Rd	37E Greywacke	2053	2.73	25
Quarry Rd	19E Basalt	2462	3.08	20
Quarry Rd	19E Greywacke	2120	2.74	23
Quarry Rd	19 ROC Basalt	2246	3.00	25
Quarry Rd	19 ROC Greywacke	2214	2.73	19

(1) Apparent Specific Gravity BS1377 : 1967

(2) Aggregate was pit run weathered greywacke.

The objective of basecourse production should always be to crush the rock to a dense grading and to limit segregation as much as possible during laying. The product ex quarry should have a Clay Index value less than 3 or lime should be added to ensure that it is non plastic. Subbase layers should be compacted to 20 - 25 percent total voids while basecourse should have no more than 15 - 20 percent total voids prior to sealing.

10.4 Apparent Specific Gravity

NZS 4402 does not recognise what used to be called the Apparent SG. Part of the laboratory test report needs to include the air voids or preferably the total voids lines.. If the lab uses the NZS Solid Density it will under estimate the voids lines particularly for igneous rocks. If the Engineer is looking to control its compaction in terms of total voids, then the Solid Density needs to be determined on a saturated surface dry basis. Values measured as part of the BC2 and BC16 projects are given in Table 1. These are typical of the values that could be expected for the aggregates described.

11 CONCLUSION

So what has been learnt about aggregates in the last 50 or so years?

There are no bad aggregates. Basically the properties of an aggregate need to be carefully assessed so that it can be used in the right situation.

Control of moisture and of moisture movement is essential. Many poor quality aggregates perform admirably provided they are kept relatively dry.

The fact that an aggregate complies with M/4 is no guarantee that it will be stable under all conditions. By the same token there are some materials that may not meet M/4 that will, under a particular set of circumstances, provide more than adequate service.

Main arterial routes and heavily loaded areas require dense, stable, durable aggregates. Clay rich aggregates made from a hard durable rock are suitable as a surfacing for unsealed roads. For all the pavements in between there is a wide range of options available.

High plasticity clay minerals are detrimental to the performance of the basecourse. Aggregates with minor quantities of high plasticity clays can be used in heavily trafficked pavements provided the movement of moisture is controlled. Alternatively lime can be added in the quarry or in pavement to reduce the plasticity.

There are a many things that can be done and there are many people in the industry who are prepared to try new approaches. What is required is an open minded approach by the asset managers and other regulatory authorities to the concept of putting more engineering into aggregate selection.

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Field and laboratory testing of volcanically derived soils

M J Pender[1], V M Meyer, T J Larkin, L D Wesley, G C Duske

Department of Civil and Resource Engineering, University of Auckland

Introduction

This is a brief report on part of recent field and laboratory testing on the properties of volcanic ash soils. The work was funded by the Foundation for Research Science and Technology.

At three sites CPT and vane shear strength profiles were obtained, as well as in situ measurements of the shear wave velocity, and the recovery of 100 mm diameter samples for laboratory measurement of soil properties. One of the sites was in Tauranga, another in Hamilton, and the third at Ramarama (south of Auckland). A brief summary of the major findings is given in this article. More detailed results are in papers submitted for publication, copies of which are available from the authors.

Field testing

CPT results

Figure 1 shows the CPT results from the three test sites plotted on the classification chart presented by Robertson et al. (1986). The hatched and double hatched regions of these plots indicate where the CPT points plot, with the double hatched areas denoting regions with a high concentration of points. A characteristic of these plots is the rather horizontally elongated nature of the regions in which the points fall, which is particularly pronounced in Figures 1(b) and 1(c). The majority of points in these three plots categorise the soil penetrated as silty clay. Figure 1(a), which presents the results from the Tauranga site, demonstrates the greatest variation in the positioning of points, though the majority fall into the category of silty soils. Inspection of the soil in the sample tubes confirmed that these categorisations of the ash soils are appropriate. The liquid limit values for soil sampled from the three sites varied between 60 and 130 and plasticity index figures between 20 and 70; on the Casagrande plasticity chart all of these points plot below and just below the A-line (in the MH region).

Since the classification used in Fig. 1 was developed other techniques for classification of soils based on CPT data have been proposed, but these require knowledge of effective stresses, information not available at the three sites investigated as the water table was beneath the soil investigated. This means that the pore water in the ash was probably in a state of suction and the determination of the effective stress is consequently not a routine matter.

CPT vane correlation

Theoretical and empirical research into the interpretation of the undrained shear strength of clays from cone penetration test data typically produces a relationship of the form:

$$s_u = \frac{(q_c - \sigma_{vo})}{N_k}$$

where q_c is the cone resistance, N_k a cone factor, and σ_{vo} the in situ total vertical stress. Theoretical solutions typically encompass bearing capacity or cavity expansion theories, and usually derive an N_k value of around 9. Empirical correlations on the other hand are based on reference to in situ tests which commonly result in N_k values between 9 and 20. Figure 2 presents a comparison between the recorded cone resistance and the undrained shear strength measured using the field vane at each site. The total in situ vertical stress was calculated using the mean bulk density of the samples retrieved from each test location. From Fig. 2 it can be seen that an N_k value of 12 produces a good correlation for all of the test results. Correlations on an individual basis give N_k values of 10.8, 13.8 and 10.2 respectively for the Tauranga, Ramarama and Hamilton sites. The average N_k value is 11.6 which compares favourably with the value of 12 presented in Fig. 2.

[1] MJP contact details: email m.pender@auckland.ac.nz, ph (09) 3737 599 ext. 7919, fax (09) 3737 462.

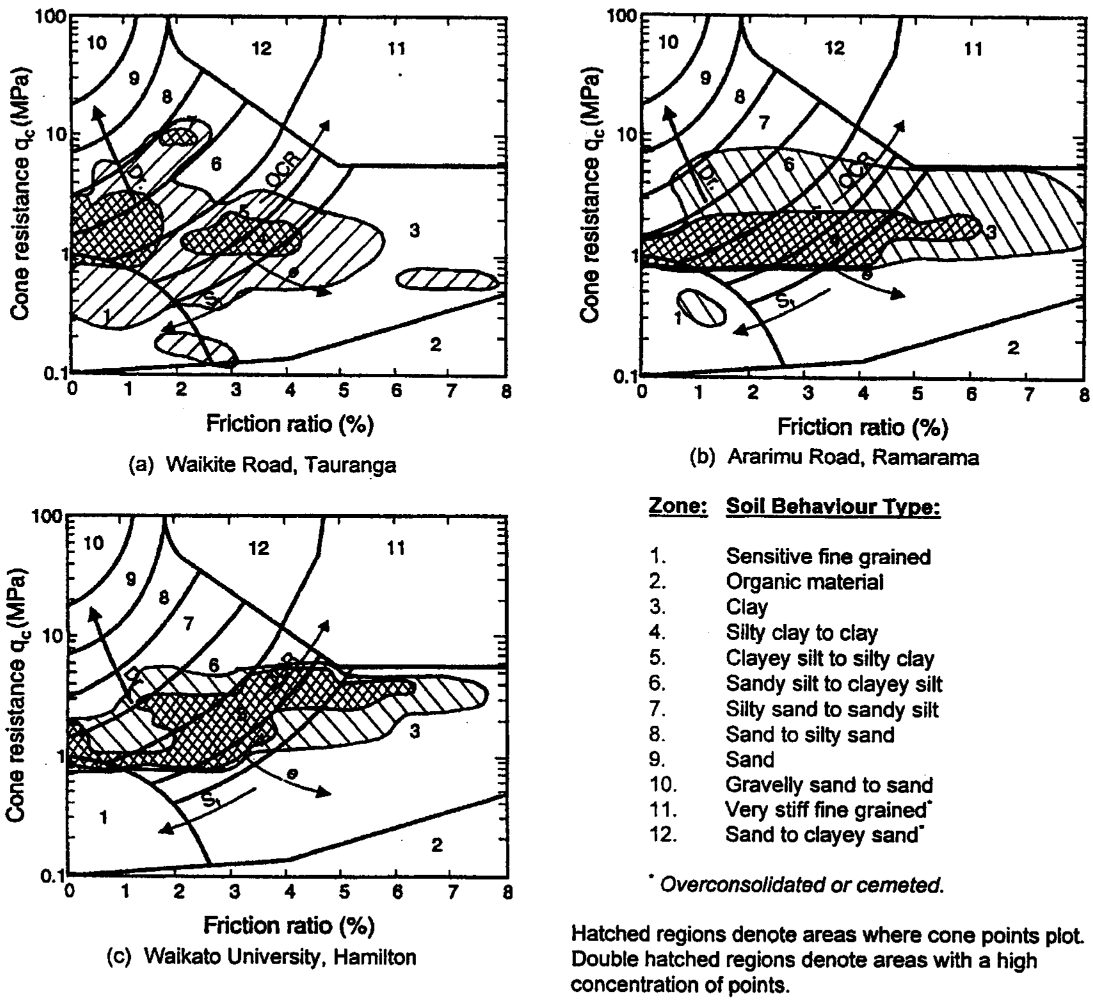

Figure 1: CPT data plotted on the classification chart of Robertson et al (1986).

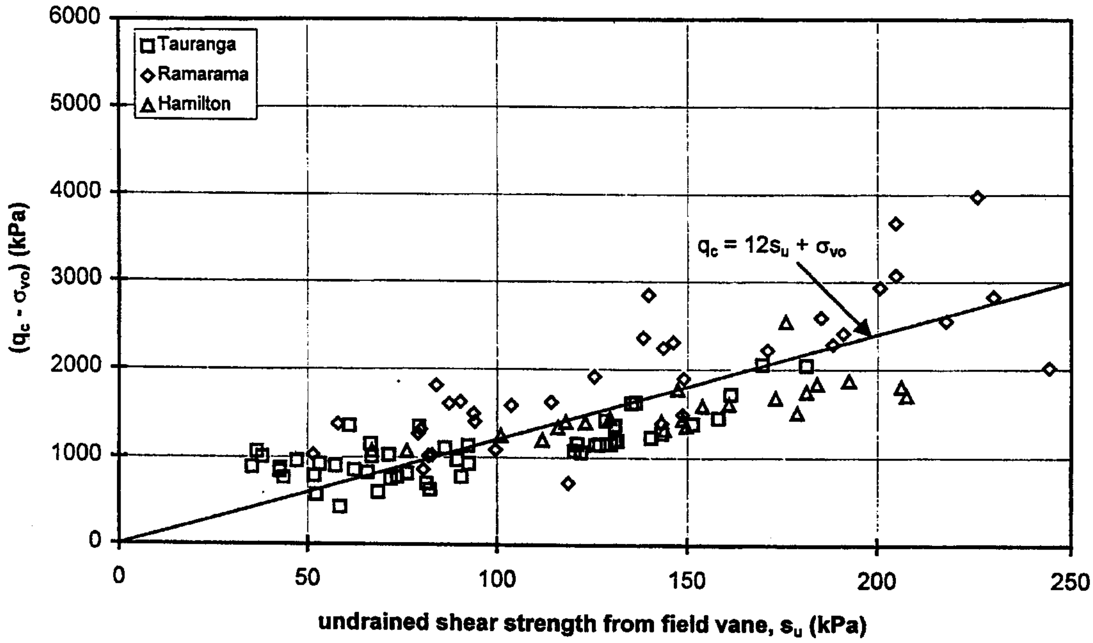

Figure 2: Correlation of cone resistance with field vane undrained shear strength.

