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N.Z. GEOMECHANICS NEWS

No. 57

JUNE 1999

A NEWSLETTER OF THE N.Z. GEOTECHNICAL SOCIETY

NZ GEOMECHANICS NEWS

THE NZ GEOTECHNICAL SOCIETY MAGAZINE

NO. 57 JUNE 1999

CONTENTS

NOTES FOR CONTRIBUTORS

EDITORIAL POLICY

NZ Geomechanics News is a magazine 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 Crawford
EDITOR

Debbie Fellows, Doug Johnson, Tony Cowbourne
ASSISTANT EDITORS

THIS IS A REGISTERED PUBLICATION

NZ Geomechanics News is a magazine 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 magazine. 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.

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BIA – BUILDING INCREASE AUTHORITY

Designed a simple retaining wall lately? Better do it quick – the price of retaining walls is due to rise soon, by about 20%. Is that news? Well may be not – Everyone's doing it - your local council (rates) power company (charges) and Metrowater. We're not talking about a rise in the cost of materials. The proposed new Building Industry Authority (BIA) revisions to the NZ Building Code, namely "VM4 – Foundations", will bring in limit state design – great! But for those of you still designing in working stress, the old minimum Factor of Safety (≥3) just went up by 20% to 3.6 when you apply the limit state factors to say a common retaining wall (with a minimal site investigation).

Check out the BIA draft revisions or easier still, try the following Letters to the Editor. Have your say to the BIA – but do it quickly. The draft is due to be finalised very soon.

HOEK BROWN MUST COME DOWN

"Based on experience to date, care must be used (in applying Hoek-Brown failure criterion) to closely jointed rock masses in New Zealand" - Stuart Read, Laurie Richards and Nick Perrin suggest lower rock material constants (m_i) for greywackes than those assigned by Hoek & Brown should be adopted. Also outlined in their enclosed paper is a modified empirical correlation with Rock Mass Rating (RMR) for assessing rock mass deformability. This paper will be presented at the 9th ISRM International (Paris) congress on Rock Mechanics.

This paper is part of ongoing research into NZ greywacke rock mass strengths. Immediately preceding their paper is a technical note calling for more data. Simply fill in the form provided - it shouldn't take more than a few minutes to complete – and forward it to Stuart Read at IGNS.

Geomechanics News is very pleased to support this novel method of data collection and look forward to publishing the results of Stuart & Laurie's further research.

THE SINCLAIR SOJOURN

Thank you Tim Sinclair for presenting the 10th Geomechanics Lecture at five different cities from Hobart (ANZ Conference) to Dunedin. While posing some fundamental questions (e.g. can the total negative skin function force generated on piles within compressible soils exceed the weight of the fill causing the compression?), Tim also touched on the development of powerful computing methods. In particular he outlined how he had been involved with the initial development of FLAC and large used strain deformation modelling to deal with complex or new geotechnical problems. He also pointed out that modern software can model the impossible, e.g. a pencil standing vertically on its end, where the laws of entropy will prevail. Chaos has the final word.

The 10th Geomechanics Lecture is presented here in *Geomechanics News* in the Technical Articles section.

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ROAD TESTING UNIT

Geotechnics offers a comprehensive road testing service which incorporates a wide range of testing applications from single lane unsealed rural accessways to multi-lane highways and motorways. The Road Testing Unit is purpose built for a range of IANZ registered services including:

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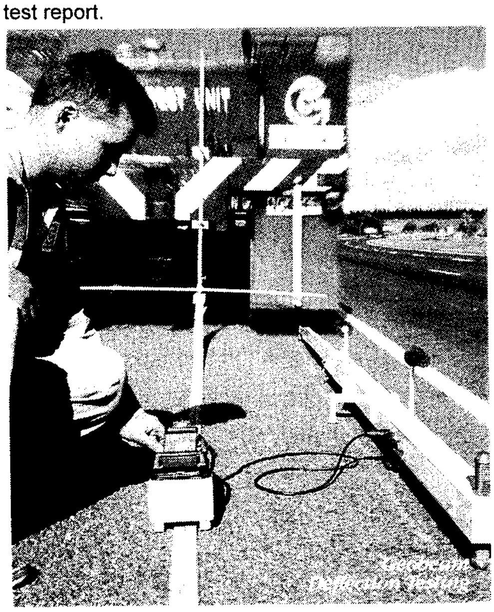
This service utilises a standard Benkleman Beam where pavement deflections are measured and recorded with preliminary results issued on site, followed up by a formal test report.



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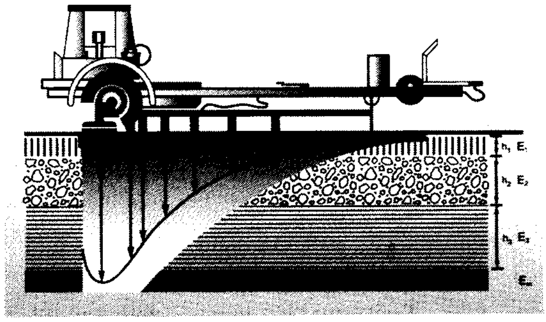

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SETTLEMENT PREDICTION COMPETITION

Twenty five responses in all – many thanks to all those who submitted their prediction. We have a winner – see article in Society News. The predictions varied by more than an order of magnitude and make for very interesting reading. Refreshingly, most entrants chose to be named.

Prof. Harry Poulos' keynote lecture from the Hobart ANZ Conference (see Technical Articles section) presents a timely and valuable summary of settlement analysis methods for shallow foundations (including rafts) and piles. The reliability of various methods is assessed and three prediction case studies are presented (see 3.3, 4.2 and 6.1 of Harry Poulos' paper). Prof. Mick Pender also compares the volcanic soils used for the prediction competition against those for recent sedimentary deposits – refer to the technical articles.

NEW EDITOR COMING UP

Eric Hudson-Smith has agreed to take over the role of editor for the next (December) issue of *NZ Geomechanics News*. I shall be retiring again, so I wish Eric well and trust that members will continue to support the magazine with the variety of contributions from members and advertisers.

Steve Crawford, EDITOR

RESEARCH ON LANDSLIP CLAIMS

Riddolls & Grocott Ltd recently completed a study for the Earthquake Commission (EQC) involving a review of selected landslip claim files to identify common geotechnical and development factors and associated regulatory matters. Relevant geotechnical information was selected to provide a basis for comparative assessment of landslip characteristics.

Analysis of the results showed while heavy rainfall was generally the trigger for failures, underlying geotechnical factors were also important.

Specific examples of claims illustrated both this fact and, in particular, that regulatory procedures were not always satisfactory. Under current legislation (Building Act 1991), the territorial authorities are responsible for ensuring design and construction requirements are carried out as intended. The potential exists for stability problems to go unchecked by consent processing staff who do not have local knowledge, geotechnical expertise, or access to a hazards register.

Many recent claims have resulted from the construction phase, e.g. there have been cases where a Construction Producer Statement was issued, and yet the slope subsequently failed. When checked, what was constructed was found to differ significantly from that which had been signed off. In other cases, cut slopes have failed because the construction procedure was not appropriate, e.g. done in one sequence, when a staged excavation should have been adopted.

An assessment of the present regulations governing building consents was made and the way these regulations are put into practice by territorial authorities. It was concluded that guidelines are needed to ensure rigorous procedures are followed by territorial authorities when granting building consents requiring consideration of slope stability."

The study also reinforced the requirement for suitably qualified geotechnical practitioners only to assess potential landslip hazards."

<u>Bruce Riddolls</u>

GIVE US A *tax* BREAK!

The following letter from a Trust Manager came across the Editor's desk, submitted by A. Nonymous.

Dear *Engineer,*

The Inland Revenue Department has queried whether the de-contamination of the ##### site recently carried out is in fact tax deductible.

In their opinion, it is not tax deductible but they consider we can claim annual depreciation (at 2%) on the value of the retaining wall as part of the work.

We now wish to endeavour to bring the concrete slab under this heading and thereby claim depreciation on an additional amount of approximately $70,000.

In order to do this, we have to establish whether the concrete slab was within any of the listed items under the Structures and Buildings list. A copy of the relevant schedule from the income tax Act is enclosed and we would ask whether you could classify the slab under any particular heading contained in that schedule. Accordingly, can this concrete slab be classified as a retaining wall lying horizontally instead of vertically?

<u>The ### Trust Manager</u>

Anyone venture to submit a reply? Anything with a slant on it will not be taken seriously – Ed.

DEVELOPMENT AT SNAIL'S PACE

[*ex NZ Herald, 20/4/99*)

Work on one of Ireland's major road improvement projects has been halted – by a snail.

The multimillion-dollar scheme is aimed at removing a notorious bottle-neck on the main route from Dublin to Cork and Limerick by constructing a bypass around the town of Kildare.

The hold-up was ordered after environmentalists claimed that the work threatened the natural habitat of a rare snail. The delay could be up to four years if the row goes to the European Union for a ruling.

LIMITED STATE FOR WALLS

There have been a number of occasions recently where the limit state factors for wall designs have been discussed at length by council and other engineers.

The current NZ Loadings Code (NZS4203) calls for soil loads (including earth surcharge and groundwater) on retaining walls to be treated as dead loads (Load factor: Lf =1.4 for dead load only, or Lf = 1.2 for dead load in combination with live load etc (refer clause 6.6.2). The strength reduction factor is nominated as 0.6 (refer clause 6.6.3).

The current NZ Building Code B1/VM4 calls for a Factor of Safety (FOS) of 1.5 for static (permanent) conditions, using unfactored loads, and FOS=1.2 for seismic conditions (refer clause 6.1.2). The strength reduction factors are outlined in Table 2 of B1/VM4 as 0.5 for static and 0.8 for seismic conditions. The proposed limit state revisions to the NZ Building Code (B1/VM4), dated 8 September 1998 call for:

- Load factor (Dead load), LfD = 1.0 for soil weight restoring force
- Load factor (Active load), LfA = 1.6 for active soil thrust

The first question arises for the case of assumed wall friction where the vertical component of the active thrust P_{av} is a restoring force, what is LfD? The example calculations (Page 36 of the draft revisions) show for P_{av}, the Lf = 1.6. Should this not be Lf = 1.0? This would be a more consistent approach and would be comparable to the old working stress approach. The use of Lf = 1.6 for P_{av} is *unconservative* (i.e. it increases the available resistance).

On the other hand, I have received comments from other geotechnical engineers that designing simple pole walls using the draft B1/VM4 revisions leads to more conservative designs because of the nominated strength reduction factors (Φ), namely:

Φ = 0.45 to 0.6 for Bearing Capacity & Passive Earth Pressure
Φ = 0.8 to 0.9 for Sliding (static & seismic)

Guidance is given in the draft as to which end of the range to select based on the sophistication of the site investigation, the extent of laboratory testing and the degree of construction control. However, for most small projects, Councils are likely to choose the lower end of the range as acceptable.

Where working stress design of retaining walls is used, a minimum FOS=3.0 was usually adopted for bearing capacity and sliding (with 100% passive earth pressure), and a minimum FOS=1.5 for sliding (with included 50% passive earth pressure). All things being equal, the "equivalent FOS's" using the proposed limit state design approach would be 3.6 (= 1.6/0.45) for bearing capacity and sliding (with 100% passive pressure) and 2.0 (1.6/0.8) for sliding without passive.

So the proposed design approach leads to more conservative designs, which is opposite to the intention of adopting limit state design.

We suggest the limit state approach should be amended so that for the basic case (where there is limited geotechnical information), the equivalent FOS's derived from limit state factors should mimic the working stress design FOS's. Equivalent walls (e.g. embedment depths, geometry and element sizes) should be then obtained using either limit state or working stress design. For example, all things being equal:

Lf = 1.0 for soil restoring forces (including P_{av})
Lf = ***either*** the greater of 1.4 for soil (active) thrust (ie. treat soil as a dead load)
or 1.2 (Soil Load) and 1.6 (Live Load – eg. effects of traffic or live surcharge loads)
Φ = 0.45 to 0.6 for bearing capacity and passive earth pressure
Φ = 0.7 to 0.8 for sliding
Φ = 0.8 to 0.9 for seismic conditions

More economic designs could then be achieved if there is better site investigation and laboratory/field testing data, and/or greater confidence of the site ground model is obtained based on past site experience (e.g. measurements, or previous walls on the subject site or nearby sites).

It does not make sense to introduce new design (code) methods which will result in more conservative designs unless there is ample evidence that evidence that existing walls which have been designed to widely accepted engineering practice (ie. working stress design) are inadequate or not fit for purpose.

Finally, there is the issue of slope stability. The existing B1/VM4 part of the code simply requires minimum FOS of 1.5 against instability – refer clause 3.2.2. (*This has been the subject of previous debate in Geomechanics News and a comprehensive review of stability requirements is proceeding - if somewhat slowly).*

The draft B1/VM4 revisions state that B1/VM4 "assumes general ground or slope stability" and "overall ground stability needs to be verified before (B1/VM4) can be applied: this is outside the scope of this verification method" (refer clause 1.0.4). This seems to replace the existing code clause mentioned above.

We suggest the existing code clause be retained as an interim measure but add in a clause to clarify the approach to acceptable groundwater conditions for the design of permanent slopes. A suitable clause is suggested on the following pages. *(This approach has been promoted to and accepted by a number of certifying bodies including most of the Auckland region (TLA) certifiers.)*

These issues need to be resolved soon as we understand the B1/MV4 revisions are due out very soon, and it could be a long time before the next revisions are made. We urge members to forward a short notice outlining their position to:

Dennis P Monastra
Building Industry Authority
P O Box 11846
WELLINGTON
Fax: (04) 471 0798
Tel: (04) 471 0794
Email: monastra@bia.co.nz

(A copy of correspondence to the NZ Geotechnical Society Chairperson would be appreciated – Ed.)

Stephen Crawford

SUGGESTED SLOPE STABILITY CLAUSES FOR B1/VM4 REVISIONS

Soil Strength Parameters Groundwater Assumptions and FOS

It is essential that an engineering geological assessment is carried out for the whole of the site, and extended area affecting the stability of any development.

A suitable geotechnical model must be formulated for analyses. This model should be referenced to drawings showing surface & subsurface site data.

If no detailed investigations have been undertaken, the requirement for a factor of safety (FOS) exceeding 1.5 for full saturation is generally reasonable.

A less conservative approach can be adopted where full saturation is only likely to occur under extreme conditions and a good understanding of ground conditions is available due to the:

- Detailed engineering geological mapping and subsurface investigations
- Groundwater conditions being defined by monitoring of water levels or geohydrological assessment
- Slope geometry being defined
- Defined drainage conditions including permeability of strata being well known
- Extent of recharge and catchment area being limited
- Back analysis of existing failures being carried out to determine soil properties or groundwater conditions.
- Soil properties being known within reasonable confidence limits. These should be compared to typical parameters for local materials based on published information and previous laboratory testing. If site specific tests indicate lower strengths then the lower bound soil properties should be used.
- Precedence of low incidence of instability in the area.

A minimum factor of safety of 1.5 is recommended for the conditions which may be expected to occur during the design life of the structure - 100 years for dwellings and 50 years for retaining structures beyond 8 m from the dwelling. A reduced minimum factor of 1.2 is applicable for extreme conditions. These extreme conditions include:

- Failure of stabilisation measures and drainage systems (provided the latter includes access for maintenance)
- Full saturation where investigations indicate that there is a high confidence level this condition will not occur during the design lifetime of the structure. ie. A check on full saturation may still be applicable to ensure that failure should not occur under this extreme condition.

Factors such as limited catchments, natural drainage conditions such as permeable strata and slope geometry may preclude full saturation under design conditions. In these cases a reduced groundwater level can be determined for the design case from extrapolation of monitored seasonal levels, seepage analyses or observation of geological evidence such as weathering, staining, etc.

The designer is responsible for providing convincing evidence that a reduced groundwater condition can be used for the design condition (ie. FOS ≥ 1.5). In such cases a check on the extreme design condition of full saturation, or failure of any installed slope drainage measures, is also required to confirm that the FOS > 1.2.

Variation from these factor of safety guidelines is possible but should be based on an assessment of the level of economic risk and risk to life. Such variations should be subject to specific geotechnical peer review and approval.

Earthquake Provisions

The design loadings for a numerical analysis of a slope affecting residences should be consistent with the zoning requirements of the NZ Loadings Code NZS 4203 and a 150 year return period. It is noted that section 4.11 of this code allows for a 0.25 structural performance factor for soil loads on structures rather than the 0.67 factor for building loads. A 50-year return period should be applied for retaining structures located further than 8 m away from a dwelling.

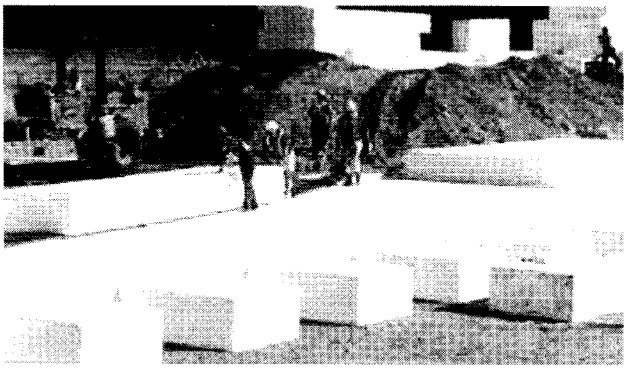
For numerical analyses of the seismic slope stability, a FOS ≥ 1.2 should be adopted for the above return periods. Potential slope failures that do not extend to within 8 m of a dwelling or cross a property boundary do not need to be analysed for seismic slope stability.

Stephen Crawford

James Hardie Building Systems

James Hardie Polyrock™ **is a purpose made expanded polystyrene (EPS) designed specifically for use in geotechnical applications such as bridge abutments and road construction projects.**

EPS has been successfully used in civil applications worldwide for over 25 years, being recently introduced to the New Zealand market.



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For further information on James Hardie Polyrock™, or any James Hardie Building Systems products, please phone Wendy Buck 025 431 739 or James Ewart 025 783 616.

Management Committee

The 1999 committee is listed below. Contact details are enclosed towards the rear of the magazine.

Geoffrey Farquhar	Chairman
Debbie Fellows	Management Secretary
Grant Murray	Treasurer
Ian McPherson	Membership Booklet, Wellington Co-ordinator
Jaime Bevin	Young Geotechnical Professionals Representative, Auckland Co-ordinator
Steve Crawford	Co-opted, Editor - Geomechanics News, Stability Guidelines
Guy Grocott	Co-opted, Symposium Organiser, Christchurch Co-ordinator
Bruce Riddolls	Ex officio, Australasian Vice-President IAEG
Mark Randolph	Ed-Officio, Australasian Vice-President ISSMGE
Garry Mostyn	Ex-Officio, Australasian Vice-President ISRM

QRA Seminar

The Society sponsored the seminar on "Quantitative Risk Assessment of Soil and Rock Slopes" held in Auckland in March. Robin Fell and Gary Mostyn were excellent presenters and the feedback received from the attendees was very positive. The only drawback was that numbers had to be limited (at the request of the presenters) and thus late registrants had to be turned away. The Society hopes to hold the seminar again in another year or two.

In holding the seminar the Society aimed to underwrite it and to offer significant discounts on fees to members. In the event nearly all participants were members. Following the success of the seminar and feedback from participants on other seminars and workshops that they would like to be held, the Society plans to host future similar events, probably one per year.

Life Membership for Professor Michael Pender

It was with much pleasure that the Society awarded Professor Pender life membership. A presentation in recognition of this achievement was made to Mick on 28 April 1999 following the 10th Geomechanics Lecture given by Tim Sinclair in Auckland. The award honours outstanding service to the Society which Mick has given over many years, stretching back to 1970 when he was the Management Secretary. He has also helped maintain the vital links between the Society and the Auckland School of Engineering.

Engineering Registration for Geologists

In the last issue on Geomechanics News I noted that engineering geologists with more than 10 years experience had applied for full membership of IPENZ and engineering registration. I know of one of those who was successful. The key element to success rather than being the primary degree is being able to demonstrate an adequate level of engineering design and responsibility in one's work. While the Society is still working with the universities to get accreditation of existing engineering geological degree courses, it is now over to engineering geologists who want to become registered engineers to apply and go through the professional interview process. Those interested should contact IPENZ in Wellington for information and application forms. For those interested, I can also put you in touch with those who have been successful.

Incorporation of the NZ Geotechnical Society

The Society management committee is currently investigating the process of setting up the Society as an incorporated society, and advancing from our present position of a technical sub-group of IPENZ. We will of course maintain our links with IPENZ but the feeling is that incorporation is likely to better define our financial position and our role and requirements with IPENZ. The management committee will review the progress and current status of two other technical sub groups who are now incorporated – the NZSEE (Earthquake Society) and SESOC (Structural Engineering Society). Any proposed changes to the current NZ Geotechnical Society will be reported to members and will of course need to be voted on at an AGM to bring about incorporation.

I would welcome any input or views from members.

50% Discount on Subscription as for First Year

The Society has recently voted in a 50% reduction in membership fee for a full member in their first year. This measure is designed to increase our membership further. Please advise any geotechnical or civil colleagues who may not be current members. Existing fees are low as compared to other technical societies. This discount does not apply to students who already get significant reductions in Society subscription fees. Of course members who rejoin after an absence do not get the 50% discount.

Geoff Farquhar
CHAIRMAN

Society Membership and Subscriptions

Society membership is currently flourishing with a total of 429 members. Whilst this is great to see, it is a shame that at least 20% of the membership have not yet paid your subscriptions. These members are encouraged to pay outstanding subscriptions as soon as possible. If any members are in doubt as to whether they owe monies to the Society, please contact me on Tel: (09) 817 7759; Fax: (09) 817 7035 or email: dfellows@xtra.co.nz

Members should note the recent rule change to Rule 8.4 now requires members who are more than one year in subscription arrears to be removed from the membership – refer item 8 in the following March 1999 AGM minutes.

New Members

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

Osama Abu-Iibda	Peter Diprose	Hugh McNaughton	Christopher Simpson
Nathan Brown	Stuart Finlan	Craig Ridgley	Alison Sprott
Glen Budden	Sunil Gulati	Lyndon Sanders	Walter Starke
Paul Burton	Robert Halton	Gregory Shaw	Matthew Wiley
Bryce Carter	Prakash Joshi	James Shorten	Darryn Wise
Maxwell Cope	Shane Lander	Christine Simpson	Eric Vautier
Rex Corlett	Michael Laws	Raymond Smith	

Resignations

W K F Brown , J H Dennett, and Philip Clarke have tendered their resignations from the Society. If any members wish to resign or decide not to continue their subscriptions, please advise myself or the Chairman, Geoff Farquhar. The Society has an obligation to pay international Society subscriptions in advance of New Zealand members who pay their subscriptions late. Adequate notice of your resignation helps avoid this problem.

Orbituaries

The Society sadly acknowledges the recent deaths of Peter North and Brian Paterson (see Obituaries later in the magazine).

Membership Booklet

The membership booklet is currently being compiled by Ian MacPherson and Debbie Fellows and should be reaching members shortly. The format will be modified slightly and will include email addresses (where available) and an updated set of rules for the Society.

Society Rule Changes & AGM

There have been some recent changes to the Society's rules as approved by the last AGM. A copy of the AGM minutes is attached as a record of that meeting.

Debbie Fellows
MANAGEMENT SECRETARY

MINUTES OF ANNUAL GENERAL MEETING OF THE NEW ZEALAND GEOTECHNICAL SOCIETY

HELD AT THE SCHOOL OF ENGINEERING, UNIVERSITY OF AUCKLAND

on Wednesday 24 March, 1999
6:00 pm

1. PRESENT :

P Morgan	G Boswell	P Lilley
P Rayudu	T Sinclair	E Hudson-Smith
W Prebble	N Al-Alusi	W Starke
A Langbein	R High	B Thomson
A W Smith	J Bevin	M Stewart
S Landen	D Stewart	M Leingaleos
T Wallis	G Murray	K Moody
A Ferguson	N Kay	P Burton
P Riley	R Burchett	D Fellows
G Farquhar	S Crawford	D Burns

2. APOLOGIES :

S Read	T Kortegast	I McPherson
P Millar	T Kayes	G Grocott
D McGuigan	P Kelsey	S Terzaghi
C Newton	T Adahaki	

3. MINUTES OF 1998 AGM

Motion put that Minutes of 1998 meeting be adopted as a true and correct record.

G Farquhar/J Bevin

Motion carried

4. MATTERS ARISING

No matters arising.

5. PRESENTATION AND ADOPTION OF CHAIRPERSON'S REPORT

Report attached to Notice of Meeting was tabled and presented by Chairperson (G. Farquhar).
Motion put that the Chairman's report be accepted.

D Fellows/G Murray

Motion carried

6. MATTERS ARISING FROM THE CHAIRMAN'S REPORT

None

7. PRESENTATION AND ADOPTION OF ACCOUNTS

Report prepared by Treasurer (G Murray) was attached to the Notice for AGM.

Motion put that accounts be received.

G Farquhar/ T Sinclair

Motion carried

8. CONFIRMATION OF 1999/2000 MANAGEMENT COMMITTEE

With the new committee structure two members needed to step-down for this election. They were I. McPherson and G. Murray. Both were prepared to stand for re-election. The call for nominations for the committee produced no response and G Murray and I McPherson are re-elected unopposed.

The following Management Committee was confirmed :

G Farquhar	Chairperson
D Fellows	Secretary (appointed position)
G Murray	Treasurer
I McPherson	
J Bevin	

Ex officio

B Riddolls	Australasian Vice-President IAEG
M Randolph	Australasian Vice-President ISSMGE
G Mostyn	Australasian Vice-President ISRM

Two co-opted members are to be confirmed at the next management committee meeting.
Chairperson moved that this Committee be adopted.

G Farquhar/T Sinclair

Motion carried

9. VOTE ON RULE CHANGES

The new wording for the rule changes was forwarded to all members with the notice of the AGM.
Rule 8.2 - a 50% reduction in membership fee for a full member in the first year.
Chairperson moved that the rule change be accepted.

G Farquhar/G Murray

Motion Carried.

David Stewart raised a question if this discount could be applied to student fees also. Chairman explained that the Society would not cover cost if the student fees were reduced any further.

Rule 8.4 - members in arrears will be removed from membership after 1 year. Chairperson moved that the rule change be accepted.

P Riley/E Hudson-Smith

Motion Carried.

10. VOTE FOR NEW LIFE MEMBER

There are currently 5 life members of the society, 4 still alive. The chairperson proposed Prof. Mick Pender as a new life member in recognition of his services to the society and to the industry.

Chairperson moved that M Pender be accepted as a life member.

G Farquhar/W Prebble

Motion Carried.

11. GENERAL BUSINESS

None

Meeting closed at 6:18 pm

G Farquhar (Chairperson): ______________________

Date: ______________________

ISSMGE Activities: October 1998 – March 1999

A Board meeting of ISSMGE was held in Paris in March 1999, and the following issues were discussed:

(1) **Guidelines are to be drawn up for organisers of conferences** held under the auspices of the ISSMGE (mainly International, Regional and TC-generated conferences) with regard to:
   (a) extent of hospitality or concessions to selected participants (such as keynote speakers etc), in order to avoid the risk of running at a loss;
   (b) reduced conference fees (for example, through shared registration fees for groups) for participants from developing countries, and also for full-time students.

(2) **The new web-site (www.issmge.org) will gradually replace the current ISSMGE newsletter,** and also provide a vehicle for accessing details of individual members. In order to avoid problems of personal privacy, it will be necessary for each member society to include a check-box on subscription forms, allowing public access to email address (and other relevant information). As a first step both NZGS & AGS should endeavour to assemble a list of members accessible through our own web-site.

(3) **The principle of a tiered voting system was agreed to**, probably along the lines of 1 vote for 1 to 50 members, 2 votes for 50 to 750 members, and 3 votes for over 750 members. The next stage will be to present a proposal to all member societies, with a view to debating the issue and agreeing a change in statutes at the Council meeting at Istanbul in 2001.

(4) **Separate Heritage Museums have been proposed from Austria, Germany and Turkey**. There was support for these initiatives, both as a means of recording historical data and exhibiting early equipment, but also as an educational facility for the general public.

(5) **The Young Geotechnical Engineers Award**, to be presented for the first time at the Istanbul conference in 2001, is to be worth GB£400. Final guidelines for the award will be circulated later this year, but it will be based on a paper submitted to the previous regional conference (e.g. Hobart) and the international conference (Istanbul), together with a cv and supporting nomination. Up to 3 awards are anticipated at each international conference.

(6) **The first international conference for Young Geotechnical Engineers** will be held in Southampton, UK, in September 2000. A sub-group of the Board is preparing a list of developing countries where travel assistance will be made available for a delegate to attend the conference.

(7) **Corporate Membership of the ISSMGE:** Mr Springall, Vice President for North America, presented an initial document on Corporate Membership of the ISSMGE. The next stage is to form an action plan to increase the extent of Corporate sponsorship of the ISSMGE. Professor Jamiolkowski is preparing a letter to promote Touring Lecture courses on different aspects of construction and design in geotechnical engineering.

(8) **The geotechnical database, SGI-line (http://public.sgi.geotek.se/sgi-line.html),** is now only available on payment of a subscription of US$250 per annum for ISSMGE members (and US$500 for non-members) . The ISSMGE will provide 24 free subscriptions in areas of need, allocated 6 each in Asia, Africa and South America, 5 in Europe and 1 in Mexico.

(9) **Manual for preparation of technical papers:** the Secretary-General elect, Professor Neil Taylor, is preparing a manual for preparation of technical papers, to be presented to the Council meeting in Amsterdam.