In situ shear wave velocities

The measurement of the shear wave velocity enables the small strain shear modulus of the soil to be calculated; this is the upper bound on the shear stiffness. A total of 10 cross-hole and 55 down-hole tests were performed during the course of this investigation to measure shear wave velocity. Downhole in situ shear wave velocities were obtained by measuring travel times between the ground surface and the geophone mounted in the CPT probe. For the cross hole measurement of shear wave velocity the transmission path for the waves is horizontal in contrast to the vertical propagation of the downhole measurement. The shear wave velocities for the three soil profiles ranged between 175 and 340 m/sec. A significant finding of from these results is that the shear wave velocity in the vertical direction is nearly same as that in the horizontal. Since the SV and SH wave velocities are similar there is little anisotropy in the stiffness of the soil at each site. The trace of the records from the cross-hole and down-hole tests also enabled P-wave velocities to be determined at various depths for each of the sites. The P-wave velocities ranged from 385 to 1188 m/s. When combined with the associated shear wave velocity the corresponding to Poissons' ratios are between 0.22 and 0.42.

Sensitivity of the Volcanic Soils

The sensitivity of a clay soil is defined as the ratio of the undrained shear strength in the undisturbed state to the undrained shear strength, at the same water content, in the fully remoulded state. The field vane is commonly used to measure the sensitivity of a clay soil, with the remoulded strength being recorded after two complete revolutions of the vane. It has however been shown (e.g. Quiros and Young (1988)) that the sleeve friction, f_s, measured by the CPT is approximately equal to the remoulded undrained shear strength. Therefore, the sensitivity of a clay can be estimated from CPT data by calculating the peak s_u using a correlation with q_c (Fig. 2), and the remoulded s_u from f_s. These sensitivities were calculated for the three sites; as with the correlation between s_u and q_c, the Ramarama and Hamilton sites show good agreement, with the Tauranga site showing the greatest variability. This variability is due in part to the continuous nature of the CPT which permits small layers of soil to be identified which are missed by the coarser resolution of shear vane testing. From the field vane tests, average sensitivities of 6.5, 3.8 and 3.0 were determined for the Tauranga, Ramarama and Hamilton sites. These sensitivities are in the range of normal to slightly sensitive soils.

Laboratory testing

Consolidated undrained triaxial testing

Consolidated undrained triaxial tests were done on samples from all three sites. For the sample taken from a depth of 6.3 m at the Ramarama site the effective stress paths for these triaxial tests are presented in Fig. 3a and the Mohr-Coulomb and approximate critical state shear strength parameters indicated on the plot. It is notable that the shapes of these effective stress paths are different from those obtained on, for example, Auckland residual clays. In Fig. 3a there is a sharp fall in the shear stress once a peak is reached. For the Auckland residual clays the curvature of the stress paths is the other way, so that the paths often reach a point where the decease in p′ stops and then both q and p′ increase. Figure 3b presents the void ratio response, in terms of void ratio and the natural logarithm of the mean principal effective stress (p′), for the triaxial tests. Also shown in Fig. 3b are the results from the oedometer test performed on the soil sampled from the same depth. Comparison of the slopes of the normal compression line from the oedometer test with the projection of the end points of the triaxial stress paths triaxial indicates that the lines are parallel with a slope of 0.41 (compression index 0.94). This observation is in agreement with the ideas of critical state soil mechanics, even though soil of volcanic origin is far removed from the soil type for which critical state soil mechanics was developed. The other point to note, which further emphasises that soil of volcanic origin is different from the "classical" sedimentary soils, is that the compression index of 0.94 is greater than the value that would be given by the Skempton relationship ($C_c = 0.7(LL - 0.1)$). The Ramarama soil at the depth of the sample has a liquid limit of about 110%; for this the Skemption relationship gives $C_c = 0.7$.

An attempt was made to correlate the undrained shear strength measured in the field with those inferred from the consolidated undrained triaxial tests. To do this s_u was taken as half of the peak deviator stress measured in the CU test, at a confining pressure equal to the in situ stress condition (assuming K_o to be 0.5. There is some disparity between these values of s_u and those measured using the field vane, with the triaxial strength being significantly less. This disparity is particularly evident for the tests performed at shallower depths, for which there are several possible explanations. Firstly, it is quite likely that negative pore pressures were present in the soil in situ. Although no direct measurements were made to determine the ground water levels, general observations of the bore holes drilled at each site indicated that the water tables were at a depths of 6 m or greater. The soil above this depth may be saturated but will be subject to negative pore water pressures.

However, the CU triaxial tests, which were done on saturated specimens, would have been tested at a lower state of effective stress than that in the field, leading to lower measured undrained shear strengths. Secondly, and possibly more importantly, there is a significant difference in the rate if testing between the CU triaxial tests and the vane tests performed in the field. Considering the vane shear test, the rate of rotation of the vane was approximately 0.1 °/sec, which in linear terms equates to around 2.72 mm/min at the vane circumference. In comparison, the triaxial tests were performed at rates of either 0.076 mm/min or 0.015 mm/min, or approximately 36 to 182 times slower than the field vane test. (These comparisons can only be approximate as the mode of shearing at the periphery of the vane is different to that in the laboratory triaxial apparatus.) It is well documented that the rate of testing has a significant influence on the measured strength of a soil (e.g. Ahmed-Zeki et al. (1999)). To further investigate the influence of the rate of strain on the measured undrained strength of these soils, a series of fast undrained triaxial tests were performed on soil sampled from the Tauranga site at a depth of between 1.8 and 1.9 m. Two unconsolidated undrained (UU) and two consolidated undrained (CU) triaxial tests were performed on this soil. The UU tests were performed with a rate of strain of 10.8 mm/min (0.18 mm/sec) and the CU tests were performed at 12.0 mm/min (0.2 mm/sec) and 102.0 mm/min (1.7 mm/sec) respectively. These stress-strain curves clearly showed an increase in shearing resistance with increasing rate of strain. In particular, the fastest CU triaxial produces an undrained shear strength which is very close to that measured using the field shear vane.

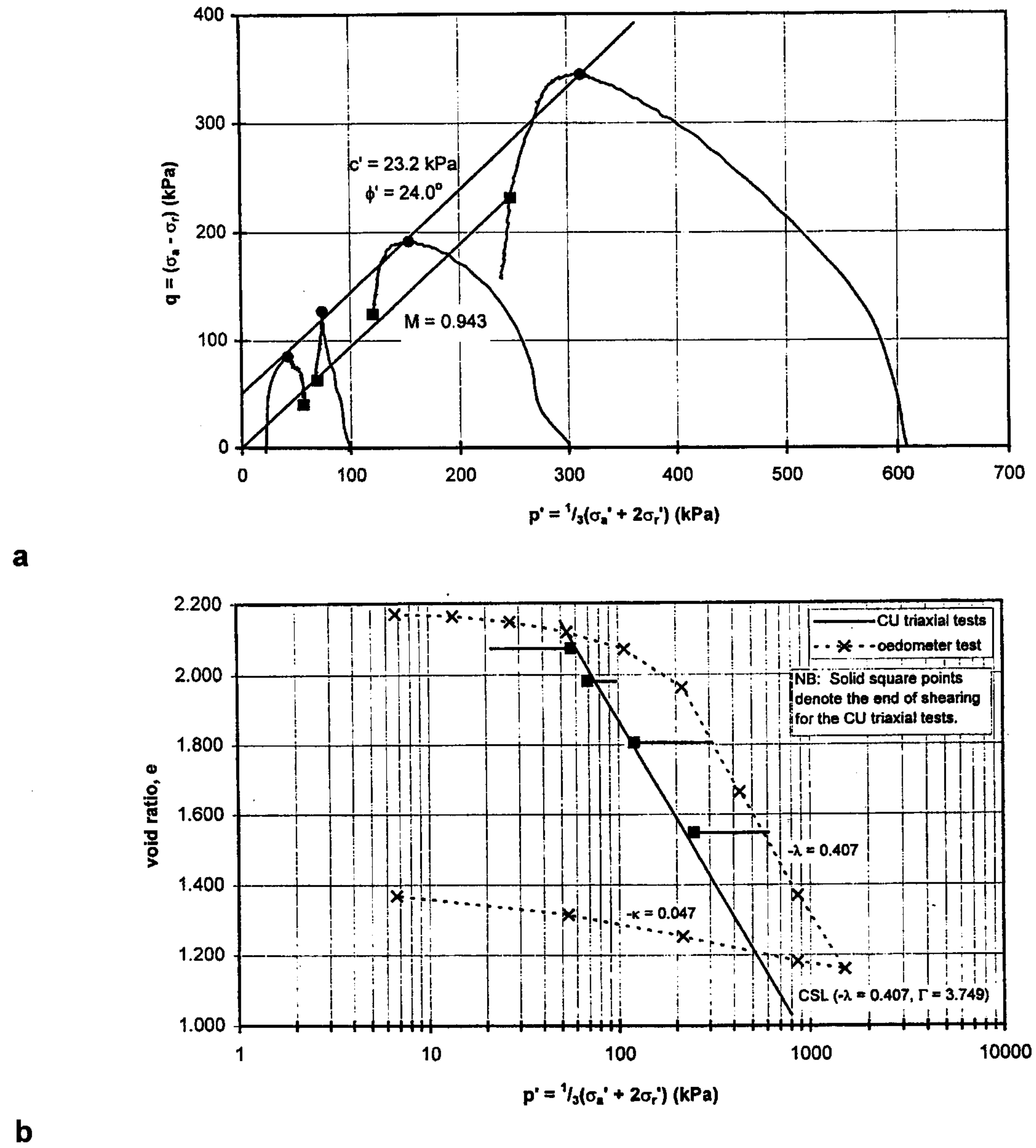

Figure 3: Laboratory test results for the sample recoverd from a depth of 6.3 m at the Ramarama site. (a) Effective stress paths for consolidated undrained triaxial tests, (b) triaxial and oedometer results.