There will be a brief Board meeting in Amsterdam, in June 1999, immediately before the Council meeting on Sunday 6th June. Australia will be represented by Max Ervin at the Council Meeting (and by myself as Vice-President).

Mark Randolph
VICE-PRESIDENT (AUSTRALASIA), ISSMGE
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	Canada	Dr R. Phillips	Prof. M. Randolph	+61 8 9380 1044
TC-4 Earthquake Geotechnical Engineering	Portugal	Dr.P. Seco E. Pinto	**Prof. M. Pender (NZ)**	**+61 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 Geosynthetics and Earth Reinforcement	Japan	Prof. H. Ochiai	Prof. M. Hausman	+61 2 9330 2633
TC-10 Geophysical Site Characterisation	Sweden	Dr.R. Massarsch	Mr B. Whiteley	+61 2 9888 9977
TC-11 Landslides	Canada	Dr. J. Locat	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-14 Offshore Geotechnical Engineering	Norway	Dr. S. Lacasse	Prof. M. Randolph	+61 8 9380 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 P.T. Cruz	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
TC-34 Deformation of Earth Materials	Greece	Prof Vardoulakis	Dr Hans Muhlhaus	+61 8 9389 1906

IAEG Activities : Dec 1998 – May 1999

Hans Cloos Medal & Richard Walters

We have received a letter dated 29/3/99 from the Secretary General calling for nominations for the IAEG's two premier biannual awards for excellence in engineering geology.

1) Hans Cloos medal - for internationally recognised work at a senior level.

2) Richard Wolters prize - for achievement by a younger member (under 40)

Nominations are normally made by National Groups and should include CV, list of publications, and a statement of support up to 2 pages long. These should be sent to the Secretary General ***before 30th June, 1999.***

Nominations will be considered by the IAEG Executive Committee at its meeting in Kathmandu on 26th September 1999, and presented at the General Assembly during the International Geological Congress at Rio de Janeiro in August 2000.

Would members please consider who might be appropriate nominees for either award. With GeolEng 2000 following closely after, it would be a bonus for recognition of our region if one of our members were to win an award.

Australasia

An informal gathering was held during the Hobart Conference to determine interest in actively advancing the practice of recognition of engineering geology in the region. There was a feeling amongst AGS members, in particular, of a need for this.

A subsequent outcome has been a move to establish an engineering geological liaison position on the AGS Management Committee. This person together with the IAEG VP will develop a regional strategy to further engineering issues.

An outline on these issues is presented following this report.

Bruce Riddolls
VICE PRESIDENT (AUSTRALASIA)
INTERNATIONAL SOCIETY FOR ENGINEERING GEOLOGY and 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:(geo@rgl.co.nz) ***tel. (64-3- 377 5696) or fax (64-3- 377 9944).***

IAEG IN AUSTRALASIA: BENEFITING PRACTITIONERS

1. Background

In addition to membership of National Groups (NZ Geotechnical Society, Australian Geomechanics Society), the International Association of Engineering Geology and the Environment (IAEG) provides professional opportunities for engineering geologists. The IAEG was founded in 1964 and is a world-wide scientific society with more than 6000 members.

Its aims are: -

- to promote and encourage the advancement of Engineering Geology through technological activities and research –
- to improve teaching and training in Engineering Geology and
- to collect, evaluate and disseminate the results of engineering geological activities on a worldwide basis.

Major activities are:

- Publication of the Bulletin of Engineering Geology and the Environment, an established international refereed journal, now four times a year.
- A Newsletter sent twice a year to all Members.
- Commissions led by eminent world specialists to review the state of the art on various topics.
- Sponsoring international meetings. The IAEG Congress is held every 4 years in an invited country. IAEG activities are also organised on the occasion on the International Geological Congress. In addition, the IAEG sponsors various international or regional symposia organised by National Committees on specific topics.

2. Australasian Membership

Membership is normally via national groups, although individual membership is possible. There are currently about 140 members in New Zealand and 240 in Australia; no figures are available as to what proportion are active engineering geology practitioners compared with membership out of general professional interest.

3. Current Regional Activities

The 4-yearly Australia-New Zealand Conference on Geomechanics is the only regional forum for engineering geological practitioners. Apart from conference proceedings, there is no regional publication.

4. Future Regional Activities

At the recent 8th ANZ Geomechanics Conference in Hobart, an informal gathering was held to determine interest in actively advancing the recognition and practice of engineering geology on a regional basis to take advantage of greater numbers. In the first instance, it is proposed to establish an email network amongst practitioners to help members keep up to date on international matters, and regional communication on practice, technical and professional issues. Please e-mail me if you would like to be part of the network.

Also an engineering geological liaison position has been established on the AGS Management Committee. Dr Fred Baynes, Baynes Geological Pty Ltd, Perth is to take on this function. He and I will co-ordinate the development of a strategy to progress regional engineering geological issues.

Please send me or Fred (fredb@iinet.net.au) your comments and ideas on what regional activities you think might be helpful, e.g.:

- meeting or workshops
- technical standards
- working parties
- professional development
- registration
- education issues

5. Membership Enquiries

Enquiries on IAEG membership may be made as follows

NZ Geotechnical Society: dfellows@xtra.co.nz
Australian Geomechanics Society: vlee@ieaust.org.au

Bruce Riddolls
VICE PRESIDENT (AUSTRALASIA)
INTERNATIONAL SOCIETY FOR ENGINEERING GEOLOGY & the Environment

Ph: 64 3 377 5696; Fax: 64 3 377 9944; e-mail: bwr@rgl.co.nz

ISRM Activities : Oct 1998 – April 1999

Introduction

This report will provide an overview of ISRM matters over the period of October 1998 to March 1999. Neither the Board or Council have met since my last report in October 1998 nor thus there is not much to report. AGS and NZGS action will be 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 Congres between Wednesday 25th and Saturday 28th August, 1999.

The Australian papers have been submitted. Authors and titles are shown on the attached table. As members will know, there is a limited number of pages allocated to each National Group. Four promised papers were not received from the Australian National Group, this was disappointing as this was not apparent until well after the due date. Thus little could be done to redistribute the page allocation and Australia did not fill its allocation. This impacts upon our allocation at future congresses.

New Zealand has one paper submitted. *(A copy of this paper by Read et al is included in this issue of Geomechanics News – Ed.)*

Interest Groups

Each National Group should expect to receive soon a mail ballot on the establishment of Interest Groups within the ISRM.

Language

There will be a vote in Paris on changes to our language statutes, if successful, it will effectively establish English as the primary language of the Society. With conferences and symposia, other than Congress, also having the host country language as a language.

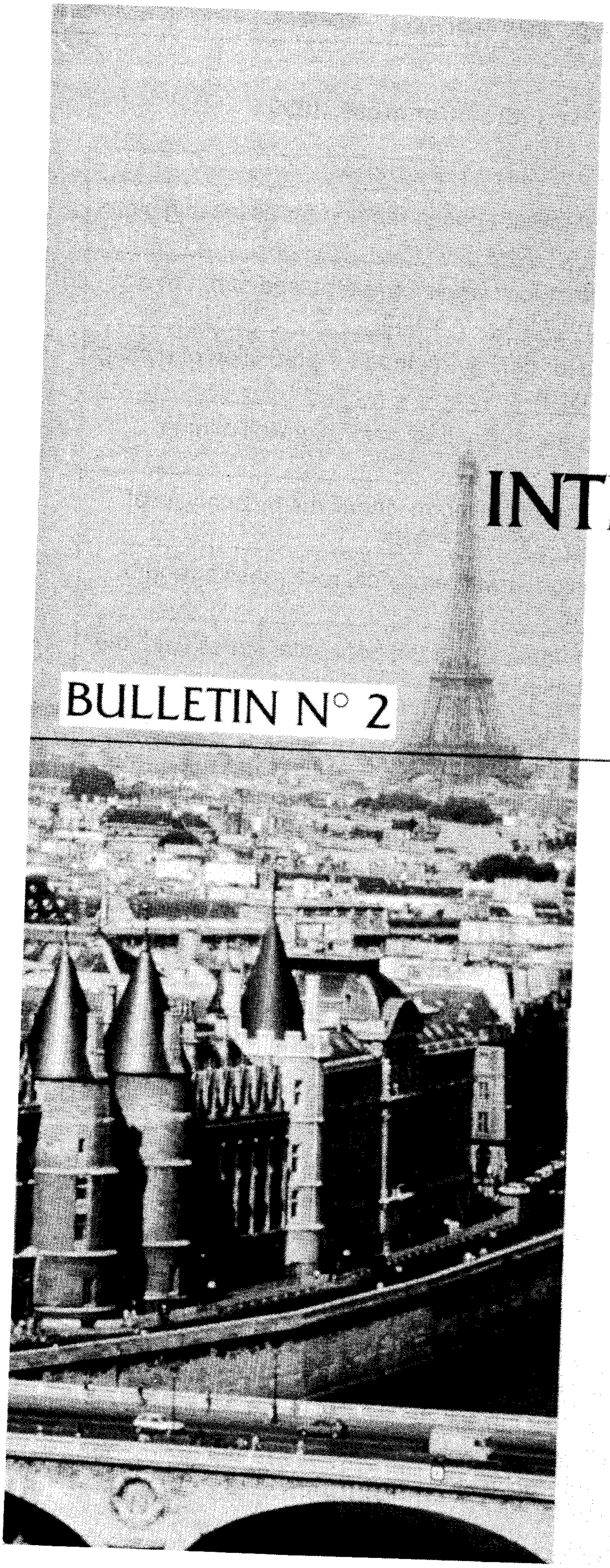
Garry Mostyn
Vice President for Australasia
International Society for Rock Mechanics

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).

Papers submitted to the Paris ISRM Congress, 1999

Author(s)	Title
Baczynski *Not received*	STEPSIM4: Step-path based method for assessing slope stability risks
Barr, Jupe & Junt	The Kaiser effect for samples pre-stressed at 820m and 2.4km with stress tensor results
Beck & Brady *Not received*	A moment tensor density method for prediction of mining induced seismic risk
Duncan Fama, Shen, Craig, Kelly, Follington &	Layout design and case studies for highwall mining
Duplancic & Brady	Characterisation of caving mechanisms by analysis of mine seismicity and rock stress
Duran & Douglas	Do slopes designed with empirical rock mass strength criteria stand up?
Enever, Gale & Fabjanczyk	A study of the variability of the horizontal stress field in a sedimentary basin
Galvin, Hebblewhite & Vasundhara	Geomechanics developments in mining coal under strong roof and weak floor conditions
Glastonbury, Mostyn & Fell	Analysis and prediction of pre collapse deformation of cut rock slopes
Indraratna, Ranjith & Gale	Deformation and permeability characteristics of rock with interconnected fractures
Kelly, Luo, Hatherly, Balusu, LeBlanc-Smith & Gale	Ground behaviour about longwall faces and its effect on mining
Lee & Mikula *Not received*	Estimation of the large scale insitu shear strengths of geologic structures at Mt Charlotte Mine, Kalgoorlie, WA
Pearce & Haberfield	Laboratory direct shear testing of large scale fractal joint profiles under constant normal stiffness
Tan, Choi & Richards	Coupled Physico-chemical and thermoporoelasticity mechanisms in shale
Windsor *Not received*	Systematic design and reliability assessment of rock reinforcement and support for jointed rock masses
Wold, Choi, Wood, George & Williams	Mining beneath a gorge: induced fracturing and the release of reservoired gases
Wu, Cox, Addis and Tan	Computer aided stress and strain paths test and their effects on mechanical behaviour of rocks
Read, Richards & Perrin **New Zealand contribution**	Applicability of the Hoek-Brown failure criterion to New Zealand greywacke rocks

9th ISRM congress
PARIS 1999



BULLETIN N° 2

9th INTERNATIONAL CONGRESS ON ROCK MECHANICS

Leçons du 20ème siècle
Défis du 21ème siècle

20th Century Lessons
21st Century Challenges

Erfahrungen des
20. Jahrhunderts-
Herausforderungen des
21. Jahrhunderts

PARIS
FRANCE

AUGUST 25-28
1999

AUCKLAND BRANCH ACTIVITIES

Groundwater Gremlins at Lake Coleridge Power Station

David Burns & Geoffrey Farquhar, Worley Consultants Limited

Following the NZGS AGM on 24 March, David and Geoff gave a presentation on the unique geological and hydrogeological setting and associated geotechnical hazards at the Lake Coleridge Power Station in the Southern Alps.

Ground Improvement Techniques

Mr Norbet Bos, General Manager, Keller Grundbau, Germany

On 29 March, Mr Norbet Bos gave a presentation outlining the ground improvement techniques of vibro-replacement, vibro-compaction and jet grouting. The presentation also touched on many case studies and examples from around the world. Mr Bos was in New Zealand at the invitation of Brian Perry Limited and Frankpile Australia Limited (a Keller Company) who were joint venture partners for foundation construction and ground improvement on the Wellington Stadium Project.

10th NZ Geomechanics Lecture
Geotechnical Analysis: Fundamentals to Fractals

Tim Sinclair, Tonkin & Taylor Limited

On 28 April, Tim Sinclair presented the 1999 Geomechanics Lecture. The presentation examined methods of analysis in geotechnical engineering, the use of numerical modelling and the presentation of some case histories. The evening was well received and attended by over 80 people.

1999 Programme

The rest of the 1999 programme is still being finalised, however, meetings are planned for every 4 to 5 weeks. Preliminary topics for this year are:

- Clifftop stability, cliff retreat, building setback lines and coastal erosion of the Auckland Region
- Regional Growth in the Auckland Area
- Auckland Airport Pavement Construction
- Reinforced Soil Walls/Segmental lock Retaining Walls
- Lessons learnt from retaining wall failures
- FoRST Research at the Department of Civil & Resource Engineering, University of Auckland
- Student Prize Evening

Confirmation of meetings will be by monthly flyer and the Auckland IPENZ Branch monthly bulletin. The usual venue is the main lecture theatre, Room 1.401, School of Engineering or another lecture theatre if 1.401 is not available. Both Maccaferri NZ Limited and Ground Engineering Limited have kindly offered to continue their sponsorship of the Auckland Branch meetings.

Young Geotechnical Professional Group

I am also trying to start a Young Geotechnical Professionals (YGP) Group in Auckland. The aims of this group would be to represent the interests of YGP's in the NZGS, provide a forum for networking, expand and strengthen the lines of communication between YGP's and to assist in the professional development of YGP's.

The success of this group will largely depend upon the support of those of you out there, so if you're a geotech person with less than 7 years experience and interested in meeting other YGP's, please get in contact with me (details below). Planned events include:

- Invited speakers on topics such as career and professional development
- Talks by senior engineers, sharing some of their past experiences
- A World Series Cricket match between the YGP's and the 'mature' geotechnical professionals
- Organised Field Trips and Social Functions

Jamie Bevin
AUCKLAND BRANCH ACTIVITIES CO-ORDINATOR
Ph: (09) 426 9797; Fax: (09) 426 9709; email: jbevin.fel@clear.net.nz

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WELLINGTON BRANCH ACTIVITIES

The Wellington Branch has a steady supply of speakers since the last Geomechanics magazine although the support from the members has been disappointing varying from 5 to 13 people. Mick Pender and Vaughan Meyer spoke about CPT research work at the University of Auckland. We had a second talk in December when Professor Yamagami of the University of Tokushima spoke about landslide control works in Japan. While there was only eight people attending there was considerable interest in the talk and the discussion and question time went on until 8:30 pm or so.

We have had one talk this year with Russ van Dissen speaking about recent findings of seismicity and faulting around Wellington. Thirteen people turned out for this talk which was most encouraging and a reflection of the interest in this topic. I am working on the programme for the rest of the year with still aiming at a talk at six to eight weekly intervals. Tim Sinclair delivered the 10th Geomechanics Lecture on 26 April and Mike Crozier was due to talk in May (as the magazine goes to print).

We are currently meeting at the IPENZ rooms but they charge us $40 per night. This becomes quite expensive, especially when only a few people bother to turn up. Future meetings will therefore most probably be held at Connell Wagner's offices in Thornton St. However, I am open to suggestions for alternative meeting locations.

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

A local branch meeting was held on Wednesday 21 April 1999 when Mr Clive Anderson gave a presentation on:

i) investigations for the new Port Hedland, Western Australia car dumper excavation and conveyor tunnel, and

ii) investigations, analysis and remedial works design for a major rock slope failure in east Java, Indonesia.

Both accounts emphasised the importance of establishing the appropriate site geological model as a basis for design and construction.

The Christchurch local branch is organizing another site visit to inspect the works of the $25 million Otira Viaduct project being constructed by McConnell Smith Ltd. on behalf of Transit New Zealand, on 15 May 1999.

Guy Grocott
Christchurch Branch Co-ordinator
Ph: (03) 377 5696; Fax: (03) 377 9944; E-mail: geo@rgl.co.nz

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OTAGO/SOUTHLAND BRANCH ACTIVITIES

10th Geomechanics Lecture

The Otago/Southland Branch has been relatively quite over the last six months. The 10th NZ Geomechanics Lecture was presented by Tim Sinclair (Tonkin & Taylor Limited). Tim gave an interesting and informative presentation on "Geotechnical Analyses: Fundamentals to Fractals" that was well received by the audience. Tim began by looking at the development and use of numerical methods in geotechnical engineering, and concluded by highlighting the need to return to fundamentals and thereby gain an understanding of complex problems.

Horseshoe Bend Hydro Electric Project
– report by Ian Walsh

The Horseshoe Bend Project involves the construction of a small hydro electric power station on the Teviot River. some 160 km from Dunedin. The developer is Central Electric Ltd who already own and operate several other small hydro power stations on the Teviot River and at other locations in the Central Otago area.

The site is located some 500 m above sea level and the project is a typical of small hydro schemes in the area which includes site access roads, a dam, tunnel, pipeline and penstock, and powerhouse. The gross head of the scheme, is some 93 metres. The project program is on target to produce electricity before the end of May 1999. Central Electric engaged Fulton Hogan Central as their Project Manager, and Opus were selected as the Design Consultant.

The river gorge is cut into relaxed quartzofeldspathic schist with flat lying foliation. A thin mantle of loess and colluvium is present, and the degree of weathering of the schist rocks is reflected in the variable side slopes of the gorge. Foliation shears are present in the abutments, although no wide shears have been identified immediately below the river channel. Steeply dipping orthogonal joint sets are present throughout the site, some with silt infilling following relaxation of the rock mass. Very high water flows were experienced in the abutment surface zones during investigation packer testing.

The 16.5 m high dam is the first to be constructed in New Zealand using Roller Compacted Concrete [RCC]. Aggregates were won from on site quarrying of schist with crushing and screening to manufacture stockpiles <5mm, 5-20mm and 20-40mm. Additional sand was imported from the Clutha River. Low heat cement has been used at the rate of around 150 kg/m^3. The dam RCC was placed over a three-week period. A moderate amount of foundation grouting has been undertaken using the grout intensity number [GIN] technique.

The tunnel is approximately 180m long, 2.5m x 2.0m section, fully concrete lined, and was excavated through schist rock. Steel sets were used for ground support in the more weathered rock near the portals, and the balance was rockbolted. The inlet structure incorporates a trash rack, and the outlet structure includes the pipeline transition and isolation valve chamber. The tunnel was constructed by Tunnel and Civil Ltd of Auckland.

T. Browne
OTAGO/SOUTHLAND BRANCH CO-ORDINATOR
(Contact details: Tel: (03) 474 8899; Fax:(03) 474 8995;
E-mail: tim.browne@opus.co.nz)

NZ Geomechanics News called for predictions of settlement for a water tank site which was preloaded. The site, located in Tauranga, is underlain by volcanic deposits.

The following data was supplied to registrants:

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

Predictions were required for the magnitude of maximum settlement at the end of the pre-load time frame and the measurement point at which this occurred. All predictions were required to be supported by brief calculations. Guesses were not accepted. Entries were free, but were restricted to individual Society members only. Entries were judged by a panel of three. Their decision is final. A prize of $100 was up for grabs.

Well, here they are, the results of the settlement prediction competition. Many thanks to all those who made submissions, particularly those of you with busy schedules who managed to squeeze in enough time to do the calculations.

The results as presented in the accompanying Table are I think you will admit very interesting. The predictions, apart from varying by more than an order of magnitude, also have a mean predicted maximum settlement, which is approximately 60% above the measured maximum settlement.

Statistical analysis shows the measured maximum settlement lies just outside the 95 percentile for the sample of predictions.

The mean of those predictions for the correct Point No.(3) was approximately 90% above the measured settlement.

There were of course the inevitable queries about what should be assumed ....

1) What is the weight/density of the preload material?
2) Where is the groundwater table?
3) Are there any c_v values for the material?
4) How long was the preload on for?
5) etc., etc. etc.

I can assure submitters that everyone received the same package of information and that no entrant received any preferential treatment The answers to these queries needed to be made based on construction experience and what materials were likely to be available for the preload. Data was presented on the density of the site soils and the saturation ratio (of close to 100%) for the tested samples could be worked out.

For interest and future reference, over 60 people were directly requested to make submissions for this competition to try and achieve a reasonable database. You will note there are 25 submissions including my own, (which of course was not eligible for the prize). It was later pointed out to me that no engineering geologists were requested to submit predictions. While not quite correct (registration was open to all readers of *Geomechanics News* & Society members, but there was of course an element of pure discrimination on my part and I am pleased to say, that the winner of the $100 prize was in fact an engineering geologist (Nick Rogers) despite the best efforts of a number of (mere) engineers. Congratulations Nick. The maximum settlement was 91 mm and was measured at Point No 3.

If any readers have any comments to make on these results I'm sure the next editor will be very pleased to hear from you especially if they are just short notes to the editor.

Once again, thanks very much to those who made submissions. There has been some discussion regarding another prediction exercise (possibly driven piles) some time in the future.

SETTLEMENT PREDICTION RESULTS

Entrant	*Prediction* (mm)	at Point No.	Calculation Method Used
Mark Davis	17	1	One Dimensional Terzaghi
Geoffrey Farquhar	47	5	
John Ko	35	5	
Phillip Onyang	676	3	
Bill Vautier	180	3	
Dr Laurie Wesley	40	3	
Bruce Horide	71	3	Modified One Dimensional Method
Dr John Hawley	35	5	
Grant Murray	65	3	
Bruce Symmans	154	3	
Robin Ni	138	3	
Gavin Alexander	214	3	
Alaa Ahmed-Zeki	166	3	Elastic Finite Element Method
Chris Bauld	122	3	
Sridhar Krishnan	101	3	
Lani Cheenikal	70	4	
Anonymous	40	3	Empirical Method
Tony Cowbourne	153	3	
James Burr	110	3	
Ian McPherson	216	5	
Grant Loney	330	3	
Nick Rogers	90	3	
Nigel Fitch	411	3	Modified Elastic Method
Aidan Nelson	110	3	
Stephen Crawford	127	3	

1999

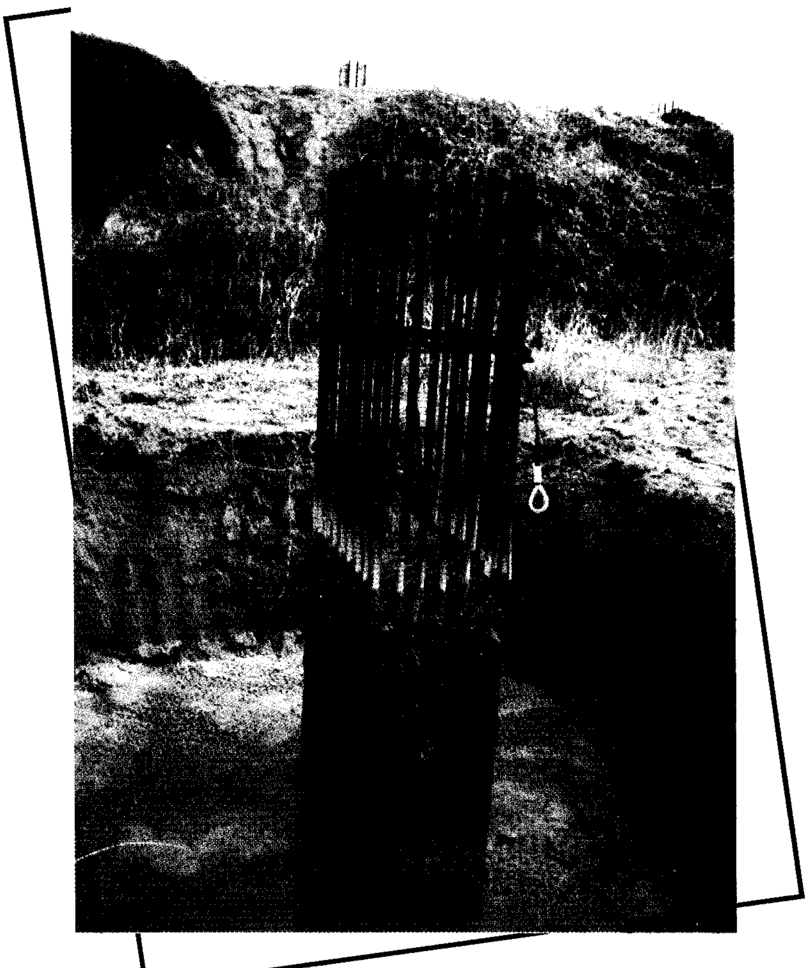
Photograph Competition

Winner

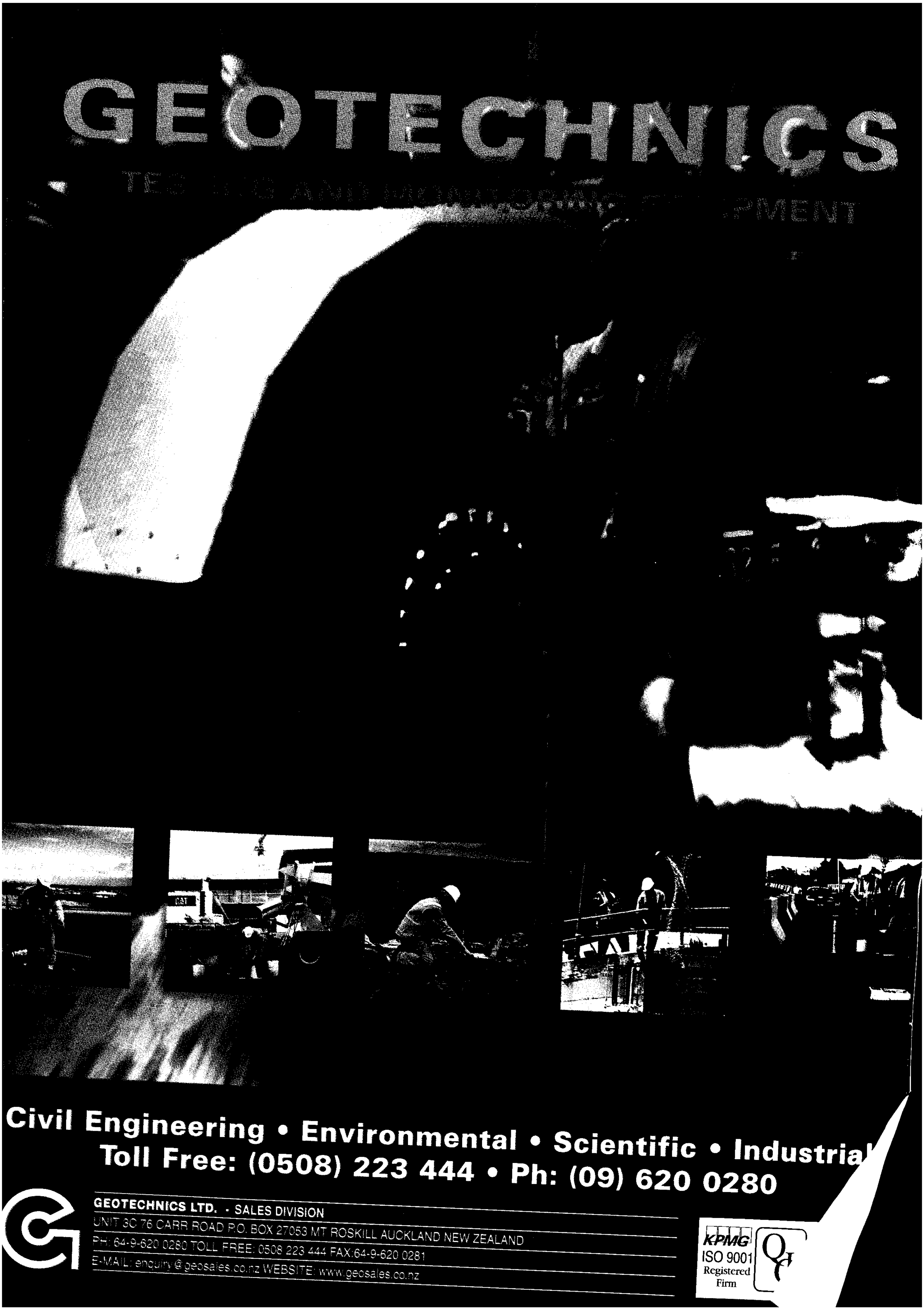
Congratulations to;

Graeme Jamieson

Bloxam, Burnett and Oliver



After the flood - Reinforced earth?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FIELD DESCRIPTION OF SOILS AND ROCKS IN ENGINEERING USE REVISION OF GUIDELINES

The Geotechnical Society feels that the time has come to review and revise the above document, and a small "committee" has been formed for this purpose. I (Laurie Wesley) am acting as convenor and currently the other members are James Burr and David Burns. If anyone else would like to be a member of the group, please contact me.