Oedometer testing

The one-dimensional constrained modulus, M, is typically measured using the oedometer test, however, linear correlations in terms of q_c have also been developed:

$$M = \frac{1}{m_v} = \alpha_m q_c$$

where m_v is the coefficient of volume compressibility and α_m is a coefficient based on soil type. Figure 4 shows a plot of M versus q_c for the soil sampled from the Tauranga, Ramarama and Hamilton test sites. The M values were calculated from the oedometer tests with a vertical stress equal to the overburden pressure for the depth from which the soil was sampled. From Figure 4 it can be seen that M is relatively constant at approximately 4 to 5 MPa for q_c values less than 2 MPa. Sanglerat (1972) presented α_m values for silts and clays of high liquid limit (MH and CH classifications) in the range of 2 to 6 for q_c values less than 2 MPa. Calculated values of α_m for the volcanic soils range between 1.7 and 28.5 with an average of 6.1. These results indicate that even for comparatively low cone resistances these volcanic soils possess significant stiffness, suggesting that deformations under applied loading may well be less than those predicted using cone resistance data.

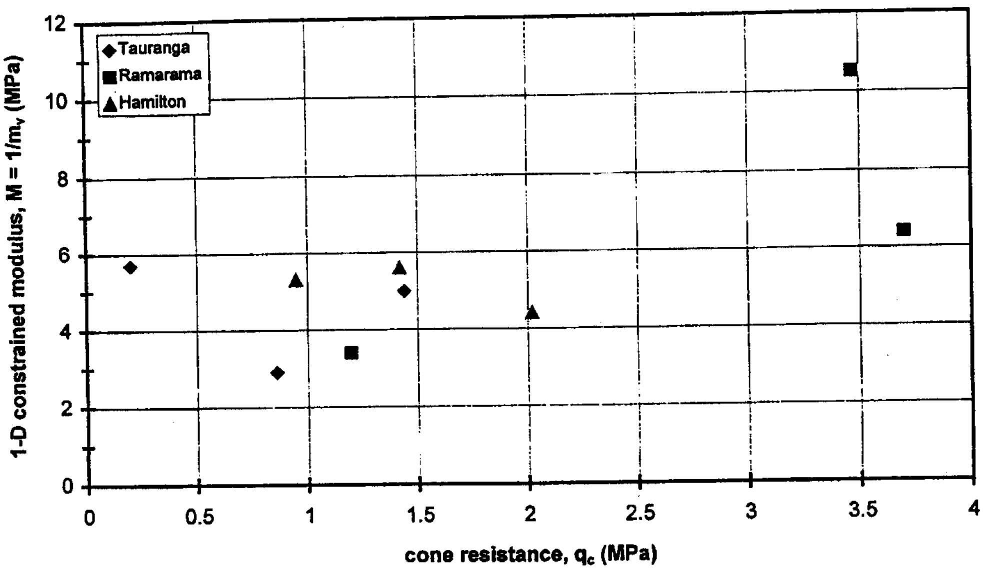

Figure 4: Modulus for one dimensional compression from laboratory oedometer tests related to the cone resistance.

Shear wave velocity and shear modulus damping relations with strain

The shear wave velocity makes it possible to calculate the small strain shear modulus for the soil (G_{max}). The G_{max}/s_u ratios determined for the volcanic soils tested during this research are of the order of several hundred. This is consistent with the values given by Hara et al (1974), Weiler (1988), and also Meyer (1997) for soils which exhibit overconsolidated behaviour.

The shear wave velocity was also measured in the laboratory using piezoelectric bender elements at each end of a 150 mm tall triaxial specimen. An electrical signal is supplied to one element and the response at the other monitored with an oscilloscope. It is also possible to determine the shear wave velocities from the small strain shear modulus derived from free vibration torsion tests. The shear wave velocities from the torsion and bender element tests show excellent agreement, however these velocities range between 0.43 to 0.79 times the field values. It is not uncommon for shear moduli determined in the laboratory to be lower than corresponding values measured in the field. Sample disturbance, resulting in the loss of fabric, interparticle bonding, and aging effects in the soil is the likely cause for this reduction.

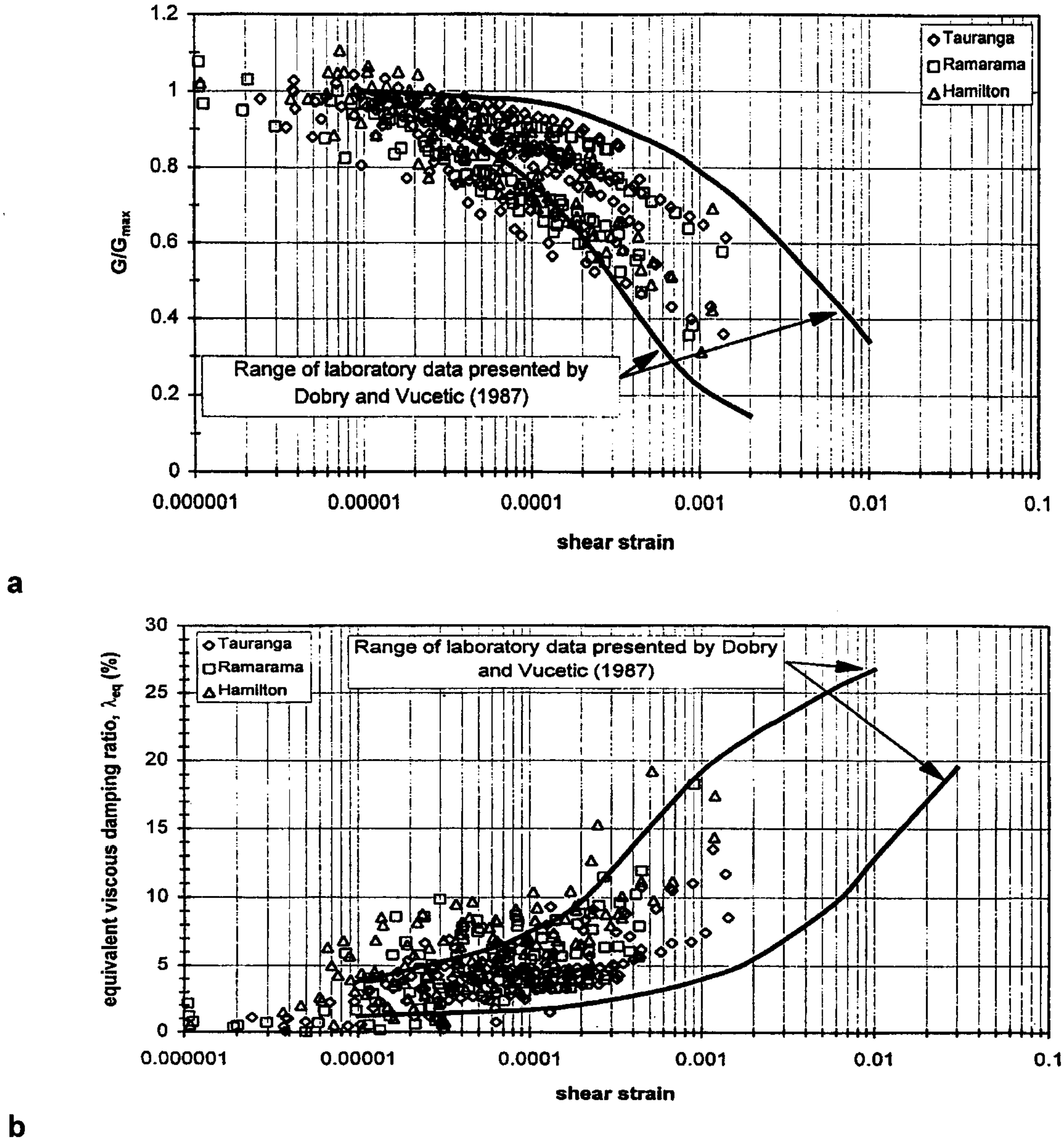

Figure 5: Shear modulus and equivalent viscous damping ratio curves obtained from laboratory torsional vibration tests. (a) Shear modulus relationship, (b) equivalent viscous damping ratio.

Figure 5a shows the normalised shear moduli degradation relationships determined from the laboratory torsion tests. The Dobry and Vucetic (1987) modulus reduction curves determined from a range of clay soils tested in the laboratory are presented in the figure for reference. Figure 5b shows the variation of the equivalent viscous damping ratios with shear strain, as determined from the laboratory torsion tests. Damping ratios for the Dobry and Vucetic (1987) data is also presented in the figure for reference.

Conclusions

In situ cone penetration and vane shear tests were carried out on three volcanic ash soil sites in the upper half of the North Island, New Zealand. 100 mm diameter tube samples were taken from each site at approximate depths of 2, 4 and 6 m which were later used for laboratory tests.

The CPT soil classification charts presented by Robertson et al. (1986) were shown to work well with volcanic soils, producing soil classifications primarily of a silty clay nature. This interpretation compares favorably with the physical classification determined from the bore hole logs for each site.

Comparison of q_c with s_u measured using the field vane produced very good correlations for the Ramarama and Hamilton sites, with the Tauranga site showing the greatest variation. N_k values of 10.8, 13.8 and 10.2 were calculated for the Tauranga, Ramarama and Hamilton sites respectively. An average N_k value of 12 was applied to the CPT results from each site, which produced good correlations with s_u measured using the field vane.

Sensitivities were calculated from the field and laboratory shear vane tests, with average values of 6.5, 3.8 and 3.0 evaluated for the Tauranga, Ramarama and Hamilton sites respectively. Sensitivities were calculated from the CPT data by considering f_s being equal to s_u(remoulded). The CPT results showed a good correlation.

Comparison of the cone resistance with the one-dimensional constrained modulus, M, measured using oedometer tests indicated that M was relatively constant at approximately 4 to 5 MPa for q_c values less than 2 MPa. These values are noticeably greater than those presented in the literature suggesting that deformations under applied loading may well be less than those predicted using cone resistance data.

The next issue of Geomechanics News will have a further article on this project covering the use of CPT data to predict settlements of foundations in volcanic soils.

Acknowledgements

The work was funded by the New Zealand Foundation for Research in Science and Technology (contract UOA604). The authors are also grateful for the assistance of Mr. David Jennings of the Hamilton office of Opus International Consultants Ltd., for providing information on suitable test sites and assistance during field testing operations.

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SETTLEMENT PREDICTION COMPETITION (\$100 Prize)

NZ Geomechanics News is calling for registrations of interest for predictions of settlement for a site which is to be preloaded. The site is underlain by volcanic deposits.

The following data will be supplied to registrants:

- **Borehole Data**
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  - **Atterberg Test Data**
  - **Foundation Soil Density Test Data**
  - **Soil Moisture Contents**

Predictions will comprise magnitude of maximum settlement at the end of the pre-load time frame and must be supported by brief calculations. Guesses will not be accepted. Entries are free, but restricted to individual Society members only. Entries will be judged by a panel of three and their decision will be final.

Please forward your registration of interest to

The Editor
NZ Geomechanics News
P O Box 5271
Auckland

Correlations Between Cone Penetration Resistance and Standard Penetration of some New Zealand Volcanic and Alluvial Deposits

Alaa S. Ahmed-Zeki[1] and Peter B. C. Bosselmann[2]

Abstract The results of cone and standard penetration tests carried out in Tauranga volcanic soils and in Whangarei alluvial soils are graphically correlated using normal arithmetic plots. The linear least square equation relating the cone tip resistance, q_C, and the standard penetration N value is in the form $q_C = \kappa * N$, where κ is a constant. The ratio q_C/N is then used to establish correlations with the type of soil and the results are compared to international findings.

1. Introduction

CPT-SPT correlations are useful for the conversion of available field test data into the form appropriate for various foundation analyses and design. It is attempted here to prepare correlations on local materials which should lead to better results than others based on internationally collected data. It has frequently been emphasised that it is imperative to rely on locally prepared correlations (e.g. Pender, 1998). A new evidence on the significance of deriving correlations on local soils comes from the finding that cone penetration resistance of pumiceous sand is not an indicator of relative density (Wesley *et al.*, 1998). Additionally interesting is the conclusion arrived at by Marks and Larkin (1998) that the low strain shear modulus of the pumice sand is significantly lower than that of quartz sand at similar relative densities and confining pressures. Pranjoto and Larkin (1995) have suggested that the grain softness of pumice sand and the high void ratio as being substantially responsible for its different behaviour in comparison to quartz sand. Nonetheless, it is worthwhile mentioning that Takesue *et al.* (1995) have shown that CPT-SPT correlations as well as pile design values for a volcanic soil (distributed in southern Kyushu-Japan and locally known as Shirasu) were consistent with quartz sand values.

2. Geology

Tauranga Basin where the field tests covered by this study were undertaken, is a Pleistocene, predominantly fluvial/estuarine basin which was partially infilled during a period of rapid subsidence after the eruption of the Waiteariki Ignimbrite (Briggs *et al.,* 1996). The infill in the basin is comprised of terrestrial and estuarine volcaniclastic sediments and non-welded or partially welded distal ignimbrites and airfall tephras (Pahoia Tephras), generally covered by a sequence of younger airfall tephra (e.g. Hamilton Ash and Rotoehu Ash). These were reworked via fluvial, lacustrine and estuarine processes (Matua Subgroup), and re-deposited in sequences interbedded with the primary volcanic units.

In Whangarei, the field tests were carried out in a low lying area consisting of undifferentiated riverbed and floodplain alluvium (Thompson, 1961; Markham, 1981). These soils typically consist of thinly to very thinly bedded, very soft to soft, unconsolidated sands, gravels, clays and silts with occasional lenses of black humus rich peat.

(1) Geotechnical Engineer, Foundation Engineering Ltd. - Auckland
(2) Senior Geotechnical Engineer, Foundation Engineering Ltd. - Auckland

3. Field Test Procedures

3.1 Standard Penetration Test

SPTs consist of driving a 50mm outer diameter split spoon sampler into the soil using a 64 kg hammer free falling through 0.75 metres. The number of blows required to drive the split spoon sampler a distance of 300mm, after an initial penetration of 150 mm is referred to as the SPT "N" value. The test is used mainly to assess the density of non-cohesive soils, but may also give an indication of the relative strengths of cohesive soils. It has the advantages of being simple and rapid, allowing a large number of tests to be undertaken at a relatively low cost, supplemented by a large database. It is a common practice not to record a blow count exceeding 50.

Patterson-Kane and North (1986) stated that due to the fact that mechanical drop hammers as used in New Zealand deliver a much greater energy per blow than the American rope and capstan method, the locally performed tests consequently give numerically lower SPT values. Therefore they suggested that the SPT results may be factored up by 20 to 25 % before application to the correlation charts prepared in North America. However, the writers of this article are unaware of anybody applying this procedure in New Zealand.

3.2 Cone Penetration Test

In these tests, a 35 mm diameter rod with a cone tipped end is pushed continuously into the soil at a rate of approximately 20 mm/second by a hydraulic jacking system. Measurements are made of the end bearing resistance on the cone (the actual end bearing force divided by the cross-sectional area) and the friction resistance on a separate 130 mm long sleeve (the frictional force on the sleeve divided by its surface area), immediately behind the cone. A 200 kN load cell was used in Tauranga soils tests while the capacity of the load cell in Whangarei alluvials was 100 kN. As penetration occurs the information is recorded digitally, processed and plotted once testing is finished.

4. Analysis and Discussion

The depths to the recorded SPT N values were considered to be at mid point of the 300 mm penetration of the sampler. At these depths the corresponding q_c values were recorded. An alternative procedure would have been to calculate average values of N and qc along the 300 mm penetration of the SPT sampler. No correction for overburden pressure was applied. The correlations were carried out for every two adjacent CPT and SPT at the sites. At one location within the volcanic soils, two CPTs were performed adjacent to an SPT location; here the q_c values were averaged at the assigned depths.

A direct q_c versus N relationships were established in the form of;

$$q_c = \kappa * N$$

where κ is a constant. The results are presented in Figures 1 and 2 which show that the correlations within the volcanic soils demonstrate a much larger scatter. In Figure 2, setting the trendline's intercept on the vertical axis to zero resulted in a negative value for correlation coefficient, so the dashed trendline representing a non-restricted linear best fit relationship is plotted to demonstrate the close comparison in magnitude for the q_c versus N values under consideration.

Power (1982) reported values for κ of 0.3, 0.5 and 0.7 for a range of Chalk types, while Chang (1988) observed values of 0.18, 0.2 and 0.23 for Singapore residual soils with a well-defined correlation.

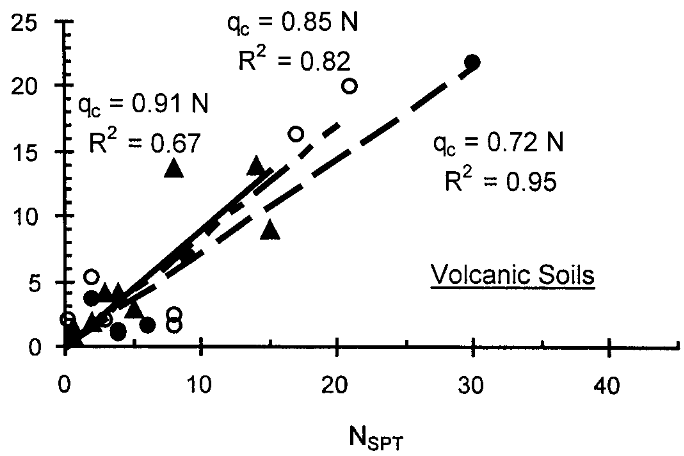


Figure 1 CPT-SPT correlations for volcanic soils. Different symbols denote pairs of CPT-SPTs at different locations of the site.

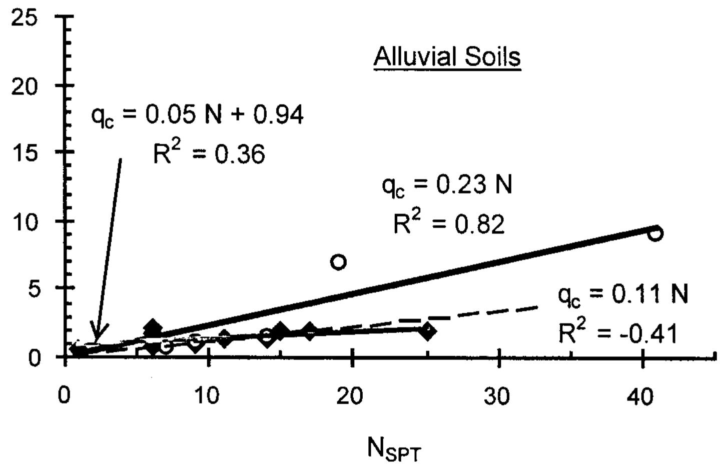


Figure 2 CPT-SPT correlations for alluvial soils. Different symbols denote pairs of CPT-SPTs at different locations of the site.

To correlate the findings of these field tests with the type of soil penetrated, Figures 3 and 4 were plotted on normal scales. No grain size analyses were available for incorporation into the current work, so the soil description presented in the borehole records was used. As can be seen, the plots for the volcanic soils show a very significant scatter. Figure 3 for the alluvial soils, however, shows a somewhat better relationship for the two parameters and the q_c/N ratio values here range between 80 and 500 for the adopted units.