I am currently working on revising the document, and to do this I have endeavoured to establish some objectives or criteria to work to. These are the following:

- the guidelines should be as clear and uncomplicated as possible
- the guidelines should conform to international practice as closely as possible, (and any departures from this should be identified and explained).
- changes to the existing document should be made only when clearly justified i.e. only when they clearly improve the document.
- emphasis to be given to describing those properties which are of engineering significance.
- the guidelines should cover both field and laboratory description. (The current document is labelled "Guidelines for the Field Description of Soils and Rocks in Engineering Practice", but clearly includes material which refers to laboratory classification - for example the Unified Classification Chart). Altering the document to cover laboratory as well as field description is only a minor matter.

My current feeling is that the document should be changed in the following ways:

(1) There should be a clear distinction between **classification** and **description** on the basis below, and the procedures should reflect this distinction:

- ***classification*** refers to the identity of the material itself i.e. what its composition and intrinsic properties are.
- ***description*** refers to the in situ properties of the material i.e. what it is like in its undisturbed state

Classification systems used in soil mechanics, such as the Casagrande system (which was the basis of the Unified System), refer only to the material itself; they give no (or very limited) information on the state in which the material exists in the ground.
Descriptive systems, which are used in logging borehole cores, investigation pits etc., are systems which enable accurate accounts to be given of the state of the material in situ (ie in addition to the classification of the material itself).

(2) The section on soil weathering should be taken right out, as it does not seem to be a useful property.

(3) Colour should be given much less prominence then it currently is since it is only vaguely related to engineering properties.

(4) Properties and the terms used in describing them need to be explained in more detail.

(5) Departures from the Unified System need to be identified and explained.

If you have comments you would like to make about this review please send them to me (Laurie Wesley) at the following addresses:

Mail: Department of Civil and Resource Engineering, University of Auckland, Private Bag 92019, Auckland.
Fax: (09) 373 7462
e-mail: lwesley@auckland.ac.nz

ERRATA: NZ GEOTECHNICAL SOCIETY GUIDELINES FOR SOIL & ROCK DESCRIPTION

Unfortunately in the race to meet the magazine deadlines for the last issue, the third column of Table 2.10 was left in (in error). The errata are therefore reprinted here in full (also see note at bottom of page.

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 (kPa)
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)
Very dense	> 50	> 11
Dense	30 – 50	7 – 11
Medium dense	10 – 30	2 – 7
Loose	4 – 10	1 – 2
Very loose	0 – 4	0 - 1

A comprehensive review of these guidelines is due to be undertaken by a technical sub-committee convened by Dr Laurie Wesley (see previous article which includes contact details). Please address any comments or suggestions for this upgrading to Laurie.

QUANTITATIVE RISK ASSESSMENT SEMINAR - FEEDBACK

The Society recently sponsored a Quantitative Risk Assessment (QRA) Seminar held over 12-13th March at the University of Auckland. Prof. Robin Fell of the University of New South Wales and Garry Mostyn were flown in for the event. They worked extremely hard to give the participants a good overview of QRA in such a short time.

The seminar was restricted to seventy participants so that the tutorial sessions could be run effectively. There was no trouble filling these places and over 20 were turned away when the all the places were taken.

Feedback from the course was generally positive. As with all opinions there are always contradictions. Some participants did not like the tutorial sessions whilst others found them to be very useful. Similarly some people found the copious course notes difficult to use whilst others had no problem.

All participants thought that this was an appropriate activity for the Society to be involved with. Based on the feedback that the Management Committee has received, it is intended to try and hold one seminar with suitably renowned New Zealand or international presenters once a year.

Suggestions were sort from the participants for ideas for future activities. Many great ideas were presented. Some of the ideas were very regionally orientated and are likely to be dealt with at a branch level e.g. engineering geology of Auckland, volcanic soils. The suggestions fell into 5 broad categories, which are suited to seminars.

These are presented below: -

- Pile Design/ Limit State Design
- Soil & Rock Descriptions in NZ
- Seismic design & other issues
- Slope stability
- Dams

The Management Committee is exploring the possibilities of the obtaining well known New Zealanders or international speakers to present these topics.

It is appropriate that the topic of Soil and Rock Descriptions be presented after the Soil and Rock description review is completed by Laurie Wesley and his team.

Debbie Fellows
Management Secretary

CONGRATULATIONS – PERTH AWARDS

Our congratulations to the Young Geotechnical Professionals who won the Society awards to attend the next YGP ANZ Conference in Perth in 1000:

- Tony Fairclough
- Nicola Taylor
- Jamie Bevin
- Peter Bosselman
- Paul Horrey

The five awards comprise sponsorship to a maximum amount of $500.00 each from the Earthquake Commission and a further $500.00 from the society.

At the last joint NZGS/AGS meeting in Hobart in February 1999, it was agreed that each Society would select one presenter at the Perth YGP for further sponsorship to attend the 1st International YGP conference. This conference is to be held in Southampton in September 2000. The successful NZ based candidate for this further sponsorship will be assessed on the quality of the written paper submitted to the Perth Conference and their oral presentation.

We wish all candidates the best of luck and a rewarding experience.

8TH AUSTRALIA NEW ZEALAND CONFERENCE ON GEOMECHANICS, HOBART

These joint conferences between the Australian Geomechanics Society and the NZ Geotechnical Society are held every 4 years with NZ hosting every third conference. The conference was held in Hobart during February over a 3 day period with optional one or three day field trips after the conference. Twenty four of the 190 attendees were from NZ and 15 of the 128 papers were written by NZers.

The theme of the conference was "Consolidating Knowledge" and an excellent keynote address which touched on the theme was given by Prof Harry Poulos on "Common Procedures for Foundation Settlement Analysis - Are They Adequate?". The measure of the presentation, which included numerical models, was summed up by an engineering geologist who commented afterwards that he could easily follow the whole lecture. For those who deal with foundation settlements, Harry's paper is well worth reading.

The other major lectures included our own Tim Sinclair who made the first presentation of his 10th NZ Geomechanics Lecture and David Coffey who presented the Australian Geomechanics Society John Jaeger Memorial Lecture on "Commercial Geomechanics Development in Australia".

The remaining papers covered the full range of geotechnical endeavour. Approximately half of the papers were presented in sessions and the remainder in poster sessions. These conferences always have a good spirit and it is enjoyable to be meet the Australian geotechnical community. The Society's management committee also takes the opportunity at these conferences to hold a joint meeting with the Australian Geomechanics Committee.

Conference proceedings can be purchased through:

Australian Geomechanics Society	Phone	61-2-6270 6558
11 National Circuit	Fax	61-2-6273 2358
Barton	email	valerie_lee@ieaust.org.au
ACT 2600		
Australia		

Timber Framed Buildings - NZS 3604 Out Soon!

Despite the rigors of New Zealand's sometimes harsh maritime climate, timber-framed buildings have remained as the most popular form of domestic dwelling. It is estimated that 90% of all new houses built in New Zealand today are made out of timber. Over the past century, significant advances in building technology and design have occurred, which allow the building industry to come up with more innovative, well-designed and safer timber-designed houses. Standards New Zealand (SNZ) have documented that knowledge through the Standard NZS 3604 *Timber Framed Buildings*.

During its life span, the Standard has been updated three times to ensure it contains the most current and relevant information possible. A new version is to come out in May 1999 which incorporates significant changes in the industry since the last version was produced in 1990. Undoubtedly, the advent of the New Zealand Building Code (NZBC) in 1992, marks the biggest change in the building regulatory environment, over that time. Since its introduction the building industry has called for NZS3604 to be revised, so that it is aligned with the NZBC. A small amendment was made in 1992 but it was recognized that a more thorough revision of NZS 3604 was necessary. "Consequently, much of the content of the latest version of NZS 3604 has been driven by the requirements of the NZBC", says Neil Gerrish, the chairman of the committee revising the Standard.

The main aim of the NZS 3604 is to provide suitable methods and details for the design and construction of timber-framed buildings up to three stories high. The type of buildings it applies to are generally domestic dwellings, and some commercial and other buildings not requiring specific engineering design. Buildings *not* generally covered by NZS 3604 include medium to large office blocks, large public buildings such as museums and art galleries or buildings that are dedicated to the preservation of human life, such as hospitals.

'The Standard will be a little bit of a culture shock to the building industry, as it appears to carry a lot more information than before, which may be intimidating when initially using it", Gerrish says. But in fact, it is a 'dead easy' document to use, as it is written in easy-to-read English with a commentary and explanatory notes that are included in the margin, and run throughout. The document is intended for builders, architects and engineers and also for other house designers, including owners. The Standard is split into two parts, sections 1-16 which refer to the normative content, that is, information which directly refers to the requirements, and section 17 to 20, which contains content which is purely informative.

One reason for the document's increased thickness is that it now also contains most of the detail from the MP3600 Builders' Guide. Not only does this mean that all the relevant information is found in one place, but the new format, which puts all the information into an A4 ring binder, with clearly marked sections, makes for quick and easy reference, Gerrish says.

Gerrish is the Building Control Manager at the Porirua City Council and so is very familiar with the NZBC. "This Standard will make it easier for builders and designers to gain building consents. In a lot of areas the NZBC is asking for a little bit more, because a lot of research has gone on in the building industry gathering information which wasn't available 10-15 years ago. As a result the NZBC aims for buildings that are as safe and healthy as possible without going overboard."

One area which has attracted a lot of research is earthquake resistant buildings, particularly following the Kobe earthquake of 1995. This information has had a big impact on building requirements here, given the earthquake prone nature of New Zealand.

Another requirement under the NZBC which has resulted in an entire new section in the Standard, is the durability of materials. Claire Benge, an architect and also a technical adviser at the Building Industry Authority was on the sub-committee drafting the new durability section.

"In New Zealand the exposure to the sea laden winds is high - people have to face the fact that we live in a harsh climate. As our cities grow, buildings are on more exposed sites with more complex foundations. Added to that, in the last 10-15 years the use of timber piles with metal fixings has grown enormously. At the same time we now have a building code which specifically deals with the durability of materials, requiring structural fixings to have a durability of not less than 50 years."

"Initially it was thought that all subfloor metal fixings within 5km of the sea would need to be stainless steel. The use of protective systems such as powder coatings and paint applications on site has reduced this requirement to within the sea spray zone while demand has reduced the cost of those fittings that must be stainless steel. The cost of compliance will therefore not be nearly as great as was initially thought and it is no longer such an issue," Benge says.

In fact, an independent study has shown that the cost of building a new house, of varying types, in Wellington and Auckland would go up by less than 1%, Gerrish says. "Concern in the building industry that the cost of compliance is going to go up dramatically just isn't justified".

He also adds that NZS 3604 is not a mandatory document, it merely provides one means of complying with the NZBC. "It is a very valuable resource for those in the building industry, but it is not the only resource and if someone thinks its methods are too excessive, they are perfectly at liberty to use something else. At the end of the day it is the Territorial Authorities that have to be satisfied on reasonable grounds that the proposed work will satisfy the performance and functional requirements of the building code, not 3604."

Ian Gould, the representative for Master Builders' Federation, who was also on the committee, says he is absolutely confident the document will stand the test of time. He adds that it is important to realise that the Standard will never be a perfect document but it is a *good* document.

"This latest version of 3604 addresses certain failures in building industry methods and materials which have been well-documented. The building industry as a whole cannot turn a blind eye where these failures occur. The revised Standard contains a lot better information than it did in the past and I would be very disappointed if the industry doesn't accept it as a far superior document."

Gould is also at pains to point out that the document is a consensus document, which may mean that some areas won't be relevant to builders and others won't be relevant to engineers, but on the whole, it is a better document for everyone.

Ian Garrett, who was the Institute of Professional Engineers New Zealand (IPENZ) representative in developing the Standard, backs up this viewpoint. "A lot of effort has gone in to updating the Standard so that it makes more sense and has more relevant information for each group". For example, as far as engineers are concerned, access to the most up-to-date information about loading requirements for buildings is a top priority. An anomaly, which has now been corrected in the new Standard, is the requirement for snow loadings. Prior to the NZBC coming out, all buildings throughout New Zealand had to allow for snow loadings, even in areas where they would never experience it, such as Auckland. The revised Standard includes a table indicating snow loadings for different areas and conditions. It also gives explicit information for laying foundations, which under the NZBC must not show any signs of destabilising for 50 years.

"New Zealanders tend to be lateral thinkers and this Standard provides people in the building industry with more opportunities to use innovative methods in design and construction", Garrett says.

Those involved in drafting the Standard say that for it to be used to its full potential, with the minimum of problems for the building industry, education on how to use it correctly needs to take place as quickly as possible. To that end, the Building Industry Association, Building Research Association of New Zealand, Registered Master Builders' Federation and SNZ are running a series of seminars on how to use NZS 3604. Details on dates and venues are given below. For further information on how to register contact Barbara Kerr at BRANZ on ph:04 235 7600 or fax:04 235 6070.

As one of its most important standards, SNZ has made NZS 3604 the first of its standards to be available in interactive CD-Rom, with all 400 pages presented in Portable Digital Format (PDF) and viewed through a web browser (e.g. Netscape or Internet Explorer – included on the disk). The Standard contains numerous drawings and diagrams on CAD files which purchasers of the Standard will be able to download and modify. The CD-Rom was created for SNZ by Terabyte Interactive and is a model of how SNZ will publish key standards in digital format in the future.

Pre-orders of NZS3604 Timber Framed Buildings can be made by contacting SNZ Sales at ph:04 498 5991, fax:04 498 5994 or off the SNZ website www.standards.co.nz at the following prices:

Hard Copy of the Standard

SNZ members	$185.60 plus GST
Retail	$232.00 plus GST

Standard/CD Rom bundle

SNZ members	$316.00 plus GST
Retail	$395.00 plus GST

CD Rom only

SNZ members	$236.00 plus GST
Retail	$295.00 plus GST

DATES & VENUES FOR NZS 3604 SEMINAR

June

15	Gisborne	Cosmopolitan Club, cnr Grey & Derby Streets
16	Napier	War Memorial Centre, Marine Parade
17	Masterton	Cosmopolitan Club, cnr Chapel & Queen Streets
22	Whangarei	Quality Hotel, Riverside Drive
23	Auckland am	Society of Accountants, Ohinerau Street
23	Auckland pm	Society of Accountants, Ohinerau Street
24	North Shore	Bruce Mason Centre, Takapuna

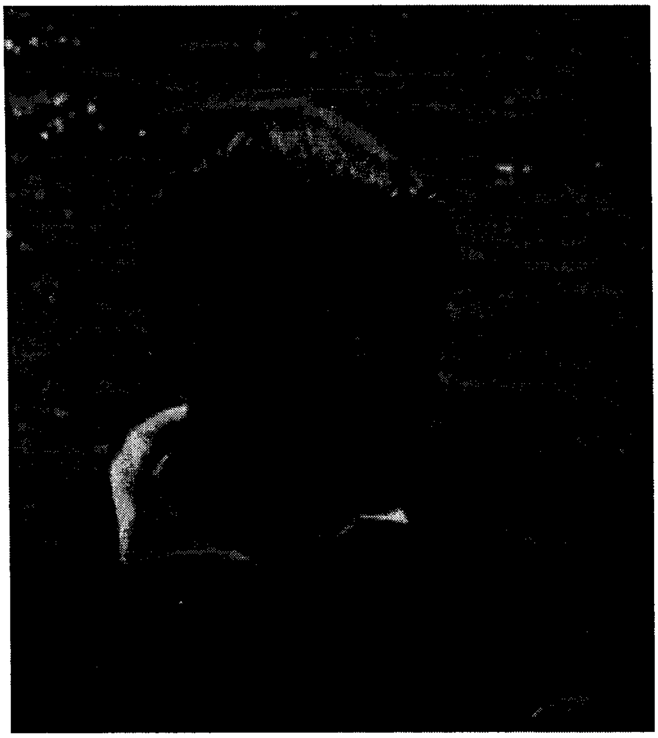
July

5	Taupo	Great Lakes Centre, Tongariro Street
6	Hamilton	Glenview Hotel, Ohaupo Road
7	Rotorua	Quality Hotel, Fenton Street
8	Tauranga	Hotel Armitage, cnr Willow & Park Streets
13	Palm. North	Centennial Centre, Main Street
14	Wanganui	Avenue Motel, Victoria Street
15	N. Plymouth	Rydges Hotel, Bell Block
20	Invercargill	City Council Chambers, Tay Street
21	Alexandra	Centennial Court Motor Inn, Centennial Avenue
22	Dunedin	Carisbrook Lounge, Surrey Street
27	Hokitika	Southland Hotel, Revell Street
28	Christchurch	Hotel Grand Chancelor, Cashel Street
29	Timaru	District Council Chambers, King George Place

August

3	Nelson	Copthorne Rutherford Hotel Trafalgar Square
4	Wellington	Air New Zealand Suite, Wellington Convention Centre
5	Kapiti	Kapiti Community Centre, Ngahina Street
6	Wellington	BRANZ Conference Room, Judgeford
10	Auckland	BUILDEX '99, Ellerslie Racecourse
11	Auckland	HERA, Gladding Place, Manukau
12	Blenheim	Blenheim Country Lodge, cnr Alfred & Henry Streets

BRIAN PATERSON 1937 - 1999



We were saddened to learn that Brian passed away after a struggle with cancer. Our condolences go to his wife and family who have been caring for him over recent months.

We remember Brian for his twinkle–in-the-eye sense of humour, his rolled rr's that he retained even after working for several years in Western Australia and which reflected his southern roots, and his sense of fair play and consideration for his fellows. Brian spent most of his career working for some 40 years in applied geology, mainly as an engineering geologist with NZ Geological Survey, initially on the Tongariro Power Development based at Turangi, then at the Christchurch where he eventually became District Geologist. It was during the DSIR restructuring process in the early 1990's when IGNS made moves to close the Christchurch office that Brian decided to stay put rather than move, and with Glen Coates formed his own engineering geology consulting company.

Shortly after graduating in geology from Otago University, Brian went to Western Australia as a field geologist for several years where he worked, amongst other projects, on the Ord River scheme , and ground deformation associated with fault displacement of the 1969 Mekering earthquake. He returned to NZ to work in the Engineering Geology Section of NZ Geological Survey, at Turangi on the Tongariro Power Scheme. Here he helped establish engineering geology as a practising science and art form in NZ. In those days of the early 1970's most engineering geos in NZ were employed by NZ Geological Survey, working mainly on national development projects such as large hydro schemes. Brian, fondly known as "Chops" for the prominent sideburns he sported in the early '70s, played a leading role in establishing methods that are still used today for engineering geological logging of drill cores, tunnels, excavations, and for reporting on site investigations. Brian also played an active role in the organisation and management of the NZ Geomechanics Society, and the Geological Society of NZ, where he was always ready and willing to lend a hand.

Technically, Brian was involved with describing and monitoring (for early warning systems) the volcanic hazards associated with Mt Ruapehu while in Turangi. He extensively studied and reported on the tunnelling difficulties experienced in the alpine conditions encountered in Moawhango to Tongariro tunnel on the Tongariro scheme. (Brian also enjoyed a reputation unequalled by others for enthusiastic participation with the Italian tunnellers at kilometre and hole-through celebrations). He took his skills to Manitoto in the trouble-some schist terrain and also studied numerous landslides on both road and rail corridors. In the last few years Brian has worked on the investigations and construction of the Otira Viaduct, work he delighted in showing us during a landslides workshop held at Cass last year. Thank-you Brian for your contributions and fond memories.

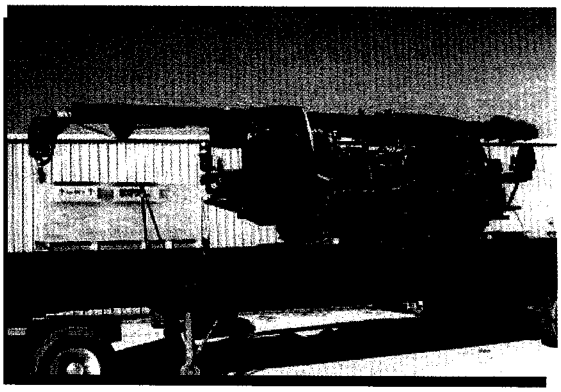
From your colleagues and friends:

Dick Beetham, Stuart Read, Nick Perrin, Graham Hancox, Bernard Hegan, Les Oborn, Simon Nathan and others too many to mention individually.

GEOTECHNICAL & ENVIRONMENTAL DRILLING

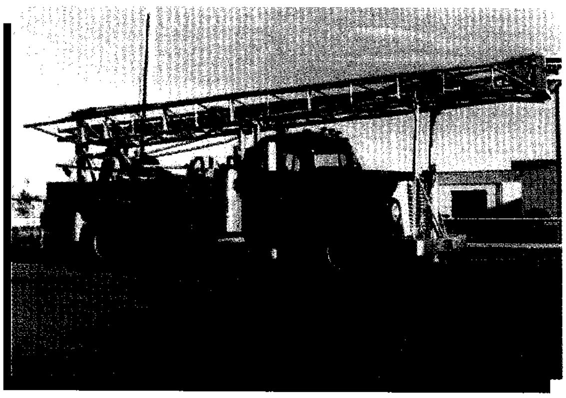
GEMCO HP7
Hydraulic rig, 1.5 metre drilling stroke, angle capabilities, coring pump. Suited to confined sites and shallow investigation.
Base rig rate $125/hr

ATLAS COPCO
Small track rig, fully hydraulic, 3 metre drilling stroke, coring pump, will drill any angle. Ideal for horizontal drainage holes.
Base rig rate $145/hr.



SIMCO 5000 TR3
Compact, powerful and flexible. Suited to difficult access sites. Angle capacity, fully hydraulic, multi speed top head and high volume triplex pump.
Base rig rate $155/hr

SIMCO 5000
Fully hydraulic, multi speed tophead, 6 metre drilling stroke, angle capacity, high volume/pressure triplex pump & 5 x 6 mud pump. Photo shows rig in Hollow Stem Auger mode.
Base rig rate $155/hr



GEFCO 1250
Kelly drive drilling rig, complete with 5 x 6 mud pump. Very fast sedimentary and SPT rig.
Base rig rate $155/hr

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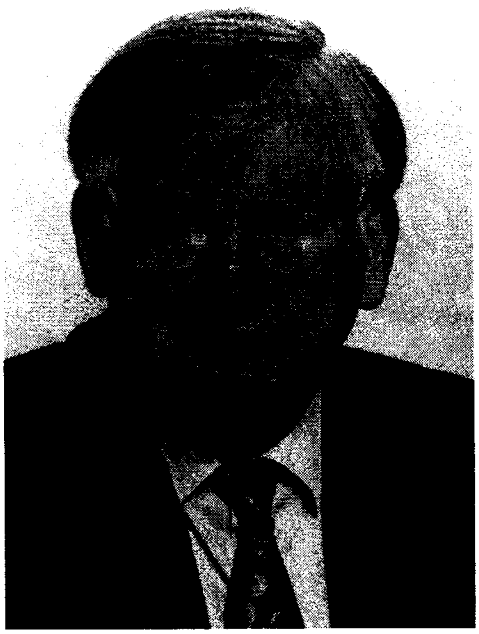
* Base rig rates exlude GST.

PETER JOHN NORTH



Peter North died on 23 April, of cancer. He was 59. Peter was a widely experienced civil engineer, teacher, friend and family man.

Peter had a passion for civil engineering and was a wonderful ambassador for his profession. Bridges were his special strength and he has left his own legacy to future generations because of this. If you cross the Waikato River at anytime, the chances are it will be by way of a Peter North designed bridge. Some of his other projects included the Tauranga Harbour Bridge, the Pukaki Bridge near Auckland Airport and a multitude of rail bridges on the North Island Main Trunk Line.

Peter always relished a challenge and the more difficult it was the more he seemed to enjoy it. He played a major role in the development of America's Cup bases in Auckland's Viaduct Basin. He managed the design of a major sewer tunnel, the Whaeo hydro-electric scheme and numerous marine structures.

He also enjoyed travel and work in Australia, Singapore, Hong Kong, Fiji, Malaysia, Indonesia, Thailand, the Philippines, China, Armenia, Bougainville, Vanuatu and the Solomon Islands helped him to further this enjoyment. He was recognised internationally for his work in the seismic design of bridges and was a Past President of the New Zealand National Society for Earthquake Engineering.

Peter achieved much more in the profession he loved than can be described here. One example, though, is the way he developed a special relationship with contractors who came to know him as someone who listened, understood their points of view and if anything went wrong, would stick with it until it was fixed. To quote one of his friends, "He was the leader in his field and you were always rewarded if Peter was part of your team".

For those of us who had the privilege of working closely with Peter there were many rewards too. Laughter was something you heard often when Peter was around – it was one of the many things that made him special. As one of his colleagues said recently, he was always able to see the funny side of things. One of our enduring memories of Peter will be his totally uninhibited enjoyment of things that amused him. An otherwise quiet office would suddenly erupt with the sound of his laughter as he shared a joke with someone, often several doors down the corridor.

Other things that made Peter special were his ready smile and the way he genuinely cared about people. Perhaps most special of all was the gift he gave so freely to everyone – that of his time. No matter how busy Peter was himself, he was always willing to take time out to listen to your problems. He had an amazing knack of being able to make you feel good about yourself by helping you to think you had found solutions to your own problems, whereas in reality we were learning from his life experiences and drawing on the wisdom that he so evidently possessed.

Peter was an engineer of rare skill. He was also a person with a unique set of personal qualities and a genuine concern for those around him. He had real Mana.

Our deepest sympathy goes to Peter's wife, Trish, his mum Dulcea and children Robbie and Janelle and their families.

Jim Hodges

New Zealand Geotechnical Society
GEOMECHANICS AWARD

For Best Published Paper
Awarded once every three years
Award Value NZ $500

Purpose of the award

The Geomechanics Award is bestowed on the author(s) of papers that are distinguished in their contribution to the development of geotechnics in NZ and advances the objects of the Society.

The Objectives of the Society are:-

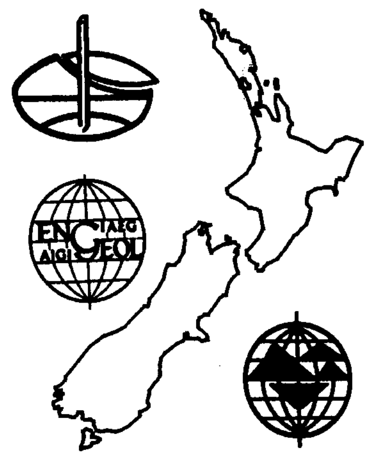
- To advance the study and application of soil mechanics, rock mechanics and engineering geology among engineers and scientists
- To advance the practice and application of these disciplines in engineering
- To implement the statutes of the respective international societies in so far as they are applicable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

Criteria :-

- The author or co-author are financial members of the Society.
- Published a paper on in a topic in the field of Geomechanics in the three-year period ending 31 July 1999.

How to apply yourself or nominate another member:-

Simply send a copy of your paper to the Societies Management Secretary.
The covering letter should include a clear reference to journal, book, conference proceedings, or other publication in which this paper was presented.
Both International and National publications are acceptable.



Applications to:-

Management Secretary
NZ Geotechnical Society
P O Box 12 241
Wellington

By:- 15 August 1999

The New Zealand Geotechnical Society
Student Prize

(NORTHERN AND SOUTHERN)
Prize - Books to the Value of $250

Two Student prizes are available:-

- **One for the North Island Students – presentation of paper to be in either Auckland or Hamilton.**
- **One for the South Island Students – presentation of paper to be in Christchurch.**

Are You? :-

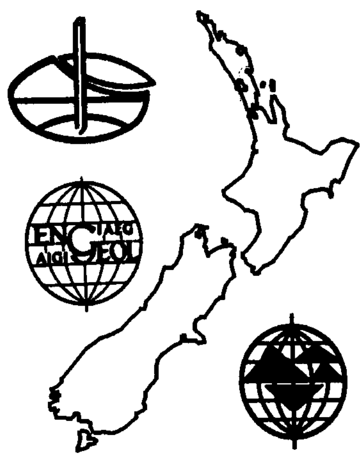
- A Tertiary Student in New Zealand
- Able to submit an abstract (400 words or one typed A4 page) on your topic in the field of Geomechanics?
- Be prepared to make a 20-minute presentation (with 5 minutes for questions) at NZ Geotechnical Society group meeting in Auckland or Hamilton, or Christchurch (depending on your location)?

IF SO, you are eligible for one of the two NZ Geotechnical Society Student Prizes.

Judging criteria.

The judging of the awards will be based on the following criteria:-

- Structure
- Clarity of explanation
- Time / Length
- Visual Presentation Clarity
- Content
- Written abstract
- Question handling



TO APPLY,
simply send your synopsis to :-

Management Secretary
NZ Geotechnical Society
P O Box 12 241
Wellington

By:- 31 August 1999

THE NZ GEOTECHNICAL SOCIETY WEB SITE

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

Note: The Australian Geomechanics Society (AGS) website can be viewed at the following address:

http://www.ieaust.org.au/societies/ags

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

SYNTEX 1001 IN THE RACE AT TE RAPA DAIRY

Last year work started on the new Powder Plant, an expansion of the Te Rapa Dairy Factory. The project centred around the construction of a new drier building and drier plant. Additional contracts resulting from the expansion included the construction of a new underpass off State Highway 1, wastewater treatment works, roading, drainage, and various electrical and mechanical installations.

Permathene Ltd supplied Syntex Geotextile for use in 2 contracts associated with the expansion work: the Main Civil Contract, awarded to Pemberton Construction Ltd of Hamilton which included construction of the stormwater diversion pond and the Waikato River Outfall Contract, awarded to McConnell Dowell Ltd. of Tauranga with Pemberton Construction as subcontractor on the gully. John Crawford and Alan Muller of Opus International Consultants Ltd, Hamilton oversaw these contracts.

The Stormwater Diversion Pond, 57 m long and 41 m across with a depth of 4.5 m was lined with Syntex 1001 non-woven Geotextile before putting a Flexible Membrane Liner (FML) on top of it. FML are generally prone to damage from even isolated and infrequent protrusions in the subgrade onto which they are deployed. Syntex 1001 provides security to FML against damage during installation and throughout the life of the facility. Syntex 1001 also helps to increase the puncture resistance of the FML and if properly stabilised and buried, Syntex non-woven geotextiles are expected to last up to 200 years.

New powder plant at Anchor Products site with Syntex 1001 Geotextile.

The laying of the 6m wide stormwater channel and 2m wide treated wastewater gully necessitated construction over a very soft swamp. Site investigations indicated the proposed subgrade was comprised of black topsoil with large rotting tree stumps and logs, underlain by extensive peat deposits. David Ward, Contracts Manager of Pemberton Construction Ltd, described the site as marshy and difficult to walk on. A 1m deep V Drain was constructed and filled with metal. Syntex 1001 non-woven geotextile was laid on top of the metal, covered with 300-400 mm of rip rap (100-300 mm size aggregates) and levelled. Syntex 1001 acts as a "separation" layer stopping the rip rap material from pushing down into the soft wet subgrade and preventing mud from contaminating the clean aggregate. This gives a more stable, wearing and sturdy solution for the drainage problems.

Syntex 1001 polypropylene, staple fibre, needle-punched, non-woven geotextiles are produced with proven technology in state of the art ISO 9002 certified production facilities by Synthetic Industries Inc. USA. Syntex geotextiles are distributed in New Zealand exclusively by Permathene Ltd. – Civil Engineering Division.

Once completed in October 1999, Te Rapa Powder Plant (Anchor Products) will be the largest milk powder plant in the world.

Moninder Bindra.
Permathene Ltd – Civil Engineering Division.

Syntex 1001 non-woven Geotextile at the Anchor Products site.

YAMUNA RIVER BRIDGE, NEW DELHI INDIA

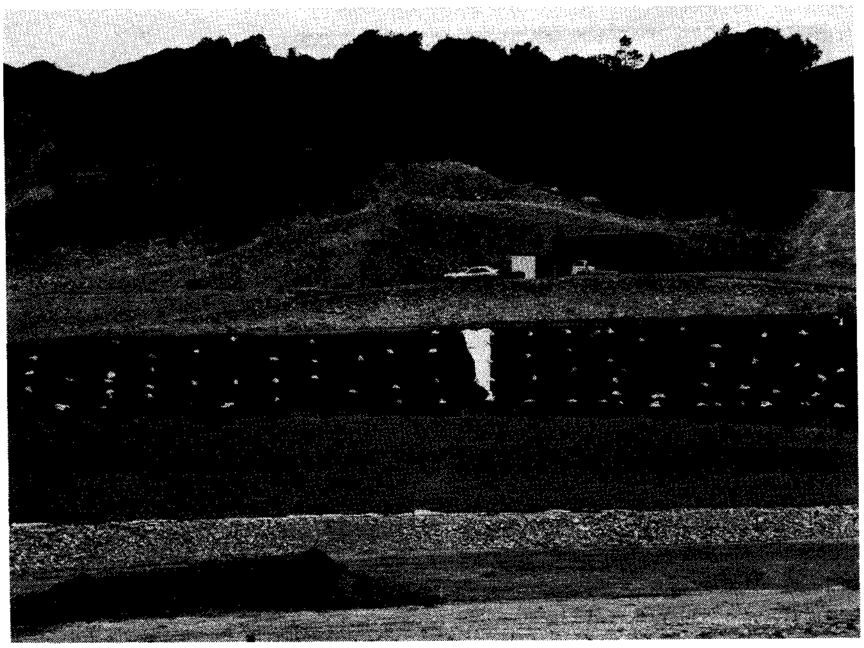
This major Indian project is centred on the construction of an 8-lane bridge across the Yamuna River linking Delhi & Noida in New Delhi, India. Noida is a prosperous industrial town on the periphery of East Delhi currently experiencing rapid population growth. The Yamuna river flows between Noida and New Delhi and all commuter traffic from Noida must use an existing 4-lane bridge in a neighbouring town which links East Delhi with New Delhi. In peak hours, it takes more than an hour to travel from Noida to New Delhi or vice versa.

The new 8-lane bridge will reduce traffic congestion during peak hours between Noida and New Delhi. The whole project has been awarded by the Noida Toll Bridge Co, New Delhi to M/s Mitsui Marubeni Corporation, a Japanese Company. They have sub-contracted the work of approaches to the bridge to Oriental Structural Engineers Ltd, New Delhi and 8 lane main bridge work to M/s Gammon (India) Ltd, New Delhi.

Oriental Structural Engineers Ltd is responsible for the construction of huge sand embankments by dredging sand from the Yamuna riverbed. The contract includes hydraulic filling, construction of slab culverts, pipe culverts, retaining wall construction and protection work along the roadway, or guide bund. The scope of work also includes the construction of road crust protection works consisting of stone pitching / gabions filled with stone boulders and placed over Syntex 801 non-woven Geotextile on slopes and apron. The embankments will also be protected with Syntex 801 covering and stone pitching / gabions. Syntex 801 non-woven geotextile was supplied by Permathene Ltd of New Zealand. This is a polypropylene, staple fibre, needle-punched, non-woven geotextile, produced to ISO 9002 by Synthetic Industries Inc. USA.

The total cost of work awarded to Oriental Structural Engineers Ltd is NZ$34.8 million. The duration of the job is 24 months. This is one of the largest bridge projects in India and Permathene is pleased to supply the total geotextile (over 250,000m^2) required for this project.

Moninder Bindra, Permathene Ltd (Civil Engineering Division)



Construction of bridge across Yamuna River New Delhi, India

THE GREEN SOLUTION – SOIL PANEL WALLS

An innovative new system for retaining road and motorway cuttings has been introduced to NZ on the Albany Puhoi Realignment (ALPURT) motorway project north of Auckland. The patented Soil Panel System, a completely vegetated 100% green solution, was developed by Phi Group in the UK in response to the need for an environmentally sound and cost-effective solution to retention of cut slopes on motorway widening works on the M1, M4 and M5. Stability and structural support of slopes as steep as 70 degrees is provided by soil nailing the newly excavated face and bolting heavily galvanised steel Soil Panel cages to the protruding nails across the exposed face. The internal surface of the 225mm deep cage forms the structural support to the embankment face and the outer, geotextile lined face allows filling with topsoil growing medium for establishment of permanent vegetation.

By reinforcing the existing insitu soils, the need for over-excavation, muck away and importation of select backfill materials generally associated with other retaining structures is avoided. Because the soil nailing proceeds with the excavation, the face is progressively stabilised from the top down thus avoiding any need for temporary earthworks retention. Further economies are provided by the rapid speed of construction.

The prime ecological advantage of the system is the separation of the growing medium from the inert structural requirements of the reinforced soil block. Soil Panel allows the structural specification of the reinforced soil block to be met while creating a thick covering of quality topsoil over the entire face. This provides the horticultural environment necessary for the assured establishment and sustainable growth of vegetation. "No-maintenance" vegetative cover is afforded by the careful selection of low growing drought resistant grasses and plants. The face also provides protection of the structural elements in the event of collision or fire damage.

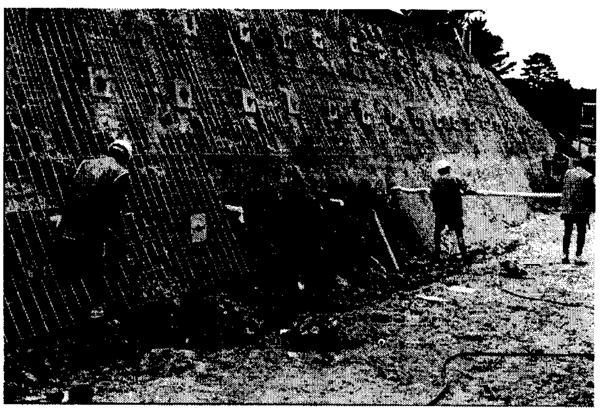
In addition to the soil nailed (SN) version of Soil Panel there is also a reinforced soil (RS) version for use in fill embankments. Utilising geogrid or steel ladder reinforcing elements it affords many of the same environmental, technical and cost advantages of the soil nailed version.

The Soil Panel system was selected and specified by Opus International Consultants for two locations on the initial A1 sector of the ALPURT project where large cut heights are required close to neighbours' boundaries. With environmental aspects of the project of prime consideration, Transit NZ approved the Soil Panel system as providing an ideal environmental, technical, aesthetic and economic solution. Phi Group NZ Ltd was nominated as the design, supply and construct sub-contractor under main contractor Stevenson Construction. The initial 1100m Soil Panel structure at Greville Rd interchange, with maximum heights of 7.5m, was commenced in mid-March and completed in late May 1998. The second wall north of Lonely Track Rd has a maximum height of 10.1m and a face area of 700m . Construction commenced in February and the structure is nearing completion (in early May).

Phi Group's UK technical director, Ian Price, was brought out by their NZ office for the duration of the Greville Rd design and construction programme to ensure optimum transfer of technology for the patented proprietary system. He commented that "Soil Panel has rapidly gained acceptance in the UK, and work from Highways Agency and local authorities now forms a substantial portion of our UK business. It is very well suited to NZ conditions and we see a very bright future for the system here." Local Phi Group Engineering Manager, Peter Marchant, and Technical Support Manager, Brian Davis, have successfully undertaken design and construction of the second wall.

With several other Transit and local authority applications currently under assessment, the Soil Panel system in both it's soil nailed and reinforced soil versions is set to establish a new local benchmark for environmentally sensitive engineering solutions for ground retention and stabilisation.

Peter Syred
PHIL GROUP (NZ) LTD



The second Soil Panel wall north of Lonely Track Rd is currently under construction

EARTHQUAKE INDUCED LANDSLIDING IN NZ AND IMPLICATIONS FOR MM INTENSITY AND SEISMIC HAZARD ASSESSMENT

G T Hancox, N D Perin & G D Dellow
Institute of Geological & Nuclear Sciences (IGNS)

The report researched and produced by IGNS is very clearly set out with an excellent presentation. The tables, figures and maps are clear and well presented.

The data base presented in report consists of description of 22 earthquake events spread between 1848 to 1995, twelve in the North Island, ten in the South Island. As might be expected most of the reported earthquakes have occurred along the east coast of the North Island and along the Alpine Fault in the South Island. Consequently the epicentres are in rugged if not mountainous terrain. One exceptional earthquake reported on was the Peria event of 23 December 1963 in the Far North. This latter earthquake is of interest as it occurred in an area where landslides from other causes are widespread.

One of the goals of the report was to revise the NZ MM Intensity Scale and a proposed environmental criterion is presented in the report. The criteria is quite detailed but presented in a logical manner in terms of landslides, subsidence, sand boils and lateral spreads. A checklist in tabular form would easily be produced from this list to be used in the field.

There are recommendations set out in the report for further research on paleoseismicity and detailed studies of the Murchison, Inangahua and Wairarapa earthquakes. This would further understanding of the NW Nelson and Wellington areas and future seismo-tectonic hazards. This may be worthy of future study but appears to widen the scope well beyond landslides. This approach should be balanced against improving data recording for future earthquake events. This research project was funded by the EQC Research Foundation.

<u>Bernard Hegan</u>

Figure 17.2: View of the "*Zig-Zag*" rock avalanche area on the Arthur's Pass highway (SH73), with the debris scree slope and very steep headscrap bluffs rising about 500m above the road. During the 1994 Arthur's Pass earthquake, the highway was damaged by a number of small-moderate rock falls (F) on the Otira side of Arthur's Pass (AP, one blocking the road and damming the Otira River for several days (LD). Photo by: G.T. Hancox 3/39:24/2/96

QUANTITATIVE RISK ANALYSIS A NEW TOOL FOR DECISIONS ON STEEP SITES

BRANZ has recently published a study report which discusses the new techniques available for quantifying the risks inherent in a building site. The report is based on research undertaken by Riddolls and Grocott Ltd, a Christchurch-based geotechnical consultancy.

Before issuing building or subdivision consents for sloping land, local authorities need to know how stable that land is. Geotechnical engineers have, traditionally, relied on experienced-based judgement to assess whether a slope is stable enough to allow development. This judgement was complemented by numerical techniques.

Now however, new techniques are available. Both numerical methods and subjective judgement are used to **quantify** the risks inherent in any system, for example a slope. These techniques are collectively referred to as Quantitative Risk Assessment (QRA).

QRA consists of assessing the probability of slope failure and identifying the consequences of failure. It is used to assess the risk of failure of engineered slopes, for example, cuts, fills and retaining walls and of land that has already been affected by slope instability. It is more difficult to assess the risk of first time sliding of natural slopes.

SUPPORTED BY THE BUILDING RESEARCH LEVY

The actual process of QRA includes three steps: risk analysis, risk assessment and risk management.

The main benefit of QRA is that all cause and effect relationships associated with slope instability can be taken into account. This is not the case with conventional analyses, which emphasise the probability of failure rather than the consequences.

The application of QRA is limited by the difficulty of accurately determining the input parameters. However, the numerical expression of risk allows the best estimate of the stability of a slope to be communicated in terms easily understood by lay personnel. This means landowners, developers and regulators are able to understand the risks on which to base their development and planning decisions.

This report discusses QRA in detail and compares it to the traditional methods of slope stability analysis. In addition the methods of determining the numerical values for risk are discussed. The final part of the report summarises the applicability of QRA for New Zealand conditions and highlights a number of areas where there is considered to be scope for additional work.

How do I buy a BRANZ study report?

BRANZ Study Report 83 (1999) "Quantitative Risk Assessment Methods for Determining Slope Stability Risk in the Building Industry" by Riddolls and Grocott Ltd ($18.00).

To buy a BRANZ study report, call freephone 0800 80 80 85 (press 3 for publication sales) and give your credit card details. Or you can mail a written order and cheque to BRANZ Publications, Private Bag 50-908, Porirua City.

WEB SITES OF INTEREST

The list below is a quick compilation of web site data which may be of interest to readers. It is intended that this list be replaced with new listing in every issue of *Geomechanics News.* Readers are urged to contribute brief information on web sites. Please forward any data to the Editor (scrawford@tonkin.co.NZ).

NZ GEOTECHNICAL SOCIETY WEB SITE LINKS

	Organisation/Author	Notes	Address
Geotechnical Societies	**AGS**	Australian Geomechanics Society	www.ieaust.org.au/societies/ags
	BGS	British Geotechnical Society	
	ISSMFE	Int. Society Soil Mechanics and Geotechnical Engineering	www2.eng.cam.ac.uk/ issmfe
	IAEG	Int. Society for Engineering Geology & the Environment	www.transport.ntua.gr/iaeg.html
	ISRM	Int. Society Rock Mechanics	
Journals, Magazines	**EJGE**	Electronic Journal of Geotechnical Engineering	geotech.civen.okstateedu/ejge/
	iGEM	Internet Geotechnical Engineering Magazine - Articles, Reviews, Website Links, Conferences, Jobs, References, 'Hall of Fame', Software, ...	geotech.civen.okstate.edu/magazine/
Research/ Libraries	**Swedish Geotechnical Database**	50,000 references - Charges apply	www.sgl.geotek.se
	CIRIA	New site (as at July 1998)	www.ciria.org.uk
	Auckland Library	University Library : Allows searching within the active Auckland University Library.	voyager.auckland.ac.NZ
Software Suppliers	**Geotechnical & Geo-environmental software Directory (Tim Spink)**	Probably the most comprehensive directory of geotechnical software on the Web – well worth a look!	www.rockeng.utoronto.ca/
	Rocscience-Geomechanics software and research (Univ. of Toronto)	Practical Rock Engineering by Dr Evert Hoek as well as software like: Phase 2, RocFall2, Swedge3, Slide 2 RocData 2.3, Dips 4, ...	www.rockeng.utoronto.ca
	Tagasoft	Includes geotechnical software, some of which can be run over the web without actual purchase of license	www.tagasoft.com index.html
	Geosolve (Dr D.L. Borin)	Includes WALLAP, GWALL and SLOPE	www.geosolve.co.uk
	Zace Services	Switzerland-based. Many of us received a mail-out recently. Z_SOIL.PCV4 software. Includes free demo version.	www.zane.com
Mailing Lists	**Engineering-geotech**	"All aspects of geotechnical engineering" (soil and rock) To subscribe: Send a plain text e-mail containing the message: join engineering – *geotch [first name] [last name] [org name]* to the following address: mailbase@mailbase.ac.uk To unsubscribe: Send a plain text e-mail containing the message: *leave engineering-geoteh stop* to the following address: listproc@usc.edu	
	Geotech	Geotech earthquake engineering and engineering seismology (NB: closed – requests to join are first vetted before acceptance) To subscribe: *subscribe geotech [first] [last]* To unsubscribe: *signoff geotech*	
Other	**Paul Bourke**	Take a fascinating photographic excursion through the Albert Park tunnels beneath Auckland City.	www.mhri.edu.au/~pdb/albert/
Newsgroups	**sci.engr.civil Ground Engineering**	Contains the articles from the "Where to go on the World Wide Web" series in recent Ground Engineering magazines. Contains lots of use-ful info' on sites, newsgroups, etiquette,etc	*dialspace,dial.pipex.com/town/terrace/qs21/www*

Tony Cowbourne, Stephen Crawford

SUMMARY OF TECHNICAL PAPERS:

- **THE 10TH NZ GEOMECHANICS LECTURE**
  Geotechnical Analyses: Fundamentals to Fractals
  T J E Sinclair

- Common Procedures for Foundation Settlement Analysis – Are they Adequate?
  (Keynote Lecture, 8th ANZ Conference, Hobart, Australia)
  H G Poulos

- Applicability of the Hoek-Brown Failure Criterion to New Zealand Greywacke Rocks
  S. Read & L. Richards

- Relation Between Settlement and Cone Penetration Resistance at Two Sites
  M J Pender

- PDA – Dynamic Testing of Driven and Cast In-Situ Piles
  M Fraser

- Back Analysis of Strength Parameters for Landslide Control Works
  T Yamagami & Y Ueta

- Development of Laboratory Loadman Test for Determination of the Elastic Modulus of Road Pavement Materials
  R Peploe

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 presented the 1999 Geomechanics Lecture firstly at the 8th ANZ Geomechanics Conference in Hobart earlier in February this year. Tim also toured Auckland and Wellington in late April and Dunedin and Christchurch in early May this year.

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 10th NZ Geomechanics Lecture is published in the following pages.

What is a fractal?

B. Mandelbrot - A rough or fragmented geometric shape that can be subdivided in parts, each of which is (at least approximately) a reduced-size copy of the whole.
Mathematical - A set of points whose fractal dimension exceeds its topological dimension.
Topology - Study of geometrical properties unaffected by changes of shape or size.

Geotechnical Analyses: Fundamentals to Fractals

T.J.E. Sinclair
M.A., M.Sc., D.I.C., C. Eng, MICE, MIPENZ
Director, Tonkin & Taylor Ltd, Auckland N.Z.

Summary The purpose of this 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.

1. INTRODUCTION

This paper offers a single view of the art and science of geotechnical engineering, drawing on 30 years of experience in consulting practice with projects ranging from the very smallest to some of the very largest. There is no attempt here to claim a leading edge in the field, just the presentation of a simple discourse on the slow accumulation of knowledge or enlightenment on the backs of others; clients, colleagues, academics and particularly, the ubiquitous gremlin.

With the power of computers ever increasing, one would expect that numerical methods should now be of great importance and hold a dominant position in our portfolio of capabilities. But that is not generally the case. One reason for this is that most of us recognise their limitations!

Conversely, there is a danger that the fundamentals of soil mechanics are forgotten even in the most familiar of circumstances. For example, without a good understanding of the concepts of effective stress, undrained strength and theory of friction, simple slope stability analyses can be meaningless.

The first part of this paper looks back on the growth years of numerical models with the main intention being to illustrate:

- The ability to model complex interactions, mechanisms and geometrics.
- The limitations due to difficulties of measuring material properties and initial conditions.
- The need to allow for chaotic changes and features.
- The range of different situations which can be addressed.

The second part of the paper suggests a parallel emphasis on the fundamentals of soil mechanics, with a recent case history to provide an example of how consideration of basic concepts can help understand complex problems.

2. NUMERICAL MODELS

2.1 A Code Conceived

I take this opportunity to claim my part in the conception and upbringing of one of our commonest numerical models in use today. FLAC, marketed by Iatasca Group, was developed under the direction, drive and genius of Peter Cundell, now at Minesota University. It all began, however, in London in 1974, though under a different name (DAMSEL: Dames & Moore, Special purpose Explicit Lagrange code). Peter Cundell had already been cultivating the "dynamic relaxation" and finite difference techniques for rock mechanics purposes, but it was the North Sea oil programme which turned him to soil mechanics.

Chevron, in partnership with Burnah Oil, were planning a gravity platform for the Ninian field. The structure, when eventually placed, was the largest ever man-made moveable object, being over 600,000 tons and having a base diameter of about 140 metres. In the design process, however, there were several foundation issues which were difficult to analyse by conventional methods.

In particular, there was an emerging concern with the possibility of liquefaction due to repeated wave loading, the scale of the structure being such as to lead to undrained conditions (Bjerrum, 1973). Even with the relatively clean sands of the sea bed, the drainage path lengths would be considerable.

There were other concerns as well as the liquefaction and the normal bearing capacity and settlement considerations, notably the "rocking- down" effects and the irregular sea-bed or

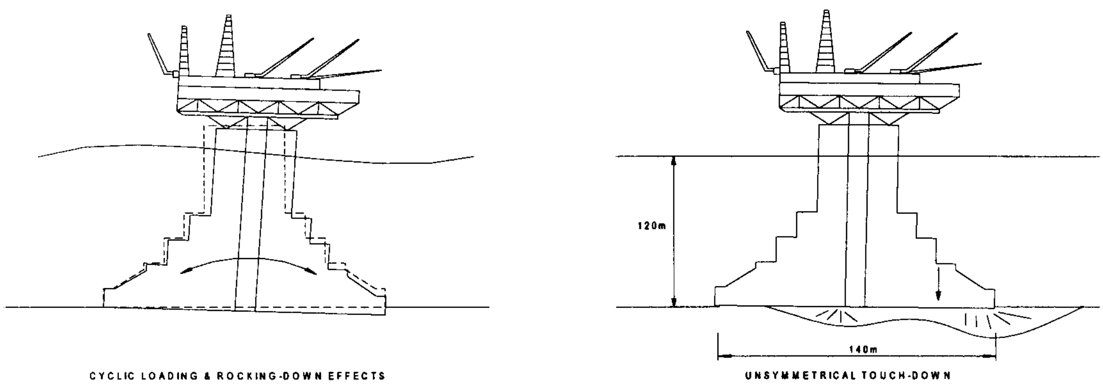


Figure 1: Foundation issues for offshore gravity platform.

unsymmetrical "touch-down" problem (see Figure 1). The former related to the irritating soil characteristic of different unloading and loading stiffness's. Each "sway" would result in less rebound than the first loading and the consequences of this needed investigating. There was even a theory that it could lead to lateral creep. The unsymmetrical touch-down was more a classic bearing capacity problem but nevertheless required estimates of yield to judge the seriousness of the issue.

I ask the readers of this paper, some quarter-century on, to ponder how they would address these same problems, even with the more mature models available.

2.2 Early Differences

Whilst our team pursued the finite difference path, others were addressing the same types of problem through finite element methods (FEM). Now, I believe, the two approaches are almost exactly comparable. Back then, however, there were some major differences, and controversy raged.

Finite element methods began life with elastic structures. They required "implicit" solutions by inversion of stiffness matrices. Large strains and yield are difficult to model, though achieved using complex "high-order" elements. The size of the problems to be analysed were governed by the size of the computer.

The Cundell approach solved the same differential exceptions, but by a quasi-dynamic finite difference method, and involved several important features, namely:

- Explicit solution: each node or zone was analysed explicitly, with each succeeding time step taking account of the effects of adjacent nodes on its state of motion. In effect, the model represents stress waves travelling through the continuum, eventually damped out to a static solution.

- Lagrangian formulation: the grid of nodes and zones was allowed to deform with the continuum (as with numerical models for fluids) so that large strains could be accommodated.

By incorporating a flow rule together with the usual failure constitutive rules, yield could be modelled without any significant increase in complexity. The size of the problem was not so much limited by the size of the computer as the speed (i.e. the time required for interactions throughout the grid). Because the solution was based on dynamics, the method could be extended to dynamic modelling, again without significant complexity.

At this time, much of the effort in programming (usually in Fortran) was devoted to making the calculations more efficient.

2.3 Three-Dimensional Models

The development and application of plane stress/strain or axi-symmetric numerical models progressed at a great pace. The DAMSEL code went through several name changes, eventually becoming FLAC. In the meantime, a three-dimensional version was developed by Peter Cundell for CANMET (Canada Centre for Mineral and Energy Technology).

The problems of 3-D analysis are well known, the major ones being the demands made on computer time and memory. To minimise these, a special 3-D finite difference program was written, but limited to small strains and rectangular geometry. The particular situations to be analysed were characterised by a reasonably rectangular geometry but otherwise involved many different material types, high lateral rock stresses and specific sequences of mining.

My role was to use the model, known then as LS3, to analyse two mines and, in particular, predict if and when vertical faces of cemented backfill would fail. The results have been described in detail (Sinclair et al., 1982), but the main points are summarised as follows:

- Stopes were backfilled with cemented tailings in 32:1 and 16:1 tailings/cement ratios. Backfill was sampled and tested by laboratory tests and pressuremeter.
- Adjoining pillars were mined out in vertical slices, gradually exposing vertical backfill faces, as much as 100 metres high in places.
- Eventual failure of all backfill faces was predicted by the numerical model (refer Figure 2).

The operations modelled for both mines were carried out after the analyses. Failure of the filled stopes did <u>not</u> occur, though some "sloughing" of the backfill was observed. The question now is why no failure?

Possible reasons for the "conservative" prediction include:

a) numerical model unrealistic,
b) sampling and testing disturbances,
c) ageing of backfill, and
d) variability in all materials.

With regard to a), validation tests against known "classical" solutions indicate that the numerical model is compatible with recognised principles of soil mechanics. However, initial stresses and boundary effects may have been significant in this case. On balance, though, it is likely the other factors are more dominant.

2.4 Thermal Problems

A topical issue, at least to those of us from Auckland, is the problem of heat conductivity in soils. In the summer of 1998, four main cables carrying power to the Central Business District of Auckland all failed at once. One possible cause of this is the over-heating of the cables following an unusually prolonged period of hot, dry weather.

In 1978, 20 years ago, I participated in a similar study of heat flow using a finite difference computer program. That study was for the analysis of the transient thermal regime surrounding buried refrigerated NGL pipelines. The purpose was to optimise the thermal insulation designs and to assess the impact of the pipelines on the agricultural environment during long term operations (Sinclair et al., 1979). Also refer to Figure 3. The main point of this discussion, however, is in relation to the soil properties. For a given dry density, thermal conductivity increases with water content. Tests for the refrigerated pipeline project are shown for a well graded sandy gravel. There is a factor of 3 between very dry and saturated. Despite this, the numerical modelling showed there was little difference in the heat flux due to the ranges of soil properties.

Returning to the issue of buried cables, we see there are important differences, as illustrated in Table 1 below.

In summer, when the conductivity is low due to drying out of the soils, and when the temperature gradient between ground surface and conduit is low (and even possibly reversed), the ability to disperse heat from cables is significantly reduced.

This brief discussion highlights the need for yet another area of expertise for geotechnical engineers and examples of the use of numerical models.

TABLE 1: Seasonal heat flux differences

Season	Physical Conditions	Refrigerated Pipelines	Buried Cables
Summer	Temperature Gradient	High	Low
	Thermal Conductivity	Low	Low
	Heat Flux	H x L = Moderate	L x L = Extra Low
Winter	Temperature Gradient	High	High
	Thermal Conductivity	Low	High
	Heat Flux	H x L = Moderate	H x H = Extra High

2.5 More Models

Numerical models have matured and multiplied. With the very rapid increase in computing capability they are now more useful and user-friendly than ever before, both for design and research purposes. They cover a diverse range of physical problems, particularly soil-structure interaction and seepage (or heat) flow. Dynamic and three-dimensional models are now routine and models can be large enough to deal more satisfactorily with the boundary problems. There are numerous solution codes: FEM, DEM, BEM, BIM, FLM etc. They overlap with the more routine engineering issues such as slope stability, wall analysis and pile design. Clearly it would be unwise for any geotechnical engineer to be left behind in this field (as I have been now!). However, the purpose of my earlier examples is to highlight that there are still issues which can severely damage credibility of these analyses, in particular:

- Limitations on our ability to measure properties of materials.
- Ability to represent the real world: the correct geological model, the vagaries of nature, the defects, the Murphy's law of events.

For the Ninian platform in the North Sea, it was necessary to assess bearing capacity assuming it may touch down slightly tilted. For the mine example, we are now able to model incredible complexity but we may never know the degree of complexity needed, for example, in relation to variations in material and interface properties, and initial stresses. For the thermal study, it is necessary to look at the extremes of weather conditions as well as the normal ambient conditions.