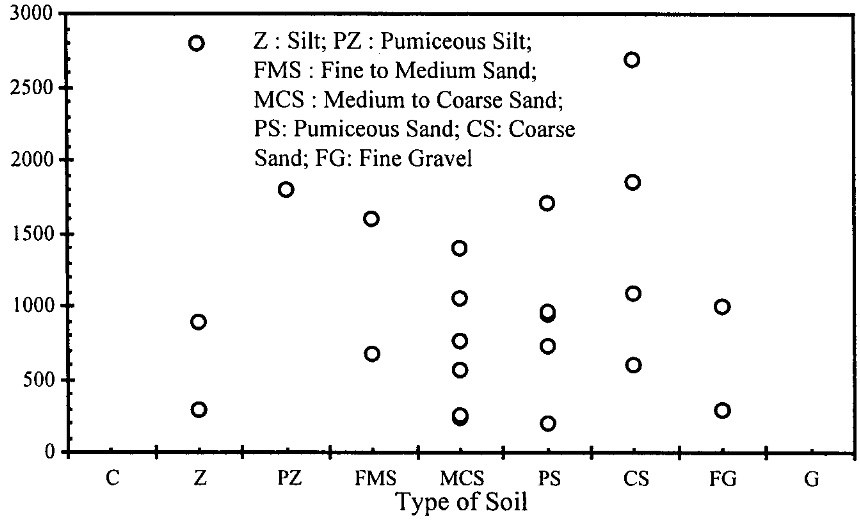


Figure 3 Correlation of q_c / N ratio versus soil type (volcanic soils)

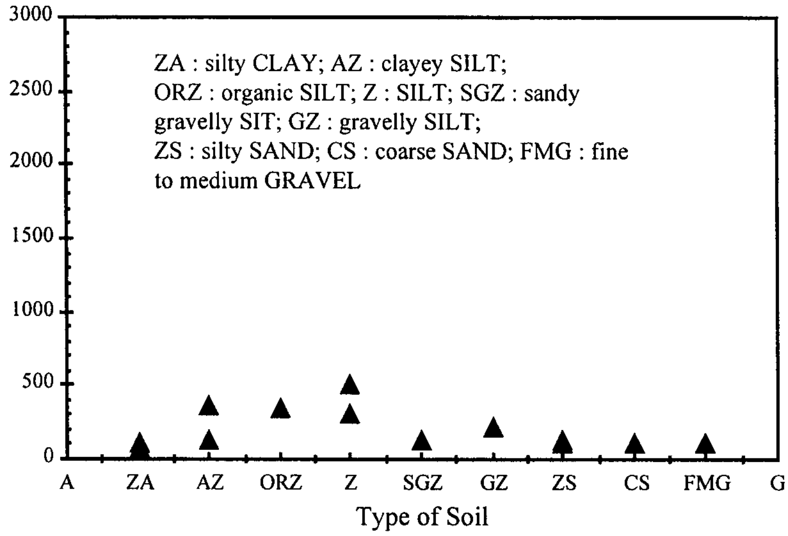
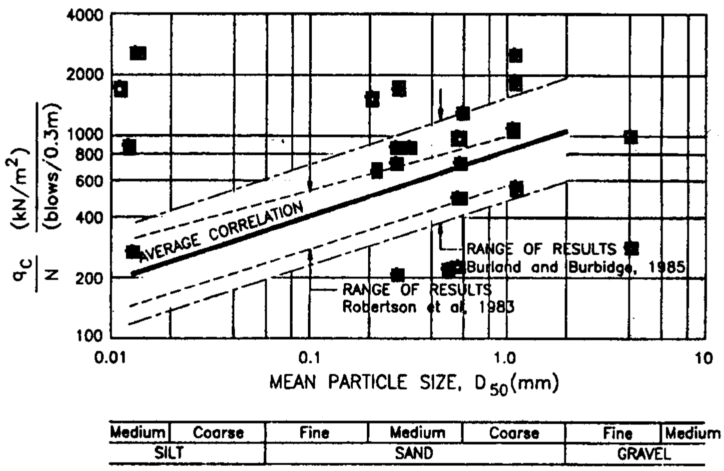


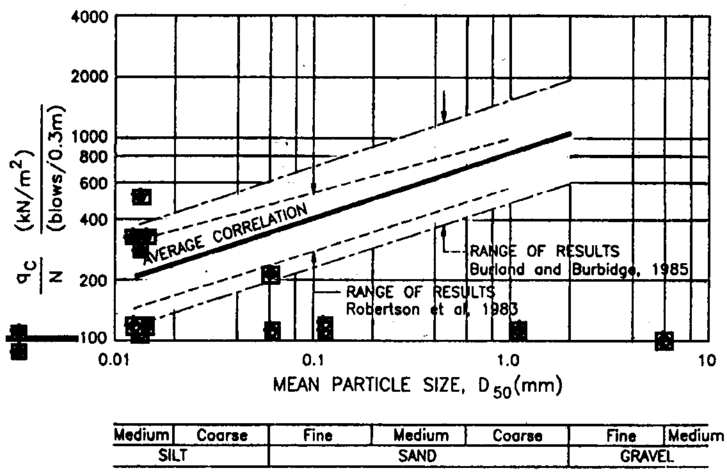
Figure 4 Correlation of q_c / N ratio versus soil type (alluvial soils)

In an attempt to compare parameters from local soils to those published in geotechnical literature overseas, the q_c/N ratio against soil type was plotted as shown in Figure 5 (a) and (b). Extremely high values, which are believed to be due to the sensitivity of the materials to disturbance where SPT N values approach zero, were discarded. The limitation on using the bore log description still applies for these figures, albeit it can be observed that finer tuning to plot the points within any soil group does not seem to alter the general picture.
It is clear on Figure 5 (a) that the points of q_c/N ratio versus soil type for the volcanic soils fall far beyond the outer limits observed by Burland and Burbidge (1985) or Robertson *et al.* (1983), regardless of the soil's particle size distribution. The corresponding relationship for the alluvial soils

as shown in Figure 5 (b) compares reasonably closely to the published relationships in the fine-grain region, but significantly lower for the sand and gravel sized soils and so it does not show dependence on the grain size distribution. For both volcanic and alluvial soils, the increasing trend of the q_C/N ratio is not visible.



(a)



(b)

Figure 5 Relative locations of q_C/N ratio versus "as-logged" soil type points for (a) volcanic soils and (b) alluvial soils, plotted on a pre-prepared figure of the variation of q_C/N ratio with the mean grain size of soil. The dashed lines show the upper and lower limits of observations by the relevant researchers.

(Figure originally printed in the Canadian Foundation Engineering Manual, 3rd edition, 1992, page 52, and is reprinted with the permission of the Canadian Geotechnical Society)

5. Conclusion

This study was an attempt to make use of some available database in formalising correlations for the cone resistance and standard penetration in local geological formations. No doubt, more data should improve the results and refine the observations.
Linear least square equations have been presented relating CPT cone resistance q_c and the standard penetration N value for volcanic and alluvial deposits found at two sites in Tauranga and Whangarei respectively.
However, rather than succeeding in observing well pronounced relationships, a substantial data scatter was observed, especially with the volcanic soils, when plotting q_c/N values against the type of soil which could indicate either that correlation relationships are difficult to obtain because of inherent variabilities in the two types of field tests or that the effect of some other soil property other than grain size distribution is influential.
In comparison to the observations published overseas, the trend of increasing q_c/N values against increasing soil particle size was absent in this study. Certainly, this study has presented a new evidence for the incorrectness of the direct application of imported correlations for design purposes.

6. Acknowledgements

The writers are grateful to Prof. Michael J. Pender and Dr. Laurence D. Wesley for reviewing and commenting on this article. The assistance of Mr. Rodney Melville-Smith, Managing Director of Foundation Engineering Limited in providing access to the Company's database is highly appreciated. Also due thanks are forwarded to Mr. Barry Coker / Foundation Engineering Limited for the valuable discussions on computer spreadsheeting.

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PROBABILITY AND CONSEQUENCES OF THE NEXT ALPINE FAULT EARTHQUAKE

Summary of EQC research report 95/193, dated March 1998
by Yetton, Mark D[1].; Wells, Andrew[2]; Traylen, Nick J.[3]

[1]Geotech Consulting Ltd and Natural Hazard Research Center, University of Canterbury

[2] Plant Science Department, Lincoln University

[3] Geotech Consulting Ltd

The Alpine Fault is the largest active fault in New Zealand and extends over 650 kilometres from Milford Sound to Blenheim. The most active part of the fault is the central and north section forming the western boundary of the Southern Alps from Haast to north of Inchbonnie. A recently completed research project funded by EQC, University of Canterbury, Lincoln University and numerous local authorities and infrastructure providers examines the seismic hazard along this part of the fault. The new evidence suggests that the fault has been more active in the last 500 years than was previously thought, however the probability of a future earthquake over the next 50 – 100 years is high.

To evaluate the probability of a future earthquake the history of past earthquakes must first be established. This has been done by a combination of four methods. The first and most direct method is the excavation of trenches and pits across the most recent area of fault rupture. By defining and dating older sheared strata, and overlying younger post earthquake sediments, the timing of past fault ruptures and associated earthquakes can be estimated. Dating requires the presence of organic material to allow the use of ^{14}C radiocarbon methods but fortunately organic material is relatively common in the forested areas of Westland. The resolution possible with radiocarbon dating is limited but the timing of the last earthquakes can be bracketed within broad date bands.

The other three methods applied are made possible because previous earthquakes in rugged forested terrain in New Zealand and overseas have demonstrated the profound effects of earthquakes on slopes, rivers and forests in the epicentral area. Earthquakes commonly:

- Trigger landslides on sloping ground
- Cause liquefaction, subsidence, and river aggradation in alluvial areas
- Shake trees until some are broken or fall

Some of the landslides can be directly dated from radiocarbon dates from buried logs. In addition new forest tends to reestablish simultaneously in clear areas created by the earthquake damage leaving a potential record of the timing of the earthquake in the age structure of the forest. Many trees are able to survive the earthquake but still suffer root damage, broken branches and tilting. This is often recorded in their growth rings which potentially provide a very accurate way to estimate the timing and extent of earthquake disturbance.

With the additional expertise of Andrew Wells, a doctoral student from the Plant Science Department at Lincoln University, all four of these methods have been applied to the Alpine Fault between the Paringa River and Rahu Saddle. They produce a consistent record which indicates two recent earthquakes on the Alpine Fault in the last 500 years. Figure 1 summarises the data for the two most recent earthquakes.