A simple mathematical model will predict that a pen placed vertically on its point will remain vertical. To represent the real situation, it would be necessary to model an "imperfect" initial condition or some small perturbation in the environment. In effect, the model must allow for chaos.

3 FUNDAMENTALS

3.1 Common Confusions

Whilst the continuing development of numerical models and other computer methods enable more realistic situations to be analysed and provide insight into complex interactions, there is a danger that the fundamentals of soil mechanics are forgotten, even in the most familiar of circumstances.

I ask the reader to consider the following common-place situations: True or False?

a) A retaining wall used to stabilise a landslide must be designed to withstand the force from the slide, together with the active earth pressures.

b) Slope stability assessments for earthquakes must use total stress analyses (and undrained strengths) because they are short term loadings.

c) Stiff retaining walls must be designed for K_o conditions.

d) Downdrag on piles cannot exceed the weight of soil causing it.

e) Slopes should be checked for fully saturated conditions to allow for extreme rainfall events.

3.2 Golden Cross Landslide

The Golden Cross mine in the Coromandel includes a tailings dam which incorporates the waste rock stack as the downstream shoulder. The tailings are retained by a "main" embankment across a valley, with a "saddle" dam to one side. Construction commenced in 1990 with a starter dam which was progressively raised in accordance with the design and mine operational requirements.

During routine site investigation and surveillance work in August 1995, some cracks were observed in the natural ground adjacent to the tailings dam. On further investigation, other signs of widespread ground movement were discovered and a major site investigation and monitoring programme was initiated. This programme, which included investigation boreholes, the installation of inclinometers and piezometers and precise survey, has continued through to 1998.

The Golden Cross landslide must now be one of the most studied in New Zealand. Several consulting organisations have been involved and many world-renowned specialists have visited.

The landslide deserves a paper of its own and, no doubt, one or more will be forthcoming in the near future.

The purpose of this particular discussion is to demonstrate how the return to simple first principles can help with interpretation of the slide behaviour.

The slide is complex and time and space constraints preclude a detailed description. The figures (Figures 4(a) to 4(f) inclusive) illustrate some of the characteristics.

The slide is about 2 km long, 600 m wide and approximately 60 M m^3 in volume. Certain stabilising measures were put in place, particularly deep dewatering using a drainage tunnel under the slide and a number of pumping wells. Fills were placed at various places, partly as stabilising measures and partly as waste disposal for normal mining operations.

Movements have been monitored using inclinometers and GPS. Typical records are given in the figures. Using all the many monitoring points, it became evident that different parts of the slide were moving at different rates and at different times. Hence the mode of ground movements appears to be one of disjointed block "shuffling", possibly initiated by part rotational movement at the toe with subsequent translational movement of separate blocks of higher ground following on in a "slip-stick' manner.

Numerous stability analyses have been performed throughout the three years since the landslide was recognised. Results obviously depended on the assumptions and chosen model. It was clear at an early stage that analyses would need to take account of the disjointed block relationships.

At this point, it is worth looking at a simple model of a large-scale block, moving on a low-angle shear surface. Because the slide is now known to have moved some distance, the shear surface is certainly at residual strength, with no effective cohesion. The stability of the block can therefore be represented by the simple force diagram shown in Figure 5. The various forces should be self explanatory. The force 'E' is some resultant external horizontal force on the block. The angle θ is given by:

$$\tan\theta = \tan \emptyset / F$$

Therefore the smaller the angle θ, the higher the factor of safety and vice-versa. Anything which reduces the angle θ improves the situation. Hence an extra weight on the block (Δw) improves the factor of safety provided there is no change to the water pressure or external forces. However, if the extra weight is widespread, its effect will be transmitted to the pore pressure at the slide surface with $\Delta u = \Delta w$ (maximum). In this case, the angle θ would be slightly increased initially, eventually reducing as the pore pressures dissipated .($\theta_2<\theta_0<\theta_1$).

This simple fundamental force diagram, straight out of any students lecture notes, can explain the behaviour of the slide. Extra fills on areas where the shear plane is inclined at less than the angle of friction are beneficial in the long term, though may cause some movement initially.

In the early days, the slide also responded almost immediately to rainfall. One reason for this could be the increase in water pressure (u). However, in general the piezometers did not seem to respond as much as would be necessary for this to be the case. It is more likely that rainwater entered the tension cracks and dilated tension zones, thus increasing the external lateral forces (E). In time, the water pressures in these zones would come to equilibrium with the general groundwater profile, and movements would slow.

In conclusion for this matter, I would not suggest that such simple workings are sufficient for analyses. The point is that they help with an understanding of the principles.

4 CONCLUSION

There is now little point arguing against the use of modern tools of the trade such as numerical models. Their benefits and failings are self evident. Perhaps not using them could be judged as negligent in some cases? Careful use provides valuable insight into complex interactions and provides design results, just as any other design method. However, moderate scepticism is healthy.

At the same time, a continual return to basic fundamentals must be of value. In particular, a good understanding of the concepts of effective stress, undrained strength and theory of friction will help with many day-to-day issues.

What is on the horizon? I was intending to list some of the developments in methods of analysis but the task was too big. Research is progressing in so many fields I would be guilty of missing a fair proportion.

One of the more esoteric techniques is neural network modelling. This has been applied, for example, to liquefaction (Goh, 1996), slope movements due to rainfall (Mayoraz et al., 1996) and settlements during tunnelling (Jingsheng Shi et al., 1998). I would be interested to know if this holds promise of being useful to practising engineers.

I return, however, to the concept of chaos. It seems to me that this is more a philosophy than a theory. Nevertheless, our geotechnical world is a chaotic system and our methods of analysis must make allowances for the mathematical imperfections. We do this intuitively, but a formalised theory may be enlightening. The theory of fractals is one of the building blocks of chaos theory. Certainly fractals have already appeared in soil mechanics, for example, in groundwater modelling and the characterisation of the shape of sand particles. Where will it all end?

5 ACKNOWLEDGEMENTS

I acknowledge the contributions and help of many clients and colleagues over the years, too many to name. My various employers, particularly Dames & Moore for 9 years and Tonkin & Taylor Ltd for 15 years, have provided the experience. I thank Coeur Gold NZ Ltd for their permission to give information on the Golden Cross Mine.

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Common Procedures for Foundation Settlement Analysis – Are They Adequate?

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Summary This paper examines and evaluates some common methods of foundation settlement prediction in the light of recent research. Four common problems are considered: settlement of shallow foundations on clay, settlement of shallow foundations on sand, analysis of strip and raft foundations, and the settlement of pile groups. The outcome of the evaluation is a recommendation on whether a method should be adopted, adapted, or discarded. The crucial importance of appropriate assessment of the relevant geotechnical parameters is emphasized.

1. INTRODUCTION

The theme of this present Conference is "Consolidating Knowledge", and it is an entirely appropriate theme for a discipline approaching its 75th birthday, i.e. since the publication of Terzaghi's classic book "Erdbaumerchanik" in 1925. Despite the significant advances made in geotechnical knowledge, particularly over the past three decades, there has been too little effort made to try and evaluate the applicability of some of the commonly-used design and analysis methods. It would appear that the state of practice in traditional areas of foundation engineering lags far behind the state of the art. Cost and time pressures often preclude the use of modern techniques of analysis and design of foundations, and result in the continued use of dated empirical procedures whose validity may be dubious.

The evaluation of commonly used design procedures received a major boost in the 1970's with the Prediction Symposia organized by Lambe and his colleagues (1970, 1973, 1974). Such Symposia attempted to reveal the ability of geotechnical engineers to carry out accurate "Class A" predictions for a variety of practical circumstances i.e. true predictions made prior to the performance details being known. In general, this ability was, at best, variable, and at worst, depressingly inadequate. Subsequent Symposia (e.g. Briaud et al., 1994; Finno et al., 1989; Brand, 1990) have reinforced the findings of Lambe, and demonstrated that accurate prediction of the performance of piles and embankments is dependent as much on the experience of the predictor, and a good amount of luck, as on the adequacy of the method employed. The selection of geotechnical parameters also plays a major part in the success or otherwise of a prediction, and may outweigh or mask any shortcomings of the method used.

The main objective of this paper is to examine and evaluate some foundation settlement prediction procedures in the light of relatively recent research. Ideally, such an evaluation should consider both the theoretical "correctness" of the methods and also their applicability to practical cases. However, primary attention will be paid here to identifying the shortcomings and limitations of the methods when compared to modern theoretical approaches. Four common problems will be considered in this paper:

- Settlement of shallow foundations on clay
- Settlement of shallow foundations on sand
- Analysis of strip and raft foundations
- Settlement of pile groups

In each case, an attempt will be made to suggest a fate for the methods considered, in one of the following three categories: adopt, adapt, or discard.

2. ANALYSIS AND DESIGN METHODS

2.1 Desirable Attributes of Practical Analysis and Design Methods

Among the desirable attributes of practical methods of geotechnical analysis and design are the following:

- they should have a sound theoretical basis
- they should capture the major features of the problem being addressed and incorporate the important parameters
- they should be able to be applied to practice without requiring excessive computational resources
- the geotechnical parameters required in the methods should be able to be estimated by conventional field and laboratory tests
- they should be able to be checked with a simpler approach.

In this context, it is worth bearing in mind the following opinion of Burland (1989):

"Any design that relies for its success on precise analysis is a *bad* design."

2.2 Categories of Analysis and Design Methods

In assessing the relative merits of analysis and design methods, it is useful to categorize the methods in some way. It has been proposed previously (Poulos, 1989) that methods of analysis and design can be classified into three broad categories, as shown in Table 1.

Category 1 procedures probably account for a large proportion of the foundation design performed throughout the world. Category 2 procedures have a proper theoretical basis, but they generally involve significant simplifications, especially with respect to soil behaviour. The majority of available design charts fall into one or other of the Category 2 methods. Category 3 procedures generally involve the use of a site-specific analysis based on relatively advanced numerical or analytical techniques, and require the use of a computer. Many of the Category 2 design charts have been developed from Category 3 analyses, and then condensed into a simplified form. The most advanced Category 3 methods (3C) have been used relatively sparsely, but increasing research effort is being made to develop such methods, in conjunction with the development of more sophisticated models of soil behaviour.

From a practical viewpoint, Category 1 and 2 methods are the most commonly used. In the following sections, attention will be focussed on evaluating such methods with respect to more refined and encompassing methods, many of which either fall into Category 3, or have been derived from Category 3 analyses.

3. SETTLEMENT ANALYSIS OF SHALLOW FOUNDATIONS ON CLAY

Estimation of settlement and differential settlement is a fundamental aspect of the design of shallow foundations. For foundations on clay, Table 2 summarizes some of the available techniques and their capabilities. The traditional approach, first developed by Terzaghi, employs the one-dimensional method in which the settlement is assumed to arise from consolidation due to increases in effective stress caused by the dissipation of excess pore pressures. Because of its still widespread use, it is of interest to examine the capabilities and shortcomings of the method, when compared with more complete two- and three-dimensional methods.

The one-dimensional method has the following limitations:

1. It assumes that the foundation loading causes only vertical strains in the subsoil;

2. It assumes that all the settlement arises from consolidation, and that settlements arising from immediate shear strains are negligible;

3. It assumes that the dissipation of excess pore pressures occurs only in the vertical direction; any lateral dissipation of excess pore pressures is ignored.

Table 1. Categories of analysis and design methods.

Category	Subdivision	Characteristics	Method of Parameter Estimation
1	—	Empirical – not based on soil mechanics principles	Simple in-situ or laboratory tests, with correlations
2	2A	Based on simplified theory or charts – uses soil mechanics principles – amenable to hand calculation; simple linear elastic or rigid plastic soil models	Routine relevant in-situ or laboratory tests – may require some correlations
	2B	As for 2A, but theory is non-linear (deformation) or elasto-plastic (stability)	
3	3A	Based on theory using site-specific analysis; uses soil mechanics principles, Theory is linear elastic (deformation) or rigid plastic (stability)	Careful laboratory and/or in-situ tests which follow the appropriate stress paths
	3B	As for 3A, but non-linearity is allowed for in a relatively simple manner	
	3C	As for 3A, but non-linearity is allowed for via proper constitutive soil models	

Table 2. Some methods of settlement analysis for shallow foundations.

Method	Category	Immediate Settlement	Consolid'n Settlement	Settlement Rate	Creep Settlement
One-dimensional $S_{TF} = S_{CF} = S_{oed}$	2	-	✔	✔	✔ (can be incorporated)
Modified one-dimensional $S_{TF} = S_i + S_{oed}$	2	✔	✔	✔	✔ (can be incorporated)
Skempton & Bjerrum (1957) $S_{TF} = S_i + \mu S_{oed}$	1-2	✔	✔	✔	-
Elastic method $S_{TF} = S_i + (S_{TF} - S_i)$	2A	✔	✔	✔	✔ (can be incorporated)
Modified elastic $S_{TF} = S_i / F_R + (S_{TF} - S_i)$	2B	✔	✔	✔	
Elastic finite element	3A	✔	✔	✔	
Non-linear finite element	3B, 3C	✔	✔	✔	

Note: S_{TF} = total final settlement
S_i = immediate settlement
S_{CF} = final consolidation settlement
S_{oed} = one-dimensional settlement (from oedometer)
μ = correction factor (Skempton & Bjerrum, 1957)
F_R = nonlinear correction factor (D'Appolonia et al., 1971)

3.1 One-Dimensional versus Three-Dimensional Settlement Analysis

To examine the possible significance of these limitations, two very simple hypothetical examples are considered. The first involves a uniformly loaded circular footing resting on the surface of a homogeneous layer of overconsolidated clay, in which the soil stiffness is uniform with depth. The second involves the same footing on a layer of soft normally consolidated clay in which the soil stiffness increases linearly with depth, from a small initial value at the soil surface. The relationship between the one-dimensional compressibility m_v and the drained Young's modulus E' (for the three-dimensional analysis) is assumed to be that given by elastic theory for an ideal two-phase elastic soil skeleton, as is the relationship between the undrained Young's modulus E_u and E', i.e.

$$m_v = \frac{(1+\nu')(1-2\nu')}{(1-\nu')E'} \quad (1)$$

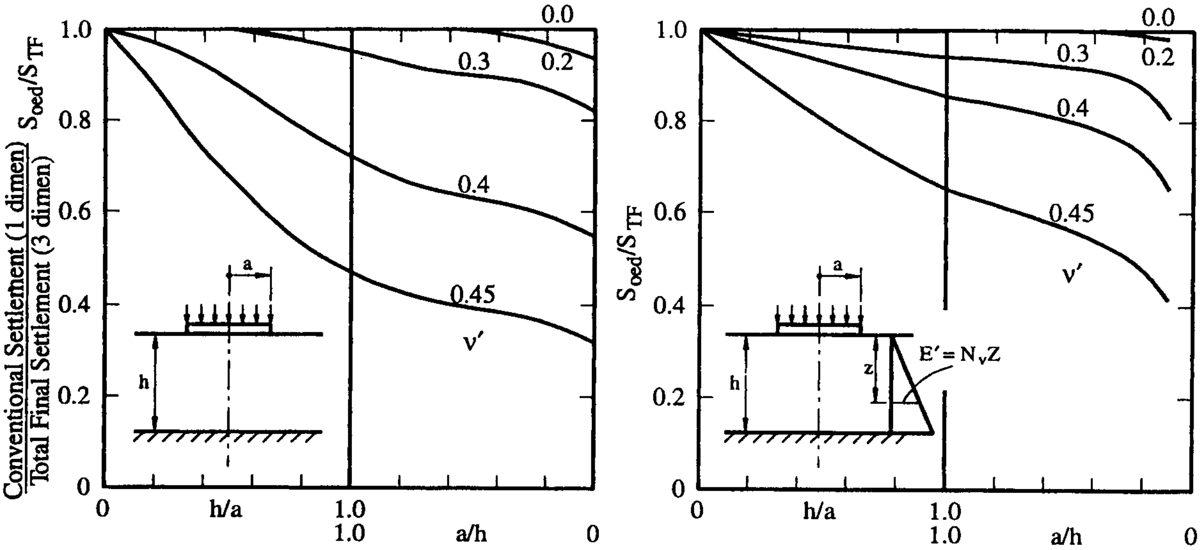
$$E_u = \frac{3E'}{2\ (1+\nu')} \quad (2)$$

where ν' = drained Poisson's ratio of soil skeleton.

Figure 1 shows the ratio of the one-dimensional settlement (excluding creep) to the correct three-dimensional total final settlement (Davis and Poulos, 1968). For the overconsolidated clay layer, the one-dimensional analysis gives a good approximation to the correct total settlement when the drained Poisson's ratio of the soil layer is less than about 0.35, even for relatively deep soil layers. The one-dimensional analysis tends to under-predict the final settlement as the drained Poisson's ratio of the soil increases or the relative layer depth increases. For the soft clay layer, the one-dimensional analysis again gives a remarkably good approximation to the total final settlement if the drained Poisson's ratio of the soil layer is 0.35 or less. It should be noted that, for the theoretical upper limit of ν' = 0.5 (which is not realistic for soils), the one-dimensional method predicts zero final settlement.

Burland et al. (1977) provide a detailed discussion of the ratio of one-dimensional settlement to total settlement, and demonstrate that soil anisotropy can have some influence on this ratio. They also argue that, while S_{oed} is a good approximation to S_{TF} for stiff clays, it is more likely to approximate the final ***consolidation*** settlement S_{CF} for normally consolidated clays, because of the yielding of such a soil and the consequent irrecoverable strains.

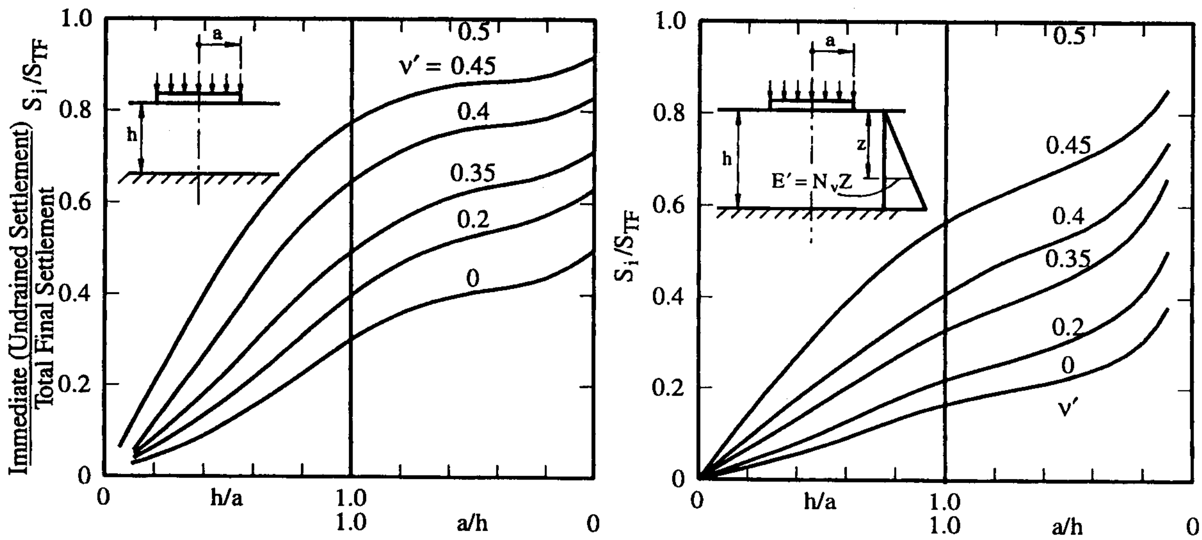
Figure 2 shows the relative importance of immediate settlement as a proportion of the total final settlement. For the overconsolidated clay layer, immediate settlement can be a significant proportion of the total settlement and hence a one-dimensional analysis may give a misleading prediction of settlement, i.e. it suggests that all the settlement arises from consolidation, whereas a significant amount may be immediate settlement. On the other hand, for the soft clay layer, the ratio of immediate to total final settlement is considerably smaller, and one-dimensional theory may provide an adequate prediction of the final settlement. In the latter case, it is also often the case that the undrained Young's modulus is significantly greater than the theoretical value which would be implied by the relationship for an ideal elastic two-phase soil. Thus, the immediate settlement may, in reality, be an even smaller proportion of the total final settlement than is indicated in Figure 2.



(a) Soil layer with uniform modulus (overconsolidated clay)

(b) Soil layer with linearly increasing modulus (normally consolidated clay)

Figure 1. Theoretical ability of one-dimensional analysis to predict total final settlement.



(a) Soil layer with uniform modulus (overconsolidated clay)

(b) Soil layer with linearly increasing modulus (normally consolidated clay)

Figure 2. Theoretical relative importance of immediate settlement.

Burland et al. (1977) quote data for stiff clayey soil which indicate that the ratio of immediate to final settlement is consistent with the predictions of the elastic analysis in Figure 2a. For example, Simons and Som (1970) studied 12 case records and found an average ratio of immediate to final settlement of 0.58 (range 0.32 to 0.74), while a study of 8 buildings on London clay by Morton and Au (1974) indicated an average ratio of 0.63 (range 0.4 to 0.82).

For normally consolidated clays, Simons and Som (1970) reviewed 9 case histories of buildings on normally consolidated clays, and found an average ratio of settlement during construction to total settlement of 0.16 (range 0.08 to 0.21). As noted by Burland et al. (1977), some consolidation probably took place during construction, so that the value of S_i/S_{TF} was probably of the order of 0.1. Such values are again consistent with Figure 2b. It should be noted that the above conclusions are relevant to structural foundations at normal safety factors. For other cases, such as embankments on soft clays at low safety factors, the immediate settlement may be a more dominant factor because of plastic yielding of the soil.

3.2 Effects of Local Yield

The commonly used methods of settlement analysis implicitly assume elastic behaviour of the soil over the range of stress applied by the foundation. While some allowance is made for nonlinear soil response by distinguishing between normally consolidated and over-consolidated states, and using different values of compressibility for each, no allowance is made for the development of local yield within the soil due to foundation loading. Davis and Poulos (1968) considered the conditions affecting the onset of local yield and showed that the applied loading at which local yield commenced was a function of both the factor of safety (i.e. the ratio of applied pressure q_u to ultimate bearing capacity q_u) and also the initial stress state. D'Appolonia et al. (1971) extended these concepts and developed correction factors for the effects of local yield on immediate settlement. In their approach, the immediate settlement S_I was given as:

$$S_i = S_{ielas} / F_R \tag{3}$$

where

S_{ielas} = immediate settlement computed from elastic theory

F_R = yield correction factor, a function of q/q_u and the initial stress ratio f, which is a function both of coefficient of earth pressure at rest and undrained shear strength. F_R is 1 for elastic conditions, and less than 1 when local yield occurs in the soil.

The effects of local yield on consolidation settlement and ratio of settlement do not appear to have been studied extensively. However, solutions presented by Small et al. and Carter et al. (1979) suggest that both the consolidation settlement and rate of settlement and the rate of settlement are not greatly affected by local yield within the soil, and that elastic theory can be used with sufficient accuracy for their estimation. Thus, the main influence of local yield is on the immediate settlement S_i, and for Category 2 methods of calculation, equation (3) can be used to estimate S_i.

In summary, it would appear that one-dimensional settlement theory can be ***adopted***, but with some measure of adaptation. In particular, there is clear evidence that immediate settlements are important and cannot be ignored, especially for stiff clays. The effects of local yield on immediate settlement should also be considered. Based on the theoretical relationships discussed above, it would appear reasonable to adopt the following procedures if employing one-dimensional settlement theory:

1) for stiff overconsolidated clays:

$$S_{TF} = S_{oed} \tag{4a}$$

$$S_{CF} = S_{oed} - S_i \tag{4b}$$

2) for normally consolidated clays:

$$S_{TF} = S_{oed} + S_i \tag{5a}$$

$$S_{CF} = S_{oed} \tag{5b}$$

The predicted settlement for the second case may be conservative, but since S_i is often relatively small in comparison to S_{oed}, the potential extent of over-estimation is unlikely to be significant in most cases.

3.3 Case Study

The performance of the various methods of settlement analysis can be gauged by applying them to a real case history in which measurements of settlement are available. The case selected for consideration here is that published by Moore and Spencer (1969). This case involved a 2-storey brick structure erected in 1890 on a thick layer of compressible soil in South Melbourne. Figure 3 shows a plan of the building and a simplified representation of the soil profile. The average loading acting on the building was about 45kPa. Oedometer and laboratory stress path triaxial test data were obtained on the dominant clay and silt layer, and these data are also shown in Figure 3.

Moore and Spencer employed a number of methods of settlement calculation, ranging from one-dimensional settlement analysis to two different types of stress path analysis. Three types of oedometer test were carried out for use in the conventional oedometer analysis.

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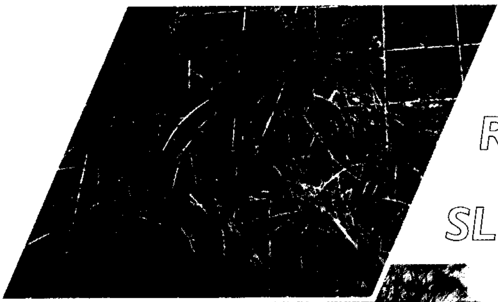

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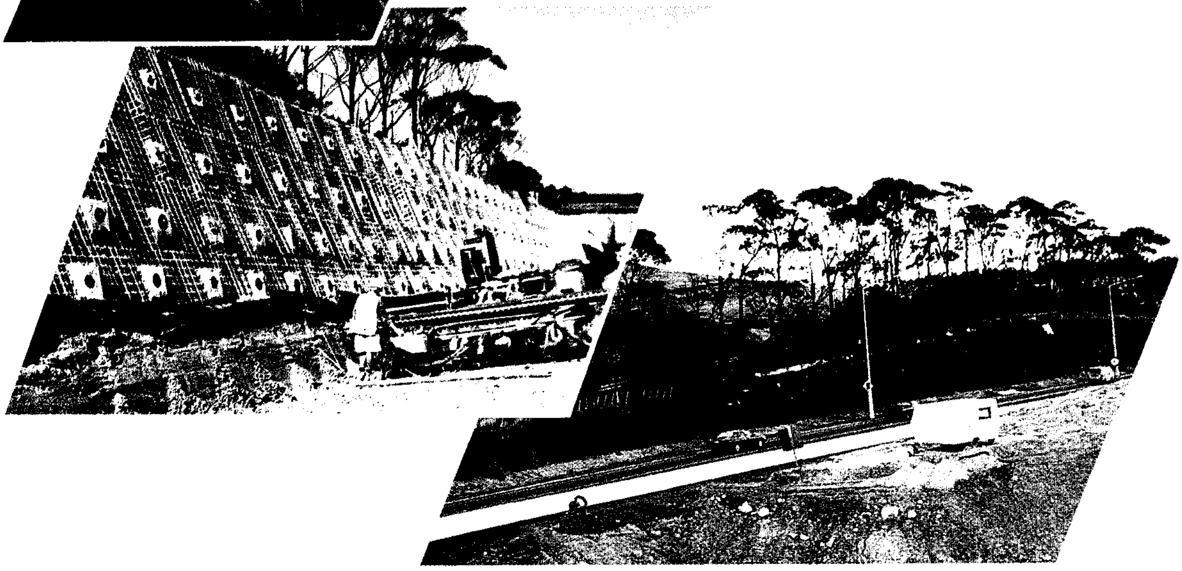

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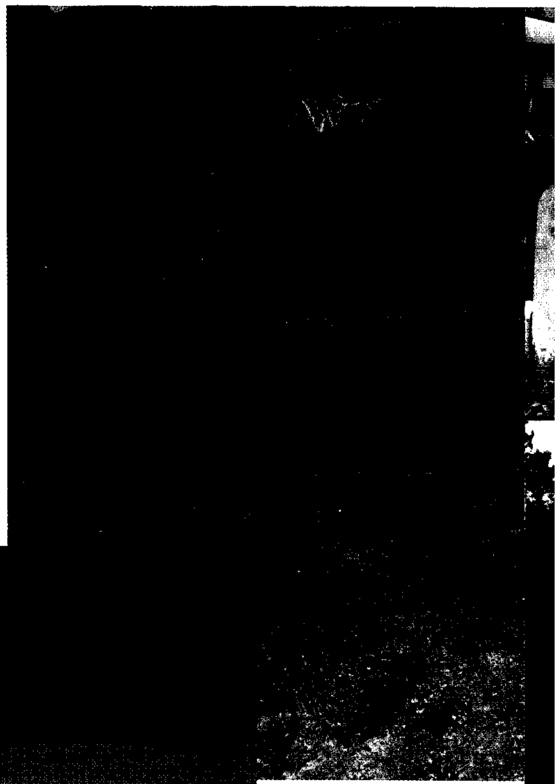
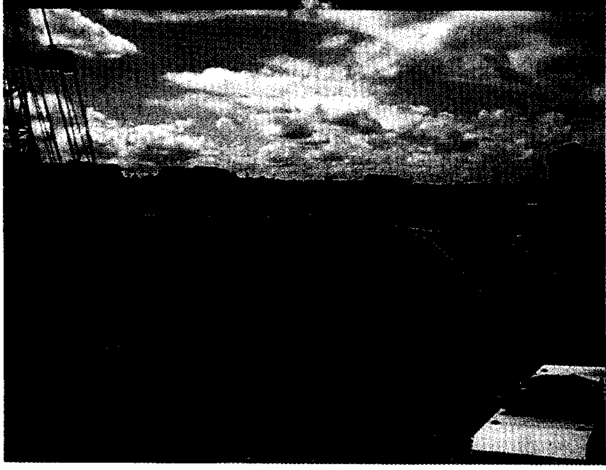
Left: Dewatering a Landslide from underground drainage drive

Right: Pahoehoe Lava, Mt Teneriffe

Below: Piling Rig, SH20

Below Left: Hydro. dam construction





Top Right: Retaining Wall

Far Right: Tension crack, Whangarei

Right: Landslide at Shore Rd, Auckland

Below: Waikato River flooding

Below left: Soil Nailing, M25, UK

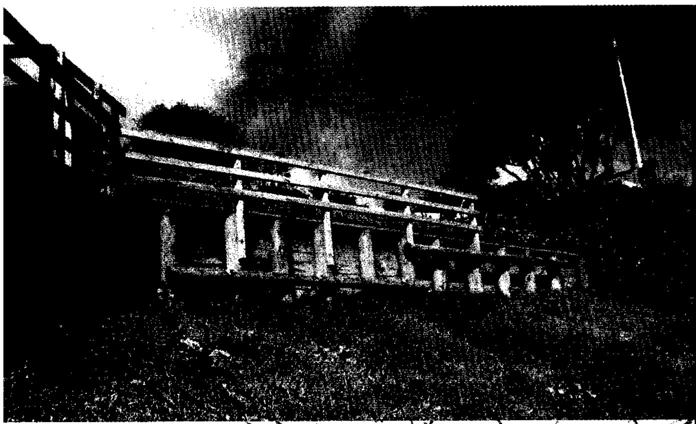
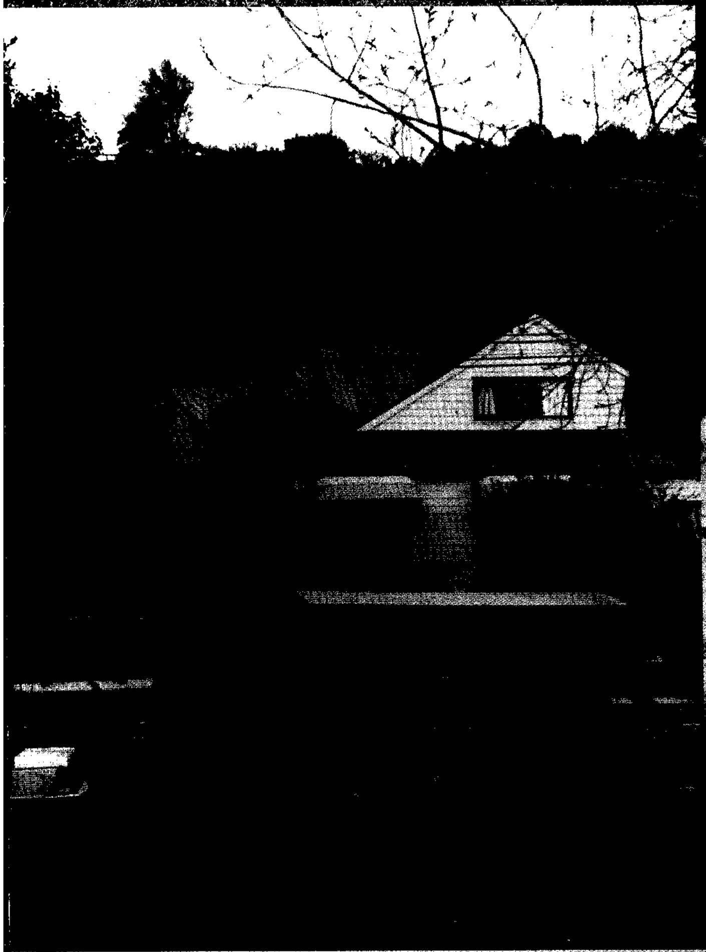



PHOTO MONTAGE: The Photos that never made the competition

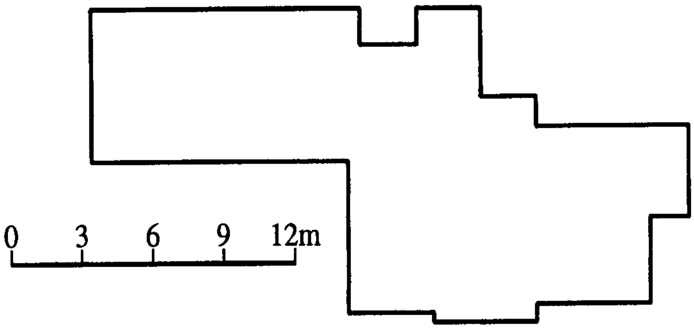
The results of the calculations reported by Moore and Spencer are shown in Table 3, together with the measured settlement.