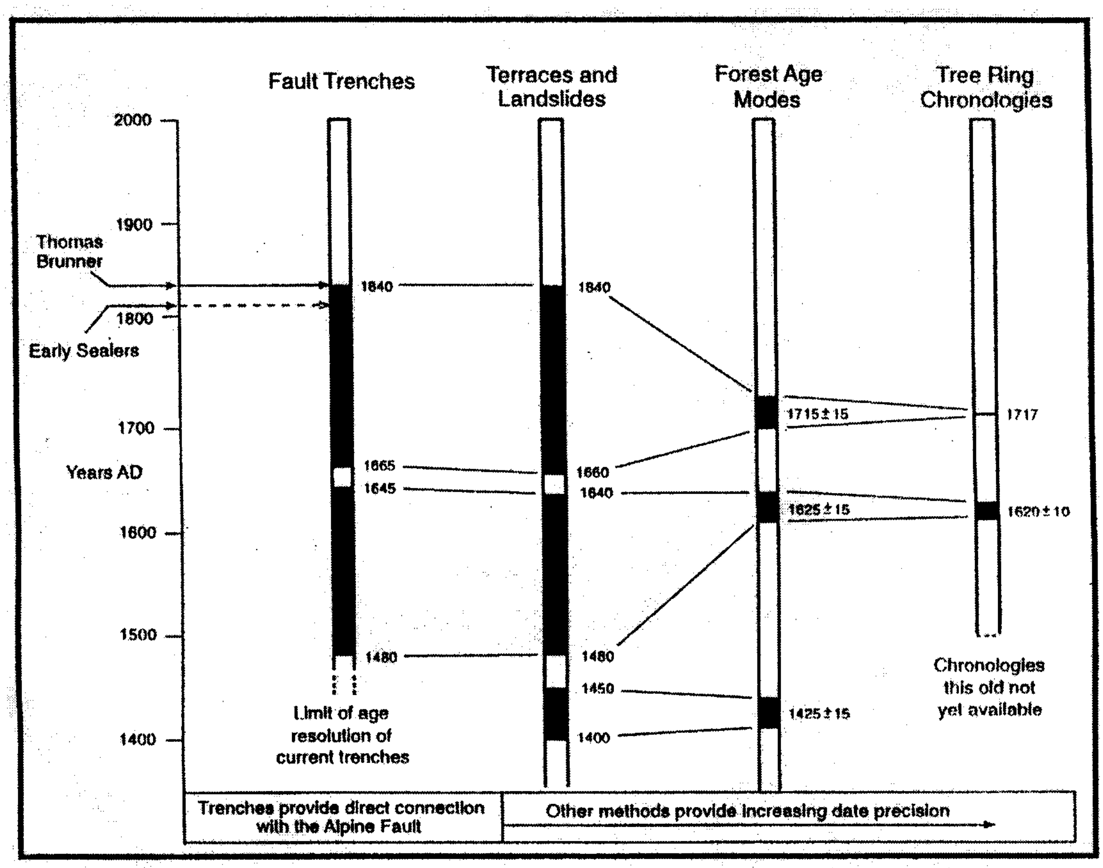


Figure 1: Summary of the four methods used to establish the timing and extent of the last two Alpine Fault earthquakes.

The most recent event appears to have taken place in 1717 AD and the surface fault rupture extended in length from Milford Sound to the Haupiri River, a distance of at least 375 kilometres. Approximately 100 years earlier, at around 1620 AD, another earthquake occurred in the north section of the fault and extended at least as far south as the Paringa River. Prior to this, another earthquake at around 1450 AD is suggested by the data, but this has not yet been recognised in trenches.

The implied pattern of earthquake recurrence is not regular but averages around 200 years, and varies from 100 years to at least 280 years, which is the lapsed time since the last earthquake. Probability estimates can be made using the record of Alpine Fault earthquake recurrence and a combined analysis of earthquake timing on similar plate boundary faults around the world. Other faults also exhibit a wide range in recurrence behavior, but for the Alpine Fault the probability estimates of the next earthquake are consistently high, with a probability of 65 ± 15 % over the next 50 years increasing to 85 ± 10 % over the next 100 years.

Based on the rupture length we estimate both of the most recent earthquakes were around Magnitude 8 and reconstruction's can be made of the most likely pattern of earthquake shaking intensity. Those earthquakes which also rupture the more northern portion of the fault, like the one around 1620 AD, have generally more impact on the main population centres and Figure 2 shows the estimated shaking pattern.

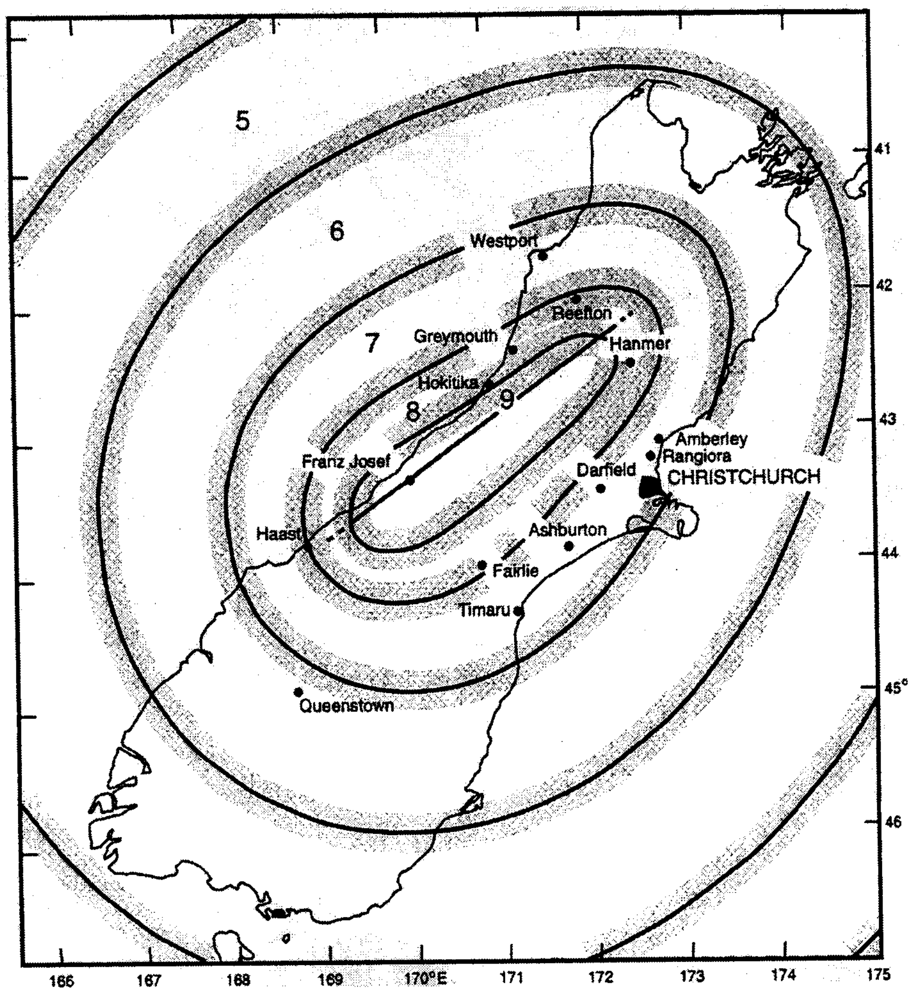


Figure 2: Estimated Modified Mercalli Intensity isoseismals (lines defining equal shaking intensity) for the Alpine Fault earthquake around 1620 AD. Modeled using the method of Smith (1995a & 1995b) and reproduced courtesy of Warwick Smith, Seismological Observatory (pers. comm., 1997).

The next Alpine Fault earthquake is likely to produce very strong shaking in locations close to the Southern Alps. In particular locations such as Arthurs Pass, Otira, Mount Cook and Franz Josef will be seriously affected. Hokitika and Greymouth will also be strongly shaken. Predicted intensities are generally less on the east coast but in virtually all central South Island locations the next Alpine Fault earthquake will be stronger than any other earthquake experienced there in the last 100 years. Figure 3 summarises the predicted intensities and compares these with those of other recent earthquakes.

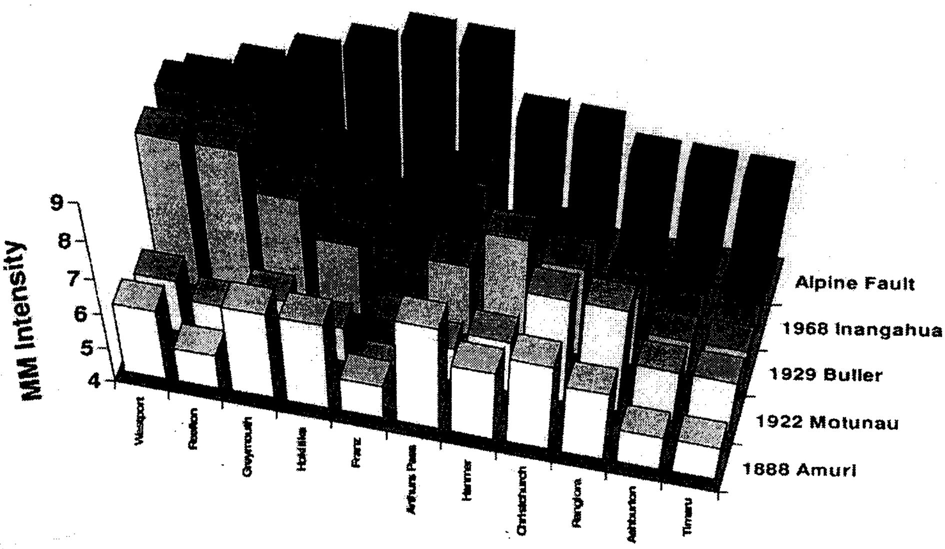


Figure 3: A comparison of predicted shaking intensities for the nest Alpine Fault earthquake with those experienced in other large earthquakes this century for a range of locations. The vertical scale shown here for 4 - 9 is the Modified Mercalli Intensity and has a highest possible value of 12.

Direct effects of the next earthquake will include landslides and liquefaction. It is likely some large rock and debris avalanches will be triggered in the Southern Alps, where the landslides will be most severe, and temporary landslide dams are likely to be created. Landslides will also be triggered in sloping ground in Greymouth and the east coast foothills. They are unlikely to be serious as far away as the Port Hills of Christchurch however the greater density of housing in this area may still result in significant property damage. Liquefaction in Christchurch is likely to cause more damage than landslides, because the city it is known to have susceptible soils, and it is well within the likely range of liquefaction. Liquefaction will also be widespread in Westland.

One of the most profound long-term impacts will be to the river regimes in river catchments draining the Southern Alps. Increased sediment load from landslide material entering the rivers will result in aggradation of the river bed and channel shifting, particularly in the upper catchment. This has potential implications for river control, bridging and hydroelectric generation.

Copies of the full report are available from the Earthquake Commission in Wellington.

AUSTRALIAN CENTRE FOR GEOMECHANICS

December 1998

TAILINGS MANAGEMENT AND DECOMMISSIONING
Perth, December 3 to 4, 1998

A practical ***course for site supervisory personnel and environmental officers*** consisting of case studies, special presentations and discussion sessions. The course will be structured around different case studies to address the conditions encountered in different mining operations and environments.

The two-day course addresses the issues facing site personnel involved with the day-to-day management of tailings storage facilities to comply with the relevant operating standards and decommissioning requirements. Special emphasis is to be placed on the discussion of decommissioning issues which are of increasing importance to mine site personnel.

A workshop session will be incorporated into the course. As specific case studies often avoid the "problems" encountered, this workshop is intended to be a forum for discussing the practical issues involved in managing a tailings facility in a way that will maximise the opportunity for discussion.

Registrants will have the opportunity to raise any suggestions they may have on this topic, or to seek advice on any problem they may have encountered in their day-to-day operations.

Presenters: S. Barker (formerly Coolgardie Gold), P. Williams (MPA Williams & Associates), M. Fahey (UWA Geomechanics), J. Reilly (Placer Pacific), B. Gilkes (UWA Soil Science), H. Jones (DME), T. Tyson (KCGM), N. Stockton (Curtin), C. Hillman (Kundana), H. Lacy (Outback Ecology), and others.

For further information or expressions of interest, please contact:

Christine Neskudla or Gillian Macmillan
AUSTRALIAN CENTRE FOR GEOMECHANICS
7 Cooper Street
NEDLANDS WA 6009
Ph: +61-8-9380 3300
Fax: +61-8-9380 1130
E-mail: acg@acg.uwa.edu.au

The 8th Australia New Zealand Conference on Geomechanics "Consolidating Knowledge"

15–17 February 1999
Hobart, Tasmania, Australia

The ANZ Conference on Geomechanics is held every four years in a city of Australia or New Zealand. It brings together practitioners, consultants, educators and researchers in the fields of soil mechanics, rock mechanics and engineering geology. The theme of the next conference, "Consolidating Knowledge", is aimed at reviewing past progress in geomechanics, describing current trends and exploring promising future directions for research.

VENUE

The conference will be held at Hobart's Wrest Point Hotel Casino, which offers a waterfront location not far from the centre of the city.

TECHNICAL SESSIONS

There are fifteen technical sessions planned over the three days of the conference, mostly in two parallel streams. Each session will highlight five or six papers. This provides for about 80 papers to be delivered by their authors, followed by discussion. Sessions will also be set aside for poster presentations of some 40 more papers, with the authors available for informal discussion.

Session topics include:

- Slope instability
- Expansive soils
- Piles
- Materials testing
- Foundations
- Roads
- Dams
- Numerical modelling
- Anchors and reinforcement
- Rock mechanics
- Environmental geotechnics

EXHIBITION

A trade exhibition will be held at the Wrest Point Hotel Casino over the three days of the conference. Any firms wishing to take part may contact the conference organisers for details.

POST-CONFERENCE TOURS

A choice of post-conference tours will be offered. Destinations include various lakes, dams, mines and power stations of West and South-West Tasmania.

ENQUIRIES

Dickensons Conference Management
PO Box 431 Moonah
Tasmania 7009
Australia
Ph. 61 3 6228 1932
Fax: 61 3 6278 2956
Email: travel@southcom.com.au

1999

FEBRUARY 15-17, 1999
Hobart, Australia
8TH ANZ CONFERENCE CONSOLIDATING KNOWLEDGE (see flier enclosed)

JUNE 7-10 , 1999
Amsterdam, The Netherlands
XIIth European Conference on Soil Mechanics and Geotechnical Engineering
Geotechnical Engineering for Transportation Infrastructure.
http://www.delfgeot.nl/xiiecsmge.99

JUNE 21-25, 1999
Lisbon, Portugal
2ND INTERNATIONAL CONFERENCE ON EARTHQUAKE GEOTECHNICAL ENGINEERING
Topics:
- Dynamic Characterisation of Soils
- Strong Motions
- Soil Structure Interaction
- Seismic Behaviour of Buried Structures
- Liquefaction
- Seismic Behaviour of Buried Structures
- Liquefaction
- Seismic Behaviour of Slopes and Embankments
- Seismic Design Criteria and Safety Evaluation
- Lessons Learned from Recent Earthquakes

JULY 19-21, 1999
Tokyo, Japan
INTERNATIONAL SYMPOSIUM ON GEOTECHNICAL ASPECTS OF UNDERGROUND CONSTRUCTION IN SOFT GROUND
Organised by ISSMGE, TC28
Theme: Case histories and other information concerning the design and construction of tunnels and deep excavations in the urban environment.
Abstracts by 31 July 1998.

AUGUST 8-12, 1999
Foz do Iguaca, Brazil
XI PANAMERICAN CONFERENCE ON SOIL MECHANICS AND GEOTECHNICAL ENGINEERING.

AUGUST 16-20, 1999
Seoul, Korea
11TH ISSMFE ASIAN REGIONAL CONFERENCE
Themes:
- Engineering Properties of Soils and Rocks
- Deep and Shallow Foundations
- Underground Excavation and Tunnelling
- Earth Structures and Slopes
- Natural Hazards and Ground Improvements
- Environmental Geotecnique
- Soil Dynamics and Earthquake Engineering
- Case Histories in Geotechnical Engineering

http://www.kict.re.kr/enghome/arcsmfe

AUGUST 25-28, 1999
Paris, France
9TH INTERNATIONAL CONGRESS ON ROCK MECHANICS (ISRM)
- Applied Rock Mechanics, Safety And Environment
- Coupling Between Mechanical, Thermal, Hydraulic
- Rock Dynamics And Tectonophysics
- Monitoring, In Situ Tests And Field Measurements

http://www.ensmp.fr/isrm99

SEPTEMBER 28-30,1999
Turin , Italy
2ND INTERNATIONAL SYMPOSIUM ON PREFAILURE DEFORMATION CHARACTERISTICS OF GEOMATERIALS – ISTORINA 99
Proposed Topics:-
- Innovation in Soil Testing
- Stress Strain Behaviour
- Applications and Case Histories

http://www.polito.it/inizlati/ist99

OCTOBER 6-9,1999
Guangzhou, China
INTERNATIONAL CONFERENCE ON ANCHORING AND GROUTING TOWARDS THE NEW CENTURY (ICAG)
Conference themes:-
- Anchoring and Grouting in the New Century
- Anchoring Techniques
- Grouting, Deep Mixing and Jet Grouting
- Basic Engineering Properties of Rock and Soil

OCTOBER 13-15, 1999
Stockholm, Sweden
INTERNATIONAL CONFERENCE ON DRY MIX METHODS FOR DEEP SOIL STABILISATION
ISSMGE, TC17
Topics include:
- Properties of Stabilised Soil
- Behaviour of Stabilised Soil
- Prediction and Performance
- Quality Control

OCTOBER 25-27, 1999
Durban, South Africa
12TH AFRICAN REGIONAL CONFERENCE
Geotechnics for Developing Africa.

NOVEMBER 8-11, 1999
Matsuyama, Shikoku, Japan
INTERNATIONAL SYMPOSIUM ON SLOPE STABILITY ENGINEERING
Topics include:
1. Site Investigation
2. Stability Analysis of Soil and Rock Slopes
3. Effects of Seismicity and Rainfall
4. Design Strength Parameters of Natural Slopes
5. Effects of Land Development
6. Slope Stability of Waste Materials
7. Stability of Landfills
8. Stabilisation and Remedial Works
9. Reinforced Steep Slopes
10. Probabilistic Slope Stability
11. Landslide Inventory and Landslide Hazard Zonation

Abstracts by 31 October 1998.

DECEMBER 15-17,1999
Bangalore India
INTERNATIONAL CONFERENCE ON ROCK ENGINEERING TECHNIQUES FOR SITE CHARACTERISATION
Conference themes:-
- Engineering Characterisation of Rock Masses
- Modern Techniques of Rock Mechanics Investigation
- Geophysical Imaging Techniques
- Mapping of Voids and Water Logged Workings
- Probing ahead of Tunnels
- Laboratory Testing Methods
- Numerical Modelling
- Rock Mass Improvement and Support Systems
- Instrumentation and Performance Monitoring
- Case Studies involving Special Geotechnical Problems

Abstracts by December 19 1998

2000

JAN-FEB 2000
Auckland, New Zealand
12th WORLD CONFERENCE ON EARTHQUAKE ENGINEERING
Technical Topic's:-
- Earthquake Risk Reduction in Developing Countries
- Earthquake Engineering in Practise
- International Issues in Earthquake Engineering
- Engineering Seismology
- Geotechnical Engineering
- Structural Engineering
- Lifeline Systems
- Design Criteria and Methods
- Social and Economic Issues
- Lessons from Recent Earthquakes

http:\\www.cmsl.co.nz/12wcee

FEBRUARY 14-18, 2000
Perth, Australia
4TH AUSTRALIA-NEW ZEALAND YOUNG GEOTECHNICAL PROFESSIONALS CONFERENCE
(see enclosed flyer)

JUNE 26-30, 2000
Cardiff, UK
8TH INTERNATIONAL SYMPOSIUM ON LANDSLIDES
BRITISH GEOTECHNICAL SOCIETY

OCTOBER 10-12, 2000
Hanover Germany
INTERNATIONAL WORKSHOP ON ENGINEERING GEOLOGY AND ENVIRONMENTAL PLANNING
Main workshop themes are :-
- Waste Disposal
- Mitigation of Natural hazards
- Water and Mineral Resources Development
- Ethics of Geoengineering

Abstracts by March 31, 2000
http://www.bqr.de/iaeg2000

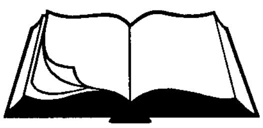
NOVEMBER 19-24, 2000
Melbourne, Australia
GEO ENG 2000 - INTERNATIONAL CONGRESS SPONSORED JOINTLY BY ISSMFE, IAEG AND ISRM