The effects of possible local yield have not been taken into account, and hence the immediate settlements are likely to have been under-estimated. It can be seen that the conventional one-dimensional analysis under-estimates the settlement significantly, (depending on which type of oedometer data has been employed), as does also the Skempton and Bjerrum method. The elastic method of David and Poulos (1968) under-estimates the settlement by only about 10%, while the Lambe stress path method gives a 23% underestimate. While this case study can in no way be used to imply that a three-dimensional settlement analysis is always necessary, it does indicate that caution should be exercised in applying one-dimensional settlement analysis to foundations on soft clay. On the other hand, the earlier studies of Skempton, Peck and MacDonald (1955) and Skempton and Bjerrum (1957) provide evidence that one-dimensional analyses provide an adequate estimate of foundations on relatively stiff clays. Thus, the one-dimensional

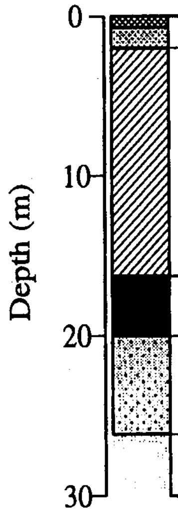
Table 3. Comparison of total settlements: Boyd Domestic College Building. (After Moore and Spencer, 1969)

Method	Settlement mm
Observed	787
Conventional Oedometer	508
Oedometer with Back Pressure	404
Oedometer with Constant Rate of Strain	404
Skempton and Bjerrum (1975)	564
Davis and Poulos (1968)	709
Stress Path, Lambe (1964)	610

Note: Oedometer methods used stress distributions for 2-layer soil mass.



Plan outline of building



Layer	Nat w/c %	LL	PL	Stress Path Triaxial tests	Oedometer Tests (conv'l) m_v m^2/MN
Fill, firm silty sand	26 - 33	26	23	--	--
Soft black clays & silts, some organic matter & shell fragments	49 - 90	91 - 98	30 - 47	E_u = 34.74 MPa E' = 0.52 MPa ν' = 0.40 A = 0.92 (at repres. depth of 8.5 m)	1.32 0.50
Soft peaty sandy silt	40 - 69	83	38 - 44	--	--
Grey silty sand	24 - 33	29	21	--	0.15
Med-coarse sand & gravel	--	--	--		

Figure 3. Boyd Domestic College Building, South Melbourne (After Moore & Spencer, 1969).

method deserves to be retained, but ***adapted***, as a means of estimating the settlement of shallow foundations on clay.

3.4 Rate of Settlement

It is well known that three-dimensional effects may significantly accelerate the rate of settlement of foundations on clay, primarily because of the ability of excess pore pressures to dissipate horizontally as well as vertically. A number of approaches have been developed for estimating the rate of settlement under two and three-dimensional conditions, including the following:

1. Category 2 design charts for strip and circular foundations on a homogeneous layer e.g. Davis and Poulos (1972), Booker (1974); the former are derived from solutions of the simplified diffusion theory of consolidation, while the latter are based on the complete Biot theory.
2. Finite layer numerical solutions for strip, circular and rectangular footings on elastic clay layers e.g. as implemented by the computer program CONTAL (Small, 1998). This would be classified as a Category 3A approach.
3. Finite element numerical analyses for linear and non-linear soil layers, e.g. Small et al. (1976); Sandhu and Wilson (1969).

From a practical viewpoint, it may not always be feasible to employ a full two-or three-dimensional consolidation analysis. However, it is possible to ***adapt*** a one-dimensional consolidation analysis to take account of this effect by using an equivalent coefficient of consolidation, c_{ve}, which is obtained by multiplying the actual coefficient of consolidation c_v by a geometrical rate factor R_f:

$$c_{ve} = R_f \cdot c_v \quad (6)$$

R_f values can be derived from three-dimensional rate of settlement solutions, such as those derived by Davis and Poulos (1972). Figures 4 and 5 show values of R_f as a function of the layer depth to footing size ratio, for circular and strip foundations. In each case, three combinations of hydraulic boundary conditions at the top and base of the layer are considered: PTPB (permeable top, permeable base), PTIB (permeable top, impermeable base) and IFPB (impermeable top, permeable base). As the layer thickness increases relative to the footing size, the factor R_f increases, reflecting the acceleration of the rate of settlement due to lateral dissipation. It should be noted that for the case IFIB (impermeable footing, impermeable base), it is not possible to adapt a one-dimensional solution since the theoretical rate of settlement is always zero. If the soil is anisotropic, then a further factor can be applied to the coefficient of consolidation, as presented by Davis and Poulos (1972).

An example of a comparison between solutions for the rate of settlement of a large flexible circular foundation (such as an oil tank) on a layered clay soil profile is shown in Figure 6. Three solutions are shown: a three-dimensional solution from a finite layer consolidation analysis using the program CONTAL (Small, 1998), a modified one-dimensional analysis in which the coefficients of consolidation have been multiplied by

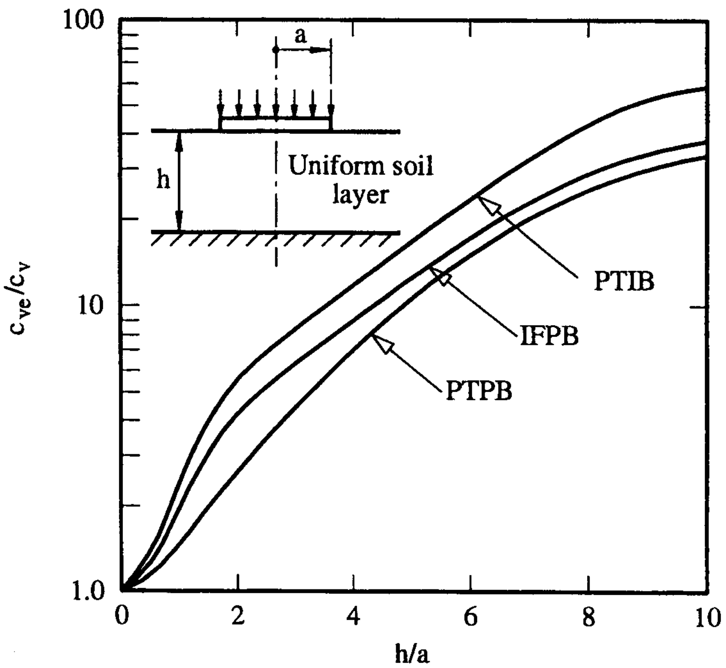


Figure 4. Equivalent coefficient of consolidation for 1-D analysis of rate of settlement - circular footing.

an R_f factor of 6 (see Figure 4 for the PTIB case), and a one-dimensional numerical analysis in which the original values of coefficient of consolidation are used. The modified one-dimensional solution is in reasonable agreement with the three-dimensional solution (although it suggests a somewhat more rapid rate of consolidation at larger times). The conventional one-dimensional numerical analysis significantly under-predicts the rate of settlement.

The author has used the above approach in several practical cases with good results, for example, for an embankment on clay (Poulos and Davis, 1975). A comparison between the measured rate of settlement and that calculated from a one-dimensional analysis using the equivalent coefficient of consolidation, shows quite reasonable agreement, certainly better than would have been obtained using a normal one-dimensional consolidation analysis. Thus, it would appear feasible in practical cases to adapt the one-dimensional consolidation analysis, as suggested above, if a proper three-dimensional analysis is either unwarranted or not available.

3.5 Creep and Secondary Consolidation

The existence of creep complicates the prediction of both the magnitude and rate of settlement of foundations on clay soils. Most practical methods of accounting for creep still rely on the early observations of Buisman (1936) that creep is characterized by a linear relationship between settlement and the logarithm of time. The gradient of this relationship is generally represented by the coefficient of secondary compression C_α,

$$C_\alpha = \Delta e / \Delta \log t \qquad (7)$$

where Δe = change in void ratio
and t = time

Mesri and Godlewski (1977) have found that C_α is related to the compression index of a soil, as indicated in Table 4. It should be noted that, for over-consolidated clays, the ratios in Table 4 apply to the recompression index; thus, the creep settlement rate is significantly smaller for an overconsolidated soil than for the same soil in a normally consolidated state.

The difficulty with applying the 'C_α' concept is that the time at which creep is assumed to commence is not well defined. Considerable controversy exists on this point, with some researchers assuming that creep only commences at the end of consolidation (e.g. Mesri et al., 1994) while others contend that it takes place simultaneously with primary consolidation (e.g. Leroueil, 1996).

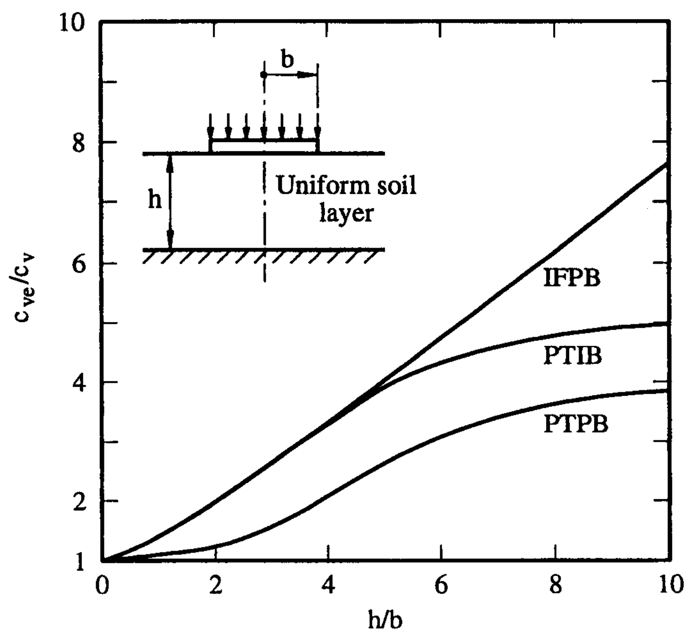
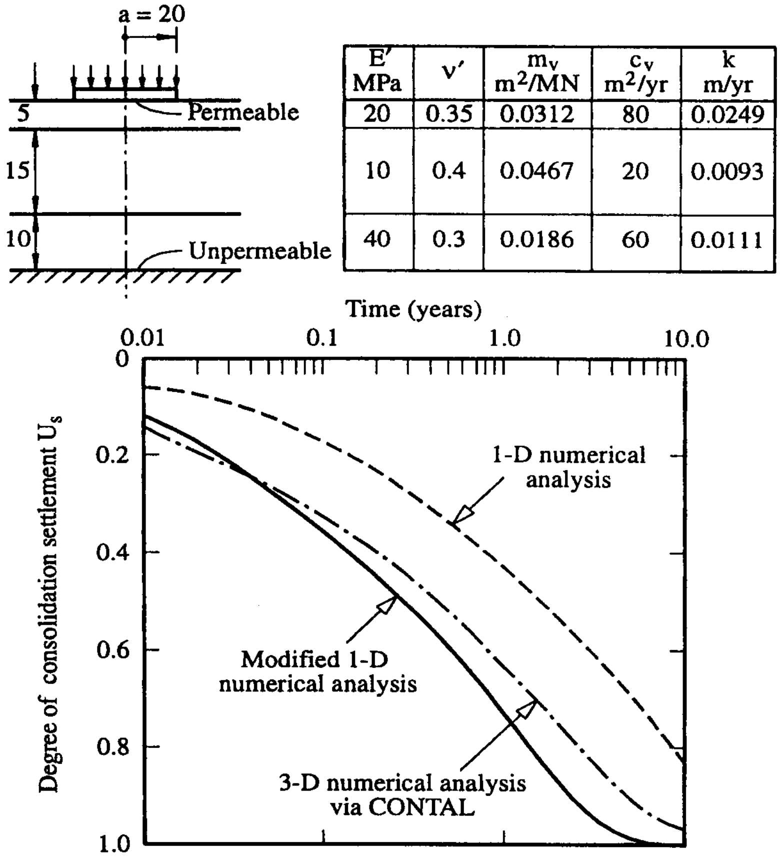


Figure 5. Equivalent coefficient of consolidtion for 1-D analysis of rate of settlement - strip footings.



E' MPa	ν'	m_v m^2/MN	c_v m^2/yr	k m/yr
20	0.35	0.0312	80	0.0249
10	0.4	0.0467	20	0.0093
40	0.3	0.0186	60	0.0111

Figure 6. Example of comparison between 3-D & modified 1-D rate of settlement analyses.

Table 4. Values of C_α/C_c for geotechnical materials. (Mesri et al., 1994)

Material	C_α/C_c
Granular soils, including rockpile	0.02 ± 0.01
Shale and mudstone	0.03 ± 0.01
Inorganic clays and silts	0.04 ± 0.01
Organic clays and silts	0.05 ± 0.01
Peat and muskeg	0.06 ± 0.01

While various creep laws can be and have been incorporated into consolidation analyses (eg. Gibson and Lo, 1961; Garlanger, 1972), it is very uncommon in practice for such analyses to be applied, even for one-dimensional problems.

From a practical viewpoint, the most convenient approach appears to be to add the creep settlement relationship to the conventional time-settlement relationship from consolidation theory, commencing at one of the following times:

1) a predetermined time after commencement of loading
2) after a predetermined degree of consolidation settlement
3) when the gradients of the primary settlement versus log time and the creep settlement versus log time relationships are equal.

While all three alternatives are arbitrary, the last of the three appears to be the easiest and most logical to apply.

Overall, it appears that, of all the aspects of settlement analysis, the issue of creep and secondary consolidation is the one in which least progress has been made in terms of fundamental understanding and in the incorporation of research into practice. In the absence of a more satisfactory approach, the method of Buisman may be adapted to provide a crude estimation of creep settlements.

4. SETTLEMENT OF SHALLOW FOUNDATIONS ON SAND

4.1 Previous Studies

A remarkable number of methods have been developed to estimate the settlement of shallow foundations on sand, yet consistent success in accurately predicting such settlements remains elusive. These methods range from purely empirical (Category 1) methods developed originally for conservative footing design (Terzaghi and Peck, 1967), to complex Category 3 nonlinear finite element methods.

Many of the methods rely on in-situ SPT or CPT data, and hence it is not possible to satisfactorily examine the theoretical relationship between the various methods. Assessments of the performance of various methods have therefore been made on the basis of comparisons with measured settlements. At least two significant studies have been reported, one by Jeyapalan and Boehm (1986), and the other by Tan and Duncan (1991).

The study by Jeyapalan and Boehm (1986) involved the statistical analysis of 71 case histories for which settlements of footings on sand were reported, and the assessment of the relative accuracy of nine methods of settlement estimation. The methods of Schultze and Sherif (1973) and Schmertmann (1978) appeared to among the more dependable approaches.

Tan and Duncan (1991) carried out an assessment of the reliability of twelve methods of estimating footing settlement on sands by comparing calculated and measured settlements for 76 cases. Each of the methods was evaluated in terms of (1) accuracy (the ratio of average calculated to measured settlement); (2) reliability (the percentage of cases in which the calculated settlement equalled or exceeded the measured settlement, and (3) ease of use (the length of time required to apply the method. Table 5 summarizes the methods considered and the parameters used in each method. Figure 7 summarizes the findings on accuracy and reliability. Values of "accuracy" range from 1.0 (the ideal value) for Alpan's method to 3.2 for Terzaghi and Peck's method. Values of "reliability" varied from 34% for Schultze and Sherif's method to 86% for Terzaghi and Peck's method. In

Table 5. Variables used in methods of estimating settlements of footings on sand. (Tan and Duncan, 1991)

	Variables Used											
Method (reference)	N	N_{cor}	q_c	B	D_W	D_f	γ_t	L	T	Soil Type	Str. Hist.	Time
Alpan (1964)		✔		✔	✔	✔	✔				✔	
Burland and Burbridge (1985)	✔			✔	✔	✔	✔	✔	✔	✔	✔	✔
D'Appolonia & D'Appolonia (1970)	✔			✔	✔	✔			✔			
Duncan & Buchignani (1976)	✔			✔						✔		✔
Meyerhof (1956)	✔			✔								
NAVFAC (1982)	✔			✔	✔							
Parry (1971)	✔			✔	✔				✔			
Peck & Bazaraa (1969)		✔		✔	✔	✔	✔			✔		
Peck, Hanson, Thornburn (1974)		✔		✔	✔	✔	✔					
Schmertmann (1978)			✔	✔	✔	✔	✔					✔
Schultz & Sherif (1973)	✔			✔		✔			✔			
Terzaghi and Peck (1967)	✔			✔	✔					✔		

N = SPT Blow Count
q_c = Cone Penetration Test tip resistance
D_f = depth of footing below ground surface
T = thickness of sand layer below footing
Time = duration of loading
N_{cor} = SPT Blow Count corrected for overburden pressure

B = footing width
γ_t = total unit weight of sand
Soil Type = silty or clean sand
D_W = depth of water table
L = footing length
Stress Hist. = max. prev. load

general, the methods which were less accurate (and more conservative) were more reliable in the sense that they underestimated the settlement relatively infrequently. Table 6 summarizes the hand computation times for a simple example. Those methods requiring correction of the SPT values generally involved the longer computation times. As concluded by Tan and Duncan, there is a tradeoff between accuracy and reliability in choosing a method of calculation.

4.2 Case Study

A comparison of the performance of a number of the methods has been made via a well-documented Prediction Symposium in which a number of people made "Class A" predictions of the settlement of footings on a natural sand profile (Briaud and Gibbons, 1994). The predictions were then compared with the actual settlement measurements.

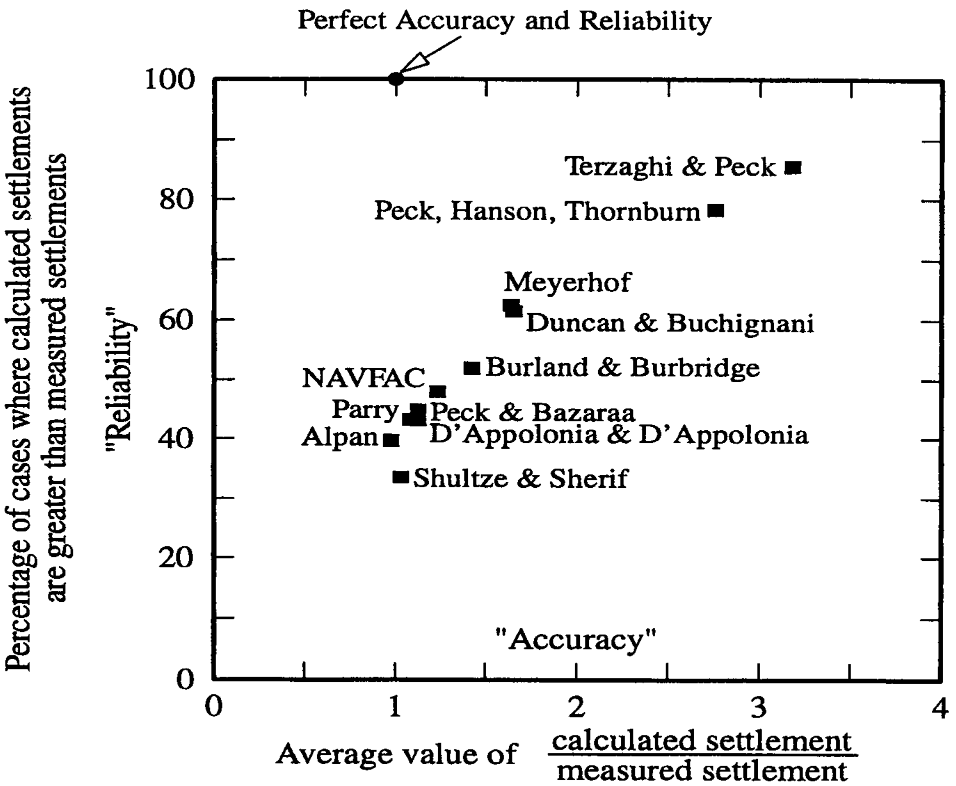


Figure 7. Relationships between accuracy and reliability for eleven methods based on SPT blow count. (Tan and Duncan, 1991).

Table 6. Computation times for methods based on SPT blow count. (Tan and Duncan, 1991)

Method	Computation Time (minutes)
Alpan (1964)	29
Burland & Burbridge (1985)	14
D'Appolonia & D'Appolonia (1970)	8
Duncan & Buchignani (1976)	9
Meyerhof (1956)	6
NAVFAC (1982)	8
Parry (1971)	9
Peck & Bazaraa (1969)	25
Peck, Hanson, Thornburn (1974)	25
Schultze & Sherif (1973)	6
Terzaghi & Peck (1967)	11

Figure 8 summarizes the soil conditions near one of the footings tested (footing 1, nominally 3m by 3m in plan). The site consisted by layers of silty sand, underlain by a hard clay layer. A substantial amount of in-situ and laboratory data was obtained, some of which is shown in Figure 8.

In the Prediction Symposium, a total of 31 persons made predictions, using a wide variety of methods. However, for the present exercise, the author has made his own application of a number of the methods to Footing 1, as well as presenting the original prediction made for this footing. An exception is the result of a finite element analysis, which was carried out by one of the other predictors.

Table 7 summarizes the results of the author's calculations for the settlement at a load of 4000 kN (corresponding to a factor of safety against ultimate failure of about 2x5). The following observations can be made from Table 7:

1. all methods over-estimated the footing settlement
2. the elastic-based methods, based on CPT, SPT and pressuremeter data, all give reasonable predictions
3. the Terzaghi and Peck method, which is meant to provide a conservative footing design to ensure a settlement less than 25mm, gives a predictably conservative settlement estimate
4. the author's original prediction, based on an elastic analysis with a strain-dependent Young's modulus, overpredicts the settlement significantly
5. the finite element method, using a nonlinear constitutive soil model, overpredicts the settlement substantially.

While it is again imprudent to draw firm conclusions on the basis of such limited comparisons, it does appear reasonable to suggest that the simple elastic-based methods (including Schmertmann's method) appear capable of providing reasonable estimates of footing settlement. The key to success lies more in the appropriate choice of the shear or Young's modulus of the sand than in the details of the method employed. Such methods therefore deserve to be retained and *adopted*. On the other hand, the more complex finite element methods appear to require far more development before being able to be used with confidence. Indeed, from a practical viewpoint, there may be relatively few cases in which such analyses are warranted.

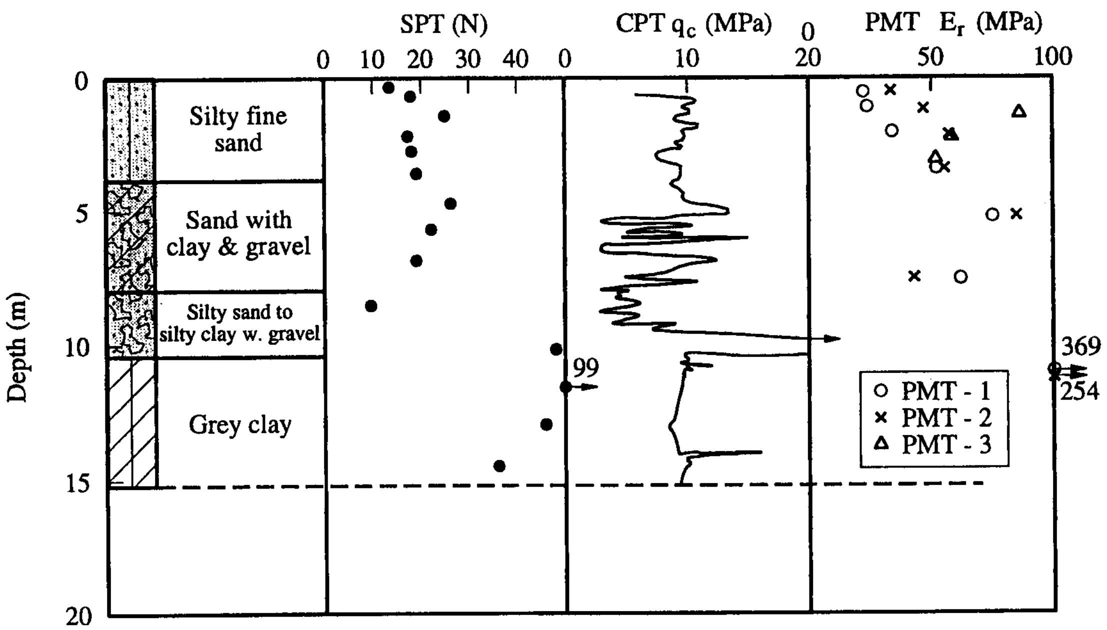


Figure 8. Summary of soil conditions near Footing 1.

Table 7. Summary of calculated & measured settlement of 3m square footing.

Method	Settlement for P = 4MN	Notes
Terzaghi & Peck (1957)	39	Av. N = 20
Schmertmann (1978)	28	
Burland & Burbridge (1985)	21	Average value (range 10-58 mm)
Elastic theory, using E_s = 3N MPa	18	Decourt (1989)
Elastic theory, using PMT data	24	Reload modulus values
Strain-dependent modulus	32	Poulos (1996), Class A prediction
Finite element analysis	75	Chang (1994), Class A prediction, using constitutive soil model
Measured	14	After 30 minutes

5. ANALYSIS OF RAFT AND STRIP FOUNDATIONS

5.1 Introduction

The analysis and design of raft and strip foundations usually involves the following assessments:

- bearing capacity under the design loadings
- settlements and differential settlements
- bending moments and shears for the structural design of the foundation.

Attention will be focussed here on the latter two aspects. Ideally, analyses should take account of the stiffness of the raft or strip, together with the stiffness of the structure being supported. Such structure-foundation-soil interaction analyses, while becoming more common with major structures, are still the exception rather than the rule, and most analyses ignore the effects of superstructure stiffness.

5.2 Subgrade Reaction versus Elastic Continuum Soil Models

Table 8 summarizes and categorizes a number of methods commonly used for the analysis and design of raft and strip foundations. All but the simple rigid footing approximation give settlements and differential settlements, as well as moments and shear forces. The majority of these methods consider the stiffness of the raft or strip, and differ primarily in the manner in which the supporting soil is modelled. There are two usual methods of modelling of the soil:

1. by use of the subgrade reaction method, in which the soil is modelled as a series of independent springs (often called the "Winkler spring model" after one of the originators of the concept)
2. by use of elastic continuum theory, in which the soil is modelled as an elastic continuum.

The first approach has long been favoured by many structural and foundation engineers because of its theoretical convenience, and because, prior to the computer age, analytical solutions were available for strip foundations resting on a Winkler soil model. However, despite its theoretical convenience, the Winkler soil model has a number of important limitations which are not always appreciated. These include the following:

1. A Winkler soil model only deflects if a pressure is applied to it. Thus unloaded areas in a Winkler soil model do not deflect, and hence there is no stress transmission or interaction within the soil
2. A Winkler soil responds to loading only in the direction of that loading. Thus, for example, vertical loading will produce only vertical displacements, and no horizontal displacements
3. A Winkler soil is usually characterised by the modulus of subgrade reaction, which has units of force/length^3. The modulus of subgrade reaction is NOT a fundamental soil parameter, but is dependent on the dimensions of the foundation.
4. A Winkler soil model cannot incorporate properly the effects of soil layering since it does not allow stress transmission. The assessment of the modulus of subgrade reaction for a layered soil profile therefore involves considerable uncertainty which is sometimes resolved by resorting to elastic theory to obtain an equivalent value.

Table 8. Method of analysis of raft and strip foundations.

Method	Category	Remarks	Typical References
Rigid footing assumption	1	Does not give settlements	Bowles (1984)
Strip on Winkler Soil	2A	Closed form solutions	Bowles (1984)
Strip on elastic soil	2A	Approximate equations for deep layer	Vesic (1961)
Design charts for strip on elastic soil	2A	Concentrated loadings, deep layer	Brown (1975)
Design charts for raft on elastic soil	2A	Uniform loadings only, finite layer	Fraser & Wardle (1976); Brown (1969)
Strip on elastic soil or Winkler soil	3A	Computer program GASP	Poulos (1991)
Raft on Winkler Soil	3A	Computer program based on finite elements	Bowles (1984)
Raft on elastic soil	3A	Finite elements for raft	Wood (1977)
Raft on nonlinear soil	3B	Approx. allowance for local soil yield and raft lift-off; program GARP	Poulos (1994a)

The first two limitations are at variance with our knowledge of real soil behaviour, while the third has led to some significant difficulties, with inadequate designs arising from the use of subgrade reaction moduli which have not been corrected for the footing dimensions.

It is of interest to examine the relationship between solutions for a loaded strip foundation on Winkler and elastic continuum soil models. Brown (1977) has presented comparisons between the computed bending moments for a strip footing subjected to increasing numbers of concentrated loads. The relative stiffness of the strip, K, is defined as follows:

$$K = 16EI\,(1-\nu_s^2)/\pi E_s L^4 \qquad (8)$$

where EI = bending stiffness of strip
E_s = Young's modulus of soil
ν_s = Poisson's ratio of soil
L = length of strip.

The Young's modulus and modulus of subgrade reaction values have been chosen such that the settlements of a rigid strip with a single central load are equal.

Figure 9 shows the comparison for a single central load and reveals quite reasonable agreement for a variety of relative stiffness values K of the strip. Figures 10 and 11 show similar comparisons for 3 and 5 loads equally spaced along the strip. The differences between the solutions becomes greater as the number of loads increases, and the general "dishing" effect which the elastic model reveals is not exhibited by the Winkler model, because the latter cannot consider interaction and stress transmission through the soil. In the extreme case of a uniform loading along the entire strip, the Winkler soil model predicts ZERO bending moment at all points in the strip, whereas the elastic model gives significant moments. In general, it may be concluded that the subgrade reaction approach may provide reasonable estimates of bending moment (and shear force) for strips subjected to isolated concentrated loads, but it becomes increasingly unsatisfactory as the loading becomes more distributed in nature.

5.3 The Analysis of a Raft as a Series of Strip Footings

It is common design practice to analyse a raft foundation by dividing it up into a series of strip footings and analysing each strip as an independent foundation subjected to the loadings applied on that strip. A simple example of this procedure is illustrated in Figure 12. While convenient, this procedure has a number of limitations, including:

- the strip method cannot give torsional moments in the raft;
- there will generally be an incompatibility between the computed settlements at the junction of the intersecting strips.

Assuming the case shown in Figure 12, and an elastic continuum soil model, Table 9 compares the key performance characteristics computed from the strip analysis and that computed from a proper analysis of the raft as a plate. The strip solutions have been obtained from the computer program GASP (Poulos, 1991) while the raft solutions are from the program GARP (Poulos, 1994).

Two solutions from the strip analysis are shown, one in which the strip sections are assumed to be isolated independent strips, and the other in which the effects of loads on the raft area outside the strip is taken into account (the 'interacting strip' solution). Assuming that the GARP analysis is the 'benchmark' solution, the following observations are made:

a) the analysis using isolated independent strips underestimates both the settlement and bending moments

b) the interacting strip solution gives a good estimation of the maximum settlement, but underestimates the minimum settlement

c) the interacting strip solution tends to underestimate the maximum bending moments.

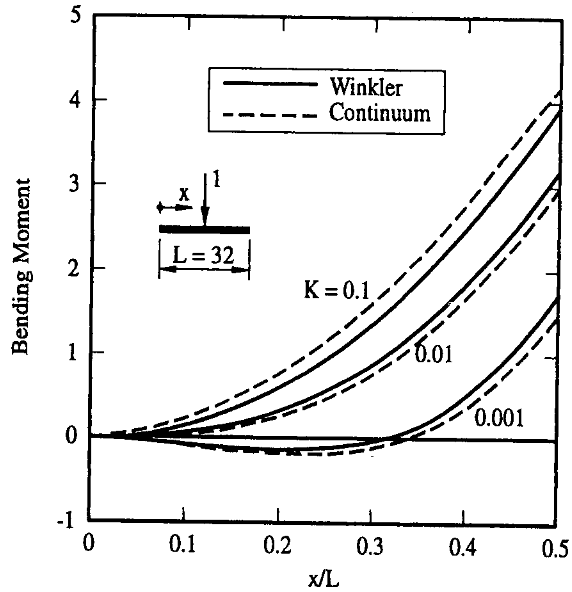


Figure 9. Bending moment distributions using Winkler and continuum soil models (Brown, 1977).

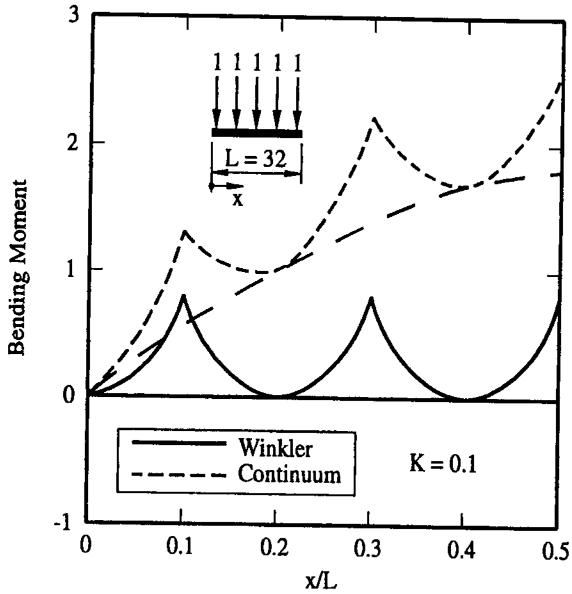


Figure 11. Bending moment distributions using Winkler and continuum soil models (Brown, 1977).

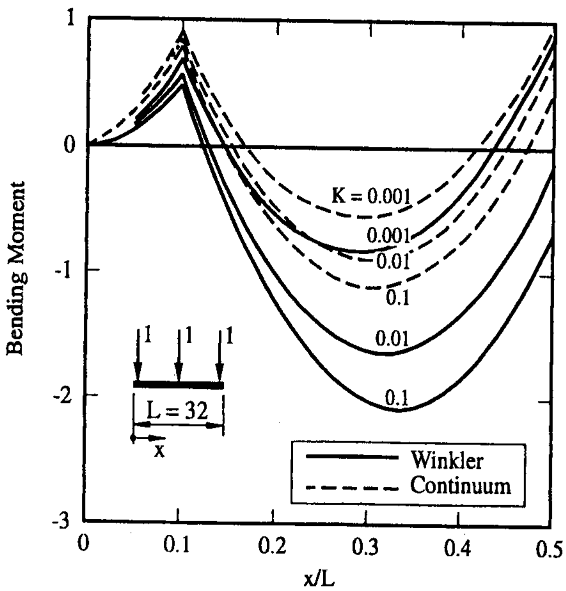


Figure 10. Bending moment distributions using Winkler and continuum soil models (Brown, 1977).

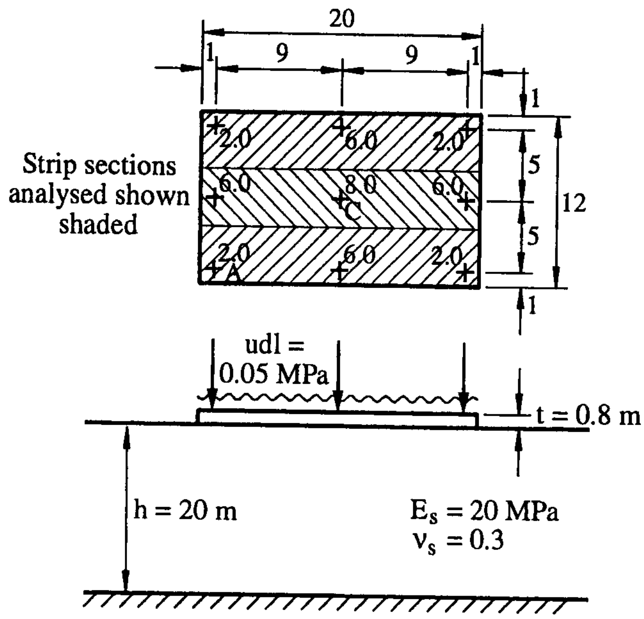


Figure 12. Example of raft analysis.

Overall, the performance of the strip analysis is disappointing and of some concern since it tends to err on the unconservative side as far as bending moments and structural design are concerned, although conversely it tends to be conservative when estimating the differential settlement between the columns in the case considered. In general, it would appear that strip analyses used to be viewed with caution, and it may be appropriate for some further research to be carried out in order to develop better procedures of ***adaptation*** of the strip method to raft analysis.

5.4 The Effects of Structure–Foundation–Soil Interaction

It has been recognised for many years that the stiffness of a structure will affect the distribution of settlements along a strip or raft foundation, and that in turn, the distribution of structural loads and moments will be affected by the foundation flexibility. Methods of incorporating the foundation-soil interaction into a settlement analysis have been described by several authors, including, Lee and Brown (1972), Lee (1975) and Poulos (1975). In general, it has been found that the stiffness of the structure generally leads to a reduction in the differential settlements, compared to the usual methods which take the structural loads as being constant and statically determinant. An excellent example of the improvement in differential settlement prediction which may result from incorporating the structural stiffness is presented by Lopes and Gusmao (1991). For a 15 storey apartment building in Brazil, supported by a system of strip footings, the settlement distribution is predicted more closely if the stiffness of the structure is included in the settlement analysis (see Figure 13).

Lee (1975) has studied the effects of raft flexibility on the column loads in two-dimensional and three-dimensional structural frames, and has found that increasing raft flexibility leads to a more uniform distribution of structural loads than is the case for a rigid foundation (the usual case assumed by structural analysts). Lee also found that the use of the Winkler soil model predicted the reverse trend, and attributed this incorrect trend to the different settlement profiles which emerge from the subgrade reaction theory. Lee made the following observation: "With the advent of large high speed computers, the justification for the Winkler model is removed, and it is clear that it is now only of historical importance¼this is no real reason for its continued use". In the intervening 24 years, computer power has increased by orders of magnitude, yet there is still an unfortunate but widespread persistence with the Winkler concept because of its convenience and simplicity. The price of this simplicity is high, given the potential for unreliable and unrealistic results and the enduring problem of assessing an appropriate modulus of subgrade reaction. The time has come for the Winkler concept to be consigned to history, and not to be perpetuated in modern-day structural and geotechnical analyses.

Table 9. Comparison of computed performance of raft.

Quantity	Calculated Value		
	Raft Analysis elastic cont'm soil	Strips with Extl. Areas	Isolated Strips
Settlement at EC mm centre col.	88.8	88.4	68.2
Settlement at A mm outer col.	75.2	55.0	33.6
M_{XX} at AC MNm/m	2.90	1.83	1.57
M_{yy} at AC MNm/m	2.40	1.08	1.12
M_{xx} at EA MNm/m	0.22	0.18	0.16
M_{yy} at A MNm/m	0.32	0.21	0.19

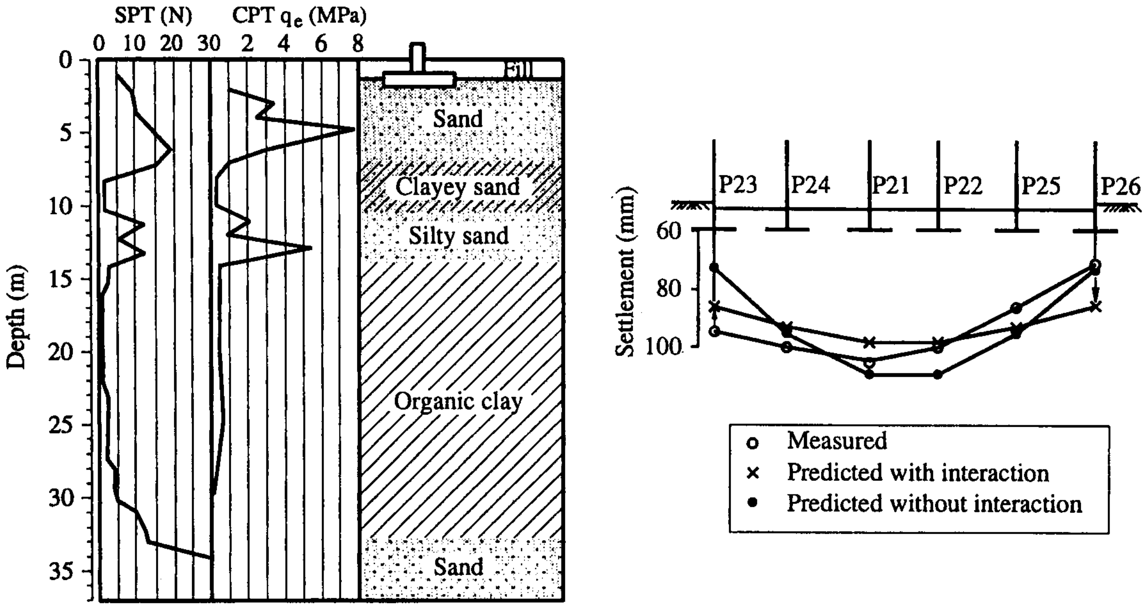


Figure 13. Effect of including structure-foundation interaction on predicted settlements (Lopes and Gusmao, 1991).

6. SETTLEMENT OF PILE GROUPS

Table 10 summarizes a selection of the many methods available for estimating the settlement of pile groups. Further details of some of these methods are given by Poulos and Davis (1980), Fleming et al. (1992), Poulos (1993, 1994) and Randolph (1994).

Attention here will be concentrated on a comparison between solutions from the interaction factor method, the equivalent pier method, and the equivalent raft method. Two idealised problems have beer analysed, as illustrated in Figure 14:

1) a floating or friction pile group, where the founding layer is underlain by a layer of different stiffness (Figure 5a);
2) an end-bearing pile group which is founded on a stratum which is stiffer than the overlying soil (Figure 5b).

For each problem, an examination is made of the influence of the number of piles and the relative stiffness of the two layers on the group settlement predicted by the three methods. The interaction factor method has been implemented via the computer program DEFPIG (Poulos, 1990).

Figure 15 plots the settlement of the floating pile group as a function of the number of piles in the group, for three values of E_2/E_1 and for typical pile spacing and pile-soil parameters. For small numbers of piles, the equivalent raft method tends to underestimate settlement, as compared with the DEFPIG (interaction factor) analysis. However, for nine or more piles, the three methods generally agree well. There is a tendency for the equivalent pier method to underpredict the settlement if the underlying layer is relatively stiff but overall, it appears that the approximate methods are capable of providing a reasonable prediction of the settlement of floating pile groups.

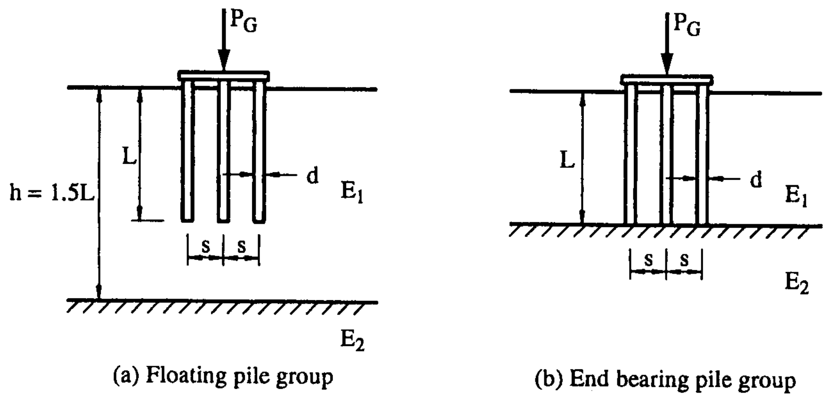


Figure 14. Pile group cases analyzed.

Table 10. Some methods of settlement analysis for pile groups.

Method	Category	Settle-ment	Diff. Settlem't	Pile Loads	Rate of Settlem't	Notes
Settlement Ratio $\rho_G = R_S \cdot \rho_1$	1	Y	-	-	-	R_S from empirical expressions (Skempton, 1953; Meyerhof, 1959) for sands
Meyerhof (1976) $\rho_G = \frac{0.9q\sqrt{B}\ I\ mm}{N}$	1	Y	-	-	-	q=net press.(kPa) B=group width(m) N=av. SPT within depth of B below piles I=(1-D¢/8B)≥0.5
Equivalent Raft (Tomlinson, 1993; Poulos, 1993)	2	Y	Y	-	Y	Various modifications are available eg. Hirayama 1995.
Equivalent Pier (Poulos, 1993)	2	Y	-	-	Y?	Treat group as single pier of piles and soil
Settlement ratio $\rho_G = R_S \cdot \rho_1$	2	Y	-	-	-	R_S from elastic analysis; can approximate as $R_S=n^w$
Interaction Factor (Poulos & Davis, 1980)	3	Y	Y	Y	-	Can be implemented via programs such as DEFPIG & PIGLET
Boundary Element e.g. Banerjee & Driscoll (1976); Poulos & Hewitt, (1986); Zhang & Small (1999)	3	Y	Y	Y	-	Implemented by programs such as PGROUP
2-D Finite Element	3	Y	Y	Y	Y*	Can idealize as plane strain or axi-symmetric
3-D Finite Element	3	Y	Y	Y	Y*	Various soil and pile models can be implemented

* Requires modelling of pore pressures and consolidation behaviour of soil.

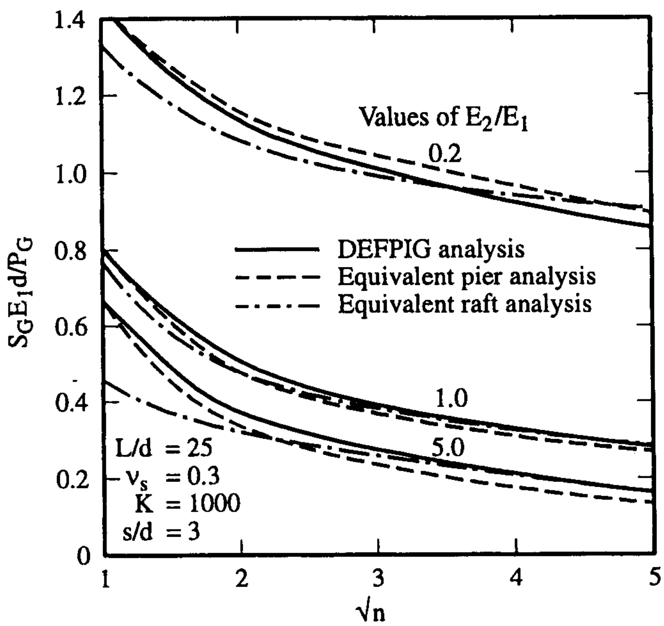


Figure 15. Comparison between solutions for total final settlement. Floating pile group in soil underlain by different layer.

Figure 16 compares solutions for the settlement of an end bearing group, for the case $E_2/E_1 = 5$. The equivalent raft method here significantly overpredicts the settlement for relatively small numbers of piles, but provides a satisfactory solution for 16 or more piles. Conversely, the equivalent pier method tends to underpredict the settlement as the number of piles in the group increases.

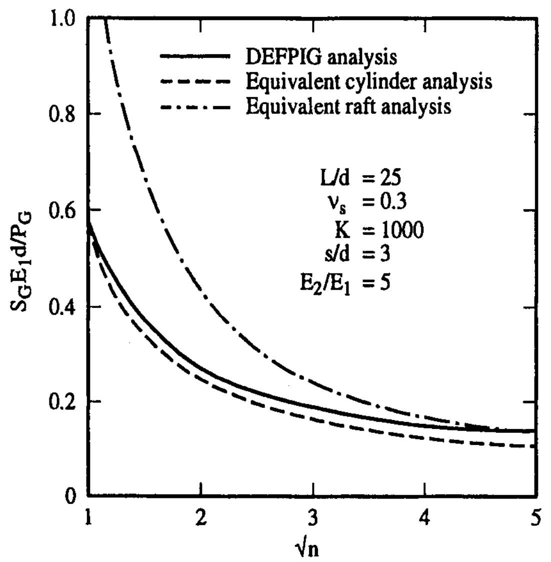


Figure 16. Comparison between solutions for total final settlement. End bearing group on stiffer layer.

In summary, the foregoing comparisons suggest that:

1) for groups containing a relatively small number of piles, the interaction factor method or the equivalent pier method can be used with some confidence.

2) for groups containing more than about 16 piles, the equivalent raft method (implemented as described in this paper) can provide a useful approach for group settlement prediction.

The inaccuracies involved in the use of the equivalent pier or equivalent raft methods are likely to be significantly less than the uncertainties involved in assessing the geotechnical parameters.

6.1 Case Study

Goosens and Van Impe (1991) have described a case involving a block of 40 cylindrical reinforced concrete silos, each 52 m high and 8 m in diameter, that covered a rectangular area 34 m by 84 m. A 75 m high tower block was also constructed adjacent to the silos. The silos were built on a 1.2 m thick foundation slab which was supported by a total of 697 driven cast in situ reinforced concrete piles. The pile length was 13.4 m and the shaft diameter was 0.52 m. The diameter of the expanded base was variable, but was judged to have an average value of 0.8 m. The average pile load under operating conditions was about 1.3 MN.

Figure 17 shows a simplified geotechnical profile near the centre of the site, average values of the static cone resistance, and assessed values of Young's modulus of the various layers.

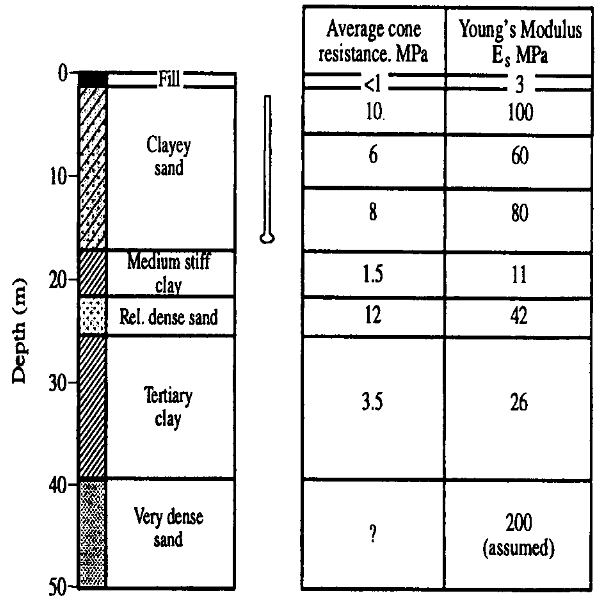


Figure 17. Geotechnical profile and model for case of Goosens and Van Impe (1991).

In the DEFPIG analysis, a ratio of 3 for small strain modulus to near-pile modulus was assumed, and the computed settlement ratios for 16 and 25 piles were extrapolated to give a value for 697 piles. The settlement ratio exponent was found to be 0.743, thus giving a settlement ratio of about 130 for the 697 piles. At the average load per pile of 1.3 MN, the computed single pile settlement was 5.0 mm, which is rather greater than the average measured value, from two pile load tests, of about 2.8 mm. The computed average group settlement was thus 650 mm. Because settlements were measured only along the outside of the silo, it was necessary to correct the computed average settlement to obtain the settlement along the outer edge of the group. On the assumption that the foundation is sufficiently large to behave as a flexible area, the settlement of the central edge is about two-thirds of the average value; consequently, the estimated settlement at the centre of the outer edge was 440 mm. Smaller settlements could have been predicted had a larger ratio of small strain to near-pile moduli been adopted, but there was little basis for assessment of this ratio in this case.

At the centre of the edge, the calculated settlement for the equivalent raft analysis is 181 mm which agrees extremely well with the measured value of 185 mm. This analysis was also carried out for various points along the periphery of the raft, in order to obtain a detailed comparison between computed and measured settlements. This comparison is shown in Figure 18, and reveals remarkably good agreement both with respect to magnitude and distribution. As is also evident, the DEFPIG analysis grossly overestimates the settlement. Had the measured single pile settlement been used for the group calculation however, the calculated settlement at the edge would have been 246 mm, which is considerably closer to the measured value than the original calculation.

This case has also been considered by Mandolini and Viggiani (1997), using an analysis similar in principle to that used in DEFPIG. However, there were some important differences in their analysis, in particular, the use of a hyperbolic nonlinear analysis for the single pile based on a low-strain shear modulus, a different approach to the computation of the interaction factors, and the use of a maximum spacing beyond which interaction effects do not occur. The settlement profile computed by Mandolini and Viggiani is also shown in Figure 18, and is found to be in good agreement with the measured profile. The authors therefore conclude that "a proper application of the interaction factors method can give satisfactory results even for large groups".

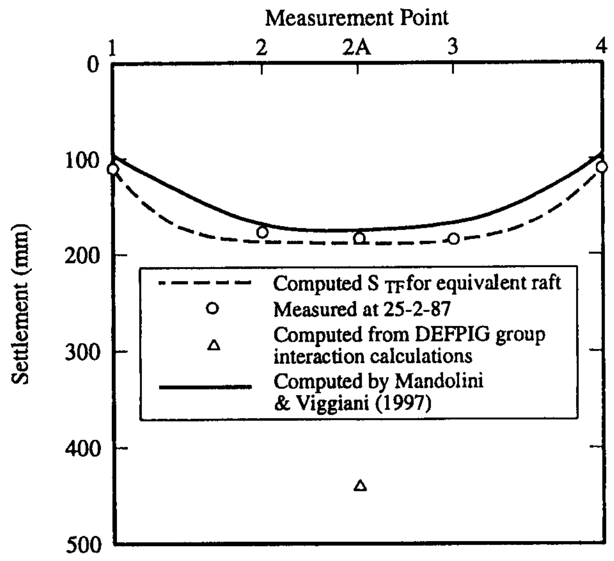


Figure 18. Computed and measured settlement along edge of silo.

While the work of Mandolini and Viggiani is encouraging, it also demonstrates that the success of some Category 3 methods may well depend on the assumptions employed and the choice of parameters, and that they will not necessarily always give a closer prediction of settlements than simpler equivalent raft and equivalent pier methods. It would appear that the latter methods can be retained and ***adopted***, (and perhaps adapted as suggested by Hirayama, 1995, to improve their performance). Interaction factor methods also deserve to be retained, particularly as they provide a means of predicting both settlements and pile loads. However, some ***adaptation*** of the original method (such as that described by Mandolini and Viggiani, 1997) may prove beneficial and may result in settlement estimates which are less conservative than the original approach, and more in agreement with measured behaviour.

7. THE PIVOTAL ROLE OF PARAMETER ASSESSMENT

It is axiomatic that an essential ingredient for successful settlement prediction is the selection of appropriate geotechnical parameters. It is the author's experience that settlement predictions are far more sensitive to the geotechnical parameters and site characterization than to the method of analysis (e.g. Poulos, 1989). Limitations of space preclude a detailed discussion of methods for assessing parameters, but there appear to be number of "maxims" which appear worthy of recognition. These include the following:

1) All practical methods of settlement analysis involve simplification of the soil behaviour to enable calculations to be made readily, e.g. the idealization of soil as an elastic material. The parameters describing this simplified behaviour must therefore not be considered to be "constants".