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

Publications of the Society

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

Publication Name	List Price Members	Non Members
New Zealand Geomechanics Society Conferences		
Proceedings of the Alexandra Symposium "Engineering for Dams and Canals" November 1983 (a joint symposia with NZSOLD)	**$10**	**$50**
Proceedings of the Auckland Symposium "Groundwater and Seepage" May 1990	**$10**	**$45**
Geotechnical Issues in Land Development Proceedings of Technical Groups Vol 22 Issue 1G Hamilton 1996	**$20**	**$35**
Proceedings of the New Zealand Geotechnical Society Symposium - "Roading Geotechnics" 98 Auckland 1998	**$20**	**$35**
Australia – NZ Conferences on Geomechanics		
Proceedings of the Sixth Australia – NZ Conference on Geomechanics, Christchurch, February 1992	**$50**	**$100**
Proceedings of the Third Australia – NZ Conference on Geomechanics, Wellington, May 1980	**$10**	**$30**
Other Publications		
Proceedings of the Second Australia- New Zealand Young Geotechnical Professionals Conference, Auckland, December 1995	**$25**	**$40**
Guidelines for the Field Description of Soils and Rocks in Engineering Use	**$10**	**$13**
"Stability of House Sites and Foundations – Advice to Prospective House and Section Owners"	**$1**	**$1**
Back Dated Issues of Geomechanics News (depending on availability)	**$0.5**	**$0.5**



NEW ZEALAND GEOTECHNICAL SOCIETY
1998 COMMITTEE

NAME	POSITION	ADDRESS	PHONE, FAX, EMAIL
Farquhar GB (Geoffrey)	Chairman	Worley Consultants Ltd P O Box 4241 AUCKLAND	09 379 1272 Work 09 379 1210 Fax gfarquhar@worley.co.nz
Fellows DL (Debbie)	Management Secretary	6 Sylvan Valley Ave Titirangi Auckland	09 817 7759 09 817 7035 Fax dfellows@xtra.co.nz
McPherson ID (Ian)	Wellington Branch Coordinator	Connell Wagner Ltd P O Box 1591 WELLINGTON	04 472 9589 Work 04 472 9922 Fax idm@ wel.conwag. co.nz
Bevin J (Jaime)		Foundation Engineering Ltd P O Box 256 OREWA	09 426 9707 Work 09 425 9709 Fax jbevin.fel@clear.net.nz
Murray J G (Grant)	Treasurer	Kingston Morrison Ltd P O Box 9806 AUCKLAND	09 520 8984 Work 09 520 4695 Fax 021 271 1992 Mobile jgm@ auck.km.co.nz
Grocott GG (Guy)	2000 Symposium, Christchurch Branch Co-ordinator	Riddolls & Grocott Ltd P O Box 2281 CHRISTCHURCH	03 377 5696 Work 03 377 9944 Fax Riddolls-Grocott@mactropolis.co.nz
Crawford SA (Steve)	Editor Geomechanics News, Stability Guidelines	Tonkin & Taylor Ltd P O Box 5271 AUCKLAND	09 366 6054 Work 09 307 0265 Fax scrawford@tonkin.co.nz
Riddolls BW (Bruce)	IAEG Australasian Vice President from January 1999	Riddolls & Grocott Ltd P O Box 2281 CHRISTCHURCH	03 377 5696 Works 03 377 9944 Fax Riddolls-Grocott@mactropolis.co.nz
Mostyn GR (Gary)	ISRM Australasian Vice President	C/- School of Civil Engineering University of NSW Sydney, AUSTRALIA 2052	61-2-9385 5021 Work 61-2-9385 6139 Fax g.mostyn@unsw.edu.au
Randolph MF (Mark)	ISSMGE Australasian Vice President	Centre for Offshore Foundation Systems Univ. of Western Australia Nedlands, WA 6907, AUSTRALIA	61-9-380 3075 Work 61-9-380 1044 Fax randolph@civil.uwa.edu.au

OBJECTS

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

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

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

MEMBERSHIP

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

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

ANNUAL SUBSCRIPTION

Annual subscriptions, which include the newsletter are:

Members *$50.00*
[IPENZ members receive a $15 rebate on their IPENZ subscription for belonging to the society]

Students *$25.00*
[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)***	*$20.00*
International Society for Rock Mechanics	***(ISRM)***	*$20.00*
International Association of Engineering Geology & the Environment	***(IAEG)***	*$15.00*
	(with bulletin)	*$42.50*

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: . . . .

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	(IAEG)	Yes/No
& the Environment	(with Bulletin)	Yes/No

<u>**DECLARATION:**</u> If admitted to membership, I agree to abide by the rules of the New Zealand Geotechnical Society

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

ANNUAL BASIC 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 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 ....

ADVERTISING IN NZ GEOMECHANICS NEWS

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

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

Advertisement Location	Single Issue	Advert. Size (mm's)
Back Cover	$275	2550 x 1100
Inside Cover (Front or Back)	$225	2550 x 1100
Full Page Internal	$200	2550 x 1100
Half Page Internal	$150	1700 x 1100
Quarter Page Internal	$125	650 x 1100
*Note: 1 All rates are excluding GST 2 Subject to availability		*All margins are 10 mm

The deadline for advertising copy for the next issue is 1 May 1999. Arranging artwork for your advertisement can be carried out at a reasonable additional cost if requested. However, advance notice is required for these additional services.

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

Management Secretary
Debbie Fellows
6 Sylvan Valley Ave
Titirangi
AUCKLAND
Tel: (09) 817 7759
Fax: (09) 817 7035
Email: dfellows@extra.co.nz

The Editor
Stephen Crawford
C/- Tonkin & Taylor
P O Box 5271
AUCKLAND
Tel: (09) 355 6054
Fax: (09) 307 0265
Email: scrawford@tonkin.co.nz

Situation Wanted

A third year student at the "L'Ecole Nationale Superirure de Geologie de Nancy (France) is looking for work experience opportunities for the period February to June 1999.

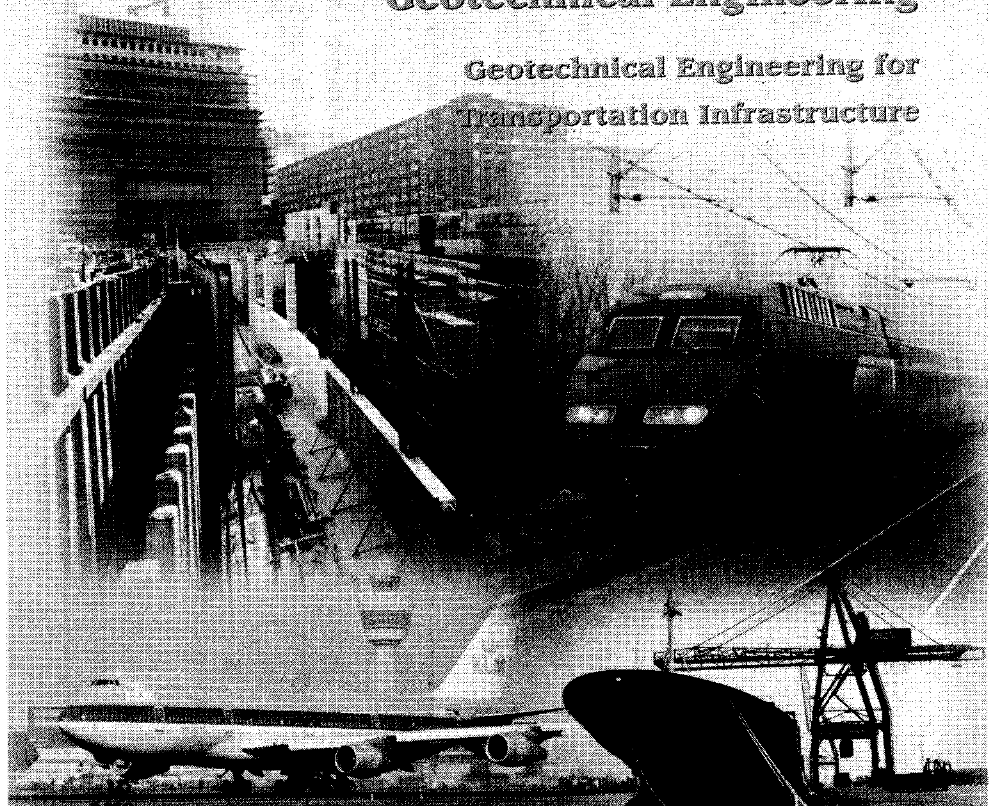
She is studying engineering and geotechnical studies and is fluent in English.

If you are interested in offering her an opportunity please contact the NZGS Management Secretary for further details.

Bulletin no. 2

XIIth European Conference on Soil Mechanics and Geotechnical Engineering

Geotechnical Engineering for Transportation Infrastructure



XII^{ème} Congrès Européen de Mécanique des Sols et de la Géotechnique

La Géotechnique dans les Infrastructures de Transport

Amsterdam, Pays-Bas
7 - 10 juin 1999

Amsterdam, The Netherlands
7 - 10 June 1999

Reply form / Coupon de reponsé

Please copy this page and fax to:
Merci de faire une photocopie de cette page et envoyez la par télécopieur au numéro suivant: **+3170 3919840**

I wish to receive Bulletin No. 3 (final programme, registration form and other details)
J'aimerais reçevoir le Bulletin No. 3 (programme définitif, bulletin de souscription et autres détails)

Mr/Mrs.: Madame/Monsieur: ______ First name: Prénom: ______

Family name: Nom: ______

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Mailing address: Adresse/Boîte Postale: ______

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City / Ville: ______

Country / Pays: ______

- ❑ I intend to participate in the Conference
  J'ai l'intention de participer au Congrès
- ❑ I wish to receive an Exhibition brochure
  Envoyez moi une brochure concernant l'exposition
- ❑ I intend to participate in the Technical Excursions
  J'ai l'intention de participer à une ou plusieures Excursions Techniques
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  Envoyez moi le Programme pour les Personnes Accompagnantes
- ❑ I am interested in Post Conference Tours
  Envoyez moi le programme des Excursions Post - Congrès

You can also contact us and register at our Internet homepage:
Vous pouvez aussi nous contacter et vous inscrire par Internet: **http://www.delftgeot.nl/xiiecsmge.99**