2) Because soil behaviour is generally non-linear and highly dependent on the effective stress state and the stress path, tests to assess the simplified parameters should be carried out using appropriate initial stresses and applied stress paths. For example, the stress-path method of Lambe (1964) and the laboratory triaxial test procedure proposed by Davis and Poulos (1968) attempt to subject a laboratory sample to a stress history similar to that in the field.

3) Extreme caution must be exercised when applying elastic theory to soils for the purpose of predicting settlements. First, the distinction between undrained and drained behaviour needs to be made for clay soils. Second, an elastic modulus derived for the settlement analysis of a spread footing may not be appropriate for the settlement analysis of a pile, because of the differences in the stress history caused by pile installation and by the predominant load transfer mechanism of the pile being shear rather than normal stress.

4) When assessing soil parameters from in-situ tests, it is desirable for the stress path followed by the test to be similar to that for the foundation being considered. Thus, for example, the results of a plate bearing test would be more relevant to a shallow foundation than to a pile foundation.

5) Different values of the equivalent elastic modulus may be relevant to different components of the settlement. As an example, Figure 19 shows that there may be four distinct values of modulus for a pile or pile group. The behaviour of a single pile is likely to be affected primarily by the soil

modulus values immediately adjacent to and below the pile, whereas a pile group may be influenced considerably by the soil modulus values away from and below the pile tips.

6) Empirical correlations between simple in-situ tests (e.g. SPT, CPT) and soil deformation parameters are often valuable for preliminary estimates of settlement. Because such tests do not generally follow the correct stress path, the potential for inaccurate settlement predictions is generally greater than if more appropriate in-situ or laboratory testing is carried out. It must be borne in mind that most empirical equations are dependent on both soil type and on foundation type, and that indiscriminate use of a correlation may lead to unsatisfactory results. As an example, the correlation developed by Schmertmann (1978) for the Young's modulus applies to shallow foundations on sand, but would be quite inappropriate for piles in clay.

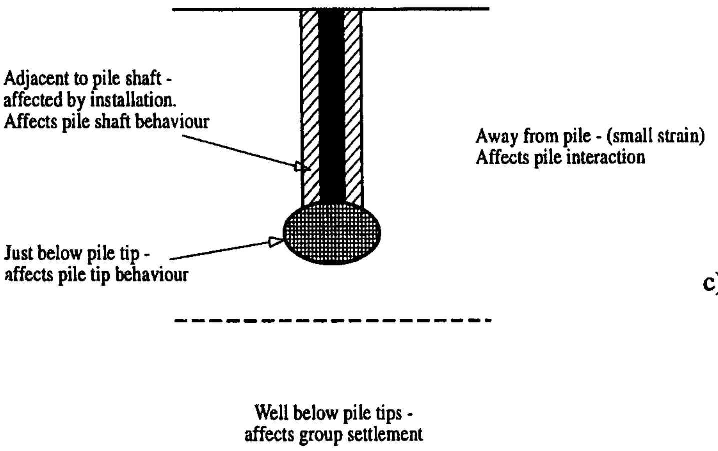


Figure 19. Different soil modulus values around a pile.

Burland (1989) emphasizes the existence and significance of non-linearity of soil behaviour, even when the strains are small. He also summarizes backfigured undrained Young's modulus values from various categories of field problems, and shows that, for a given value of load factor, the value of apparent Young's modulus is highly problem-dependent. Significantly higher values are obtained for pile foundations than for shallow footings or strutted excavations, or values from laboratory triaxial tests.

8. CONCLUSIONS

This paper has reviewed methods of settlement analysis for various types of foundations and has attempted to assess the capabilities of conventional methods of analysis and design in the light of more modern methods developed from research over the past two to three decades. Based on this review, it has been suggested that the conventional methods of settlement analysis should be adopted, adapted or discarded. The results of this assessment may be summarized as follows:

a) Methods which may be adopted -

- Schmertmann's method for settlement of shallow footings on sands
- elastic method for settlement of shallow footings on sands
- elastic analysis of raft foundations
- equivalent raft analysis for pile groups
- equivalent pier analysis for pile groups

b) Methods which should be adapted -

- one-dimensional settlement analysis of shallow footings on clay (make allowance for immediate settlement)
- one-dimensional rate of settlement analysis for shallow footings (make allowance for three-dimensional geometry and soil anisotropy)
- linear creep/secondary settlement versus log time relationship (need to consider carefully when creep commences)
- strip analysis for rafts (allow for loaded areas outside the strip section analyzed)

c) Methods which may need to be discarded -

Methods for shallow foundations based on subgrade reaction concepts; while they may sometimes give satisfactory results for isolated loadings, they can be misleading for uniform loadings and may also create difficulties with the selection of modulus of subgrade reaction because of its dependence on the foundation dimensions.

It must be stated that the above assessments contain a certain element of subjectivity. Also, it is vital to recognize that the ultimate success of settlement prediction depends as much (if not more) on appropriate modelling and parameter selection than on the method of analysis used.

In conclusion, it is sobering to recall the following comments of Terzaghi (1951):

"....foundation engineering has definitely passed from the scientific state into that of maturity...one gets the impression that research has outdistanced practical application, and that the gap between theory and practice still widens."

The gap to which Terzaghi referred is far greater now, almost fifty years later, and it would seem appropriate that a major effort be mounted for the beginning of the new millennium to assess the current state of practice in various aspects of foundation engineering, and incorporate relevant aspects of modern research and state of the art knowledge into practice.

9. ACKNOWLEDGEMENTS

The author is fortunate to have had the privilege of collaborating with a number of distinguished colleagues who have contributed significantly to the state of knowledge of foundation settlement analysis, and in particular, the late Professors E.H. Davis and J.R. Booker, the late Dr P.T. Brown, and Professor T William Lambe. Grateful acknowledgement is also given to Professor J.P. Carter and Associate Professor J.C. Small for their generous assistance in the preparation of this paper.

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HOW STRONG ARE CLOSELY-JOINTED ROCK MASSES?

Stuart Read & Laurie Richards

Closely-jointed rock masses, such as those of Mesozoic-age greywacke, are widespread in New Zealand, and defining their strength provides a significant geotechnical challenge. Engineering designs typically use the Hoek-Brown failure criterion, which was first introduced in 1980 and mainly applied to underground openings. Since then, the failure criterion has been used for an ever-increasing range of both surface and subsurface rock engineering problems while undergoing several significant revisions.

To date, the Hoek-Brown failure criterion has been applied in New Zealand to dam foundations, highway batters, open-pit slopes, and research projects, particularly in greywacke terrane. In practice, application has not been straightforward and rock mass strength estimates have often been influenced as much by a practitioner's judgement as by detailed definition of input parameters or calibration for New Zealand rocks. Challenges remain for linking geological field data to the failure criterion (initially by means of the Rock Mass Rating or more recently the Geological Strength Index), and determining reliable values of the Hoek-Brown constants by laboratory testing.

With this background, the Institute of Geological & Nuclear Sciences, in collaboration with Laurie Richards, Rock Engineering Consultant, has obtained six-year funding from the Public Good Science Fund (PGSF) administered by the Foundation for Research Science and Technology (FRST). Work on the project started in 1998, and a paper has been submitted to the 9th International Society of Rock Mechanics (ISRM) Congress in August 1999 - *Applicability of the Hoek-Brown failure criterion to New Zealand greywacke rocks.*

The paper, a copy of which follows this article, concludes that the Hoek-Brown criterion can give misleading results for the rock mass strength of unweathered New Zealand greywacke lithologies, and that care must be taken in its application. Initial determinations of the Hoek-Brown constant m_i give values for sandstone and mudstone that differ from that given by Hoek & Brown, and high unconfined compressive strengths can lead to high estimates of rock mass strengths. The paper also offers an alternative approach for determining rock mass deformability.

The paper is based largely on data collected from three project study sites (Aviemore in the South Island, and Belmont & Taotaoroa in the North Island), localities where there are extensive exposures of unweathered greywacke. Additional data from other localities throughout New Zealand are essential for verifying the conclusions to date, as well as for developing an appropriate failure criterion. Interaction with the geotechnical community provides not only a broadening of the database of greywacke properties, but also develops linkages with industry that can increase benefits from the research and produce greater confidence in future rock engineering designs.

To this end, ***the form following this short article*** has been prepared to extend our knowledge of the type of projects and the range of applications and problems, together with the scope and type of data available. We are requesting information from sites in greywacke terrane, in particular those with unweathered greywacke. We are interested in past as well as current projects, and are happy to receive responses from both first-hand and second-hand sources. The purpose of the form is to identify projects and data available so that appropriate follow up can be made. A separate copy of the form should be completed for each project - following US Government form response guidelines the reporting burden should be no more than a few minutes.

All responses will be acknowledged, and if you require additional information please do not hesitate to contact Stuart Read *(contact details at the bottom of the form).*

CLOSELY-JOINTED ROCK MASSES

Request for information on projects in greywacke

Project name:
Brief description:
Location:

Date of activity:

NZMS 260 grid reference:

Information type	Yes (tick box)	No	*Details if known and/or comments*
Geological mapping:			*Mapping scale, scope; type of classification*
• Surface mapping			
• Discontinuity surveys			
• Rock mass classifications			
Subsurface investigations:			*No. and length (m)*
• Boreholes			
• Test pits/shafts/adits			
Laboratory testing:			*Type and no.*
• Uniaxial compressive strength			
• Modulus of elasticity			
• Triaxial strength			
• Other (e.g. point load)			
In situ testing:			*Type and no.*
• Geophysics			
• Other (e.g. plate bearing)			
Design approach based on:			*Empirical, precedent, or site specific*
• Mohr-Coulomb parameters			
• Hoek-Brown criterion			
• Back analysis			
Performance monitoring:			*Include type of monitoring*
• Deformation			
• Other (e.g. load)			
Documentation:			*Provide copy of publications, if possible*
• Client report			
• Publication			

Additional comments: (e.g. presence of failures suitable for further investigation; confidentiality restrictions)

Other relevant projects:

9

Form returned by:	*Suggested information source (if different):*
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PO Box 30 368
LOWER HUTT Ph: (04) 570 4728 Fax: (04) 570 1440

Applicability of the Hoek-Brown failure criterion to New Zealand greywacke rocks

Applicabilité du critère de rupture de Hoek-Brown aux grauwackes néo-zélandais

Anwendbarkeit des Hoek-Brown Bruchkriteriums für neuseeländische Grauwackefelsen

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ABSTRACT: The Hoek-Brown failure criterion provides a practical method to estimate the strength of jointed rock masses. However, based on experience to date, care must be used in applying it to closely-jointed greywacke rock masses in New Zealand. Laboratory testing to determine the rock material constant m_i for greywacke sandstones and mudstones gives lower typical values (12 and 9 respectively) than the value for greywacke assigned by Hoek & Brown (18). High intact greywacke sandstone strengths also lead to unrealistically high predictions of rock mass strength by the failure criterion. An approach to the assessment of rock mass deformability, which uses a modified empirical correlation with Rock Mass Rating (*RMR*) and which takes account of intact rock modulus, is also outlined.

RÉSUMÉ: Le critère de rupture de Hoek-Brown fournit une méthode pratique pour estimer la résistance des masses rocheuses jointes. Cependant, les expériences récentes montrent qu'il faut être prudent quant-á l'application de ce critère aux masses rocheuses á joints rapprochés des grauwackes de Nouvelle-Zélande. Les tests en laboratoire qui permettent de déterminer la constante m_i d'un grauwacke et d'une pélite conduisent á des valeurs typiquement plus faibles (respectivement 12 et 9) que la valeur assignée au grauwacke par Hoek et Brown (18). Les résistances élevées caractérisant un grès grauwacke intact fournissent d'autre part une prédiction de la résistance de la masse rocheuse, déduite du critère de rupture, élevée et non réaliste. On présente enfin une étude de la capacité de déformation de masses rocheuses basée sur une corrélation empirique, modifiée d'après la 'côtation des masses rocheuses' (*RMR*), et qui prend en compte le module élastique caractérisant une roche intacte.

ZUSAMMENFASSUNG: Das Hoek-Brown Bruchkriterium stellt eine praktische Methode zur Einschaetzung der Festigkeit von geklueftеten Gesteinsmassen dar. Aufgrund der bisherigen Erfahrungen muss jedoch bei der Anwendung dieses Verfahrens auf enggeklueftete Grauwackemassen in Neuseeland grosse Vorsicht angewandt werden. Labortests zur Bestimmung der Gesteinsmaterialkonstante m_i fuer Grauwackesandsteine and Tonsteine ergeben tiefere Normalwerte (12 bzw. 9) als den Wert fuer Grauwacken von Hoek & Brown (18). Frischgesteinsfestigkeit von Grauwackesandsteinen fuehrt ausserdem zu unrealistisch hohen Gesteinsmassenfestigkeitsvorhersagen durch das Bruchkriterium. Eine Methode, die Deformationsfaehigkeit von Gesteinsmassen zu bestimmen, die von einer abgeaenderten empirischen Wechselbeziehung mit 'Gesteinsmasseneinschaetzung' (RMR) Gebrauch macht und die den Frischgesteinsmodul in Betracht zieht, wird ebenfalls beschrieben.

1 INTRODUCTION

Upper Paleozoic to Mesozoic-age greywacke rocks are widespread throughout New Zealand (Fig. 1). Much of the nation's infrastructure (e.g. dams, roads) is built in, on, or traverses greywacke terrane, and the rocks are also the source of concrete aggregates and road sealing chip. The rocks are composed of hard sandstones, interbedded sandstones and mudstones, and mudstones. Reflecting their complex geological history the rock masses are typically closely-jointed.

The Hoek-Brown failure criterion, initially introduced in 1980 (Hoek & Brown 1980) and recently reviewed (Hoek & Brown 1997), is often used to determine the engineering behaviour of closely-jointed rock masses.

This paper outlines the first stage of an investigation into the application of the criterion to New Zealand greywacke rocks. To date, it is based on three study sites selected from dam sites and quarries around New Zealand (Fig. 1) where there are good exposures of unweathered greywacke rock masses, together with documentation of engineering behaviour during construction and/or operation.

1.1 *Greywacke*

In New Zealand, the term 'greywacke' has previously been applied to hard grey sandstone rocks, with finer-grained rocks being referred to as 'argillite'. However, in the present, more formal usage of the term, greywacke includes very well indurated to very slightly metamorphosed, interbedded mudstones and muddy

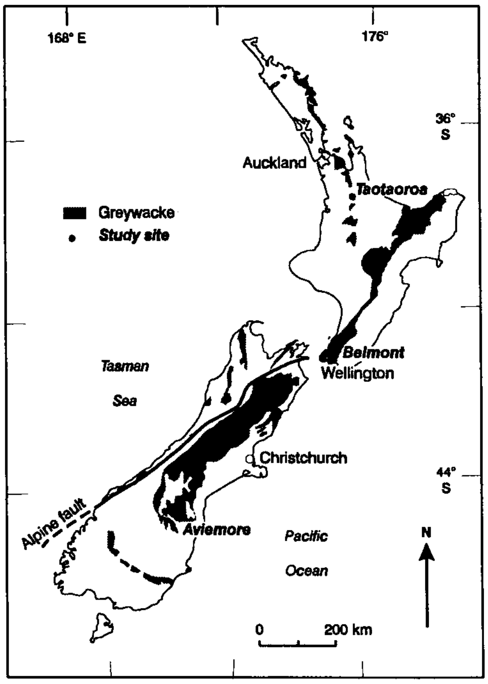

Figure 1. Distribution of New Zealand greywacke rocks (from Suggate et al. 1978) and locations of study sites.

sandstones belonging to the Torlesse Supergroup (Campbell & Coombs 1966), together with similar lithologies in other geological units of comparable age and geological history.

Torlesse greywackes are the most widespread basement lithology in New Zealand, forming the axial ranges of both main islands. Their geological history is complex; the rocks having been intensely deformed and folded. Bedding is often subvertical or steeply dipping, and defects in the rock mass are closely-spaced. Joint spacings greater than 750 mm are very rare; with average spacings being 60 to 200 mm. Defects have even closer spacings (i.e. less than 60 mm) in crushed and sheared zones. These zones may be up to tens of metres wide either in association with major geological faults or where tectonic deformation has been more intense, particularly in finer-grained lithologies.

Where unweathered, intact rock material is strong to extremely strong, with unconfined strengths up to 350 MPa and moderate to high modulus ratios (Fig. 2). Near surface weathering, to depths up to 30 m, may reduce the strength to that of stiff soils. This paper, however, is concerned with unweathered materials.

Using the Rock Mass Rating (*RMR*) scheme (Bieniawski 1976), the unweathered rock masses are commonly classified as fair to very poor, and in the *Q* system (Barton et al. 1974) from poor to extremely poor. Thick sandstone sequences may have a higher rock mass quality.

1.2 *Hoek-Brown failure criterion*

The Hoek-Brown failure criterion has been developed to provide a practical method for estimating the strength and deformability of jointed rock masses (Hoek & Brown 1980, 1997). The generalised failure criterion is defined by:

$$\sigma'_1 = \sigma'_3 + \sigma_{ci}\left(m_b \frac{\sigma'_3}{\sigma_{ci}} + s\right)^a \qquad (1)$$

where

σ'_1 & σ'_3 are the maximum and minimum effective stresses at failure respectively,

m_b is the Hoek-Brown constant m for the rock mass,

s and a are constants which depend on the characteristics of the rock mass, and

σ_{ci} is the uniaxial compressive strength of the intact rock material.

The method is strictly applicable only to intact rock or to closely-jointed rock masses that can be considered homogeneous and isotropic. In order to use the criterion for estimating the strength and deformability of jointed rock masses, the following three properties need to be determined:

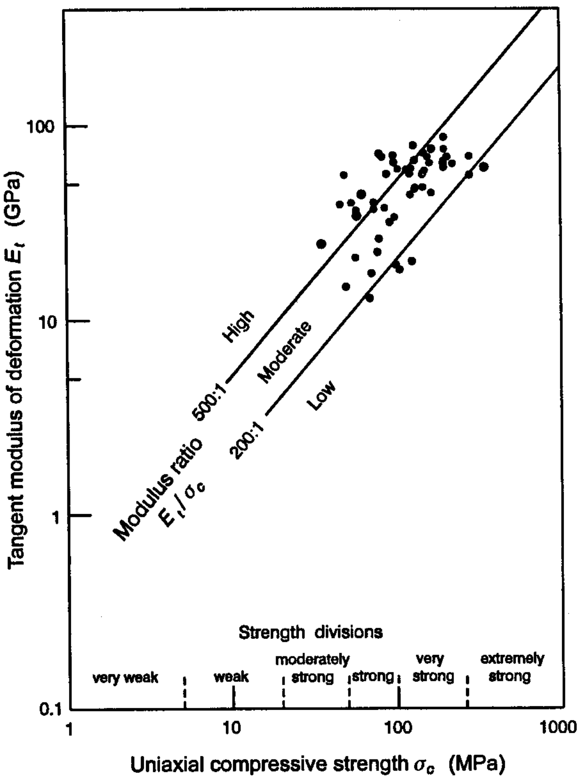

Figure 2. Classification of intact greywacke after Deere & Miller (1966) using New Zealand–wide sources (after Read et al. 1998).

1. the uniaxial compressive strength σ_{ci} of intact rock material,
2. the value of m_i, the Hoek-Brown constant m for intact rock material, and
3. the value of the Geological Strength Index *GSI* (Hoek et al. 1995) of the rock mass.

2 GREYWACKE PROPERTIES

The Hoek-Brown failure criterion has been used for a wider range of problems than originally anticipated and subsequently has undergone several developments (Hoek & Brown, 1988, Hoek et al. 1992, 1995, Hoek & Brown 1997). Use of the criterion in New Zealand for greywacke rocks has had to rely on the *a priori* value of the constant m_i given in Hoek & Brown (1997) due to insufficient determinations of this constant having been

Table 1. Greywacke study site descriptions. (Site locations in Figure 1)

	Geological description		Rock mass classifications[1]		
Site	Lithogy and structure	Defects	*RMR*	Q	*GSI*
Aviemore (dam site)					
55 m high concrete gravity dam with 55 m high battered cut face above	Interbedded sandstones and mudstones, with quartz and calcite veining. Bedding dips steep, mainly to west.	Numerous minor crushed and sheared zones, especially along bedding. Joint spacing 50-150 mm, max 500 mm.	<20–50, often 40	0.002–2 often 1	20–50 often 40
Belmont (aggregate quarry)					
80-110 m high walls with 15-20 m batters and 40° overall slope	Sandstone with mudstone beds 0.2-5 m thick. Bedding dips steeply to south-east. Little secondary mineralisation.	Several minor sheared and crushed zones. Four principal joint sets; spacing 40-200 mm, max 1000 mm	25–55 often 45	0.1-2 often 1	25–55 often 45
Taotaoroa (aggregate quarry)					
80-100 m high walls with 40° overall slope	Sandstone with secondary mineralisation along joints. Several 1-5 m thick mudstone beds. Bedding dips 35°-65°, mainly to west.	Several minor sheared zones and one major crushed zone. Three orthogonal joint sets, plus curving fractures. Spacing 50-250 mm, max 750 mm.	<20–50, often 35	0.02–1 often 0.5	10–40 often 35

Notes: [1] Rock mass classifications (Rock Mass Rating - *RMR*, Q system - Q, Geological Strength Index – *GSI*) from surface geological mapping made. The nature of the greywacke rocks, which are often heterogeneous with variable rock mass qualities, also leads to difficulties in sampling, testing, and the development of appropriate geotechnical models.

Brief descriptions of the three greywacke study sites are given in Table 1. At each site, engineering geological logging of exposed batters has been performed together with sampling for laboratory testing as outlined in Read et al. (1998).

2.1 *Intact rock testing*

Block samples (0.125 to 0.25 m^3 volume) were collected during mapping at each of the study sites. In the laboratory cores with either a 63 mm or 54 mm diameter were drilled from the blocks and prepared for testing.

Unconfined uniaxial compression tests, triaxial compression tests using either an HQ (63 mm) or NX (54 mm) -sized Hoek cell, and Brazilian indirect tensile strength tests were carried out in accordance with methods in Brown (1981) and Read et al. (1987). Loading was performed using a 120 tonne frame, giving a limit of axial stress at 500 MPa for 54 mm diameter test specimens. However, the maximum confining pressure of the Hoek cell (70 MPa) was not often used, as the sandstone strengths at moderate confining pressures were close to the limits of the loading frame capacity.

Figure 3 shows the results of laboratory strength tests on sandstones from Aviemore, Belmont and Taotaoroa, and mudstones from Belmont. There is a fair consistency in strength parameters between individual cores taken from a single block, with some variations between blocks. Mudstones, which are less common, tend to be weaker than the sandstones, and difficulties in obtaining cores also limited the number of mudstone tests possible.

Table 2 summarises the intact rock properties derived from the testing. Hoek & Brown (1997) do not include tensile testing in their procedures, but the table illustrates that similar results are obtained whether including or excluding the tensile values. Table 3 gives an *a priori* value of m_i for greywacke (18), together with values for other sedimentary rock lithologies.

The most notable feature from the testing for this study is the lower m_i value obtained for greywacke sandstone (range from 9 to 15, typical value 12). This may reflect the more typically muddy fine to medium grained textures of New Zealand greywacke sandstones, compared with coarser grained and/or better sorted sandstones elsewhere.

Greywacke mudstone m_i values (range from 7 to 12, typical value 9) are similar to that of siltstone (9) given in Table 3.

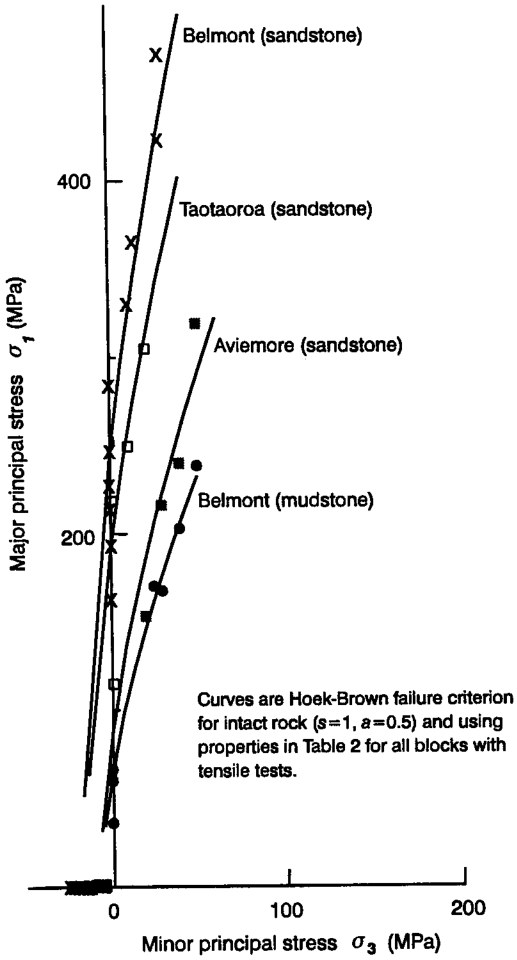

Figure 3. Results of triaxial testing of intact greywacke material from the three study sites.

Table 3. Values of the intact rock material constant, m_i, for clastic sedimentary rock lithologies (from Hoek & Brown, 1997)

Texture			
Coarse	Medium	Fine	Very fine
Conglomerate (22)[1]	Sandstone 19	Siltstone 9	Claystone 4
	---- Greywacke ---- (18)[1]		

Note: [1] Values in parenthesis are *a priori* estimates.

Table 2. Values of intact material constant, m_i, and uniaxial compressive strength, σ_{ci}, derived from triaxial strength testing of greywacke[1].

Sample blocks	Without indirect tensile tests: Number of tests	m_i	σ_{ci} (MPa)	Coefficient of determination	With indirect tensile tests: Number of tests	m_i	σ_{ci} (MPa)	Coefficient of determination
Aviemore dam – sandstone								
Blocks 1 & 3	3[2]	-	-	-	5	14.0	83	0.930
Block 2	2[2]	-	-	-	5	9.1	91	0.997
All blocks	5	24.3	56	0.753	10	11.4	87	0.914
Belmont quarry – sandstone								
Block 4	7	13.3	250	0.905	8	11.4	258	0.932
Block 5	4	14.9	201	0.614	11	12.4	214	0.792
All blocks	11	12.5	241	0.756	19	13.5	240	0.880
Belmont quarry – mudstone								
Blocks 1 & 2	4	11.9	50	0.992	9	8.7	62	0.974
Blocks 3 & 8	5	7.2	91	0.650	7	8.1	85	0.766
All blocks	9	8.4	75	0.796	16	9.0	65	0.920
Taotaoroa quarry – sandstone								
Block 1	5	9.5	202	0.643	7	11.7	193	0.810

Notes: [1] Properties determined using method in Hoek & Brown (1980,1997). [2] Constants not determined where number of tests less than four.

2.2 *Rock mass strength*

The Geological Strength Index (*GSI*), which was introduced by Hoek et al. (1995), is determined from the appearance of the rock mass and is based on the structure and surface condition of defects. Values range from 5 for sheared and crushed rock masses to 100 for intact rock material (see Table 7 in Hoek et al. 1998).

Determination of the failure criterion depends on whether *GSI* is greater than 25 (reasonable to good rock mass quality) or less than 25 (very poor rock mass quality) - Hoek & Brown (1997). *GSI* values for New Zealand greywacke rock masses typically vary from 5 to about 55. The examples illustrated in this paper are for rock masses where *GSI* >25. For this situation input parameters are defined as:

$$m_b = m_i \exp\left(\frac{GSI-100}{28}\right) \quad (2)$$

$$s = \exp\left(\frac{GSI-100}{9}\right) \quad (3)$$

$$a = 0.5 \quad (4)$$

Rock mass strengths predicted by the failure criterion for Belmont sandstones with GSI = 45 are illustrated in Figure 4. Mohr-Coulomb parameters (friction angle ϕ and cohesion c) may calculated from the rock mass failure criterion using the approach outlined in Hoek & Brown (1997). For a uniaxial compressive strength (σ_{ci}) of 240 MPa, Table 4 gives Mohr-Coulmb parameters for both the *a priori* (18 – Table 3) and determined (13.5 – Table 2) values of m_i. The friction angle (ϕ) for the determined, and therefore more acceptable, value of m_i is about 2° less than that indicated by the *a priori* value.

In both the cases shown in Table 4, the calculated rock mass strengths are much higher than expected. Evidence from stable and failed slopes at and near the study sites as well as in similar rock masses elsewhere (to be discussed in a subsequent paper) indicates that the actual rock mass strengths are substantially lower.

Table 4. Mohr-Coulomb strengths from the Hoek-Brown failure criterion for Belmont sandstone with different rock material constant m_i values.

GSI = 45, σ_{ci} = 240 MPa	Minor principal stress σ_3 (MPa) $0 < \sigma_3 < 0.25\sigma_{ci}$		Normal stress σ_n (MPa) $\sigma_n = 1$	
Rock material constant m_i	c (MPa)	ϕ (°)	c (MPa)	ϕ (°)
18	10.6	35	1.4	64
13.5	9.9	33	1.5	62

Figure 4 includes strengths predicted by the failure criterion for a range of uniaxial compressive strengths (240 to 60 MPa). The calculated Mohr-Coulomb parameters, which are summarised in Table 5, show appreciable differences over the range of intact rock material strengths (e.g. 10° in ϕ for σ_n = 1 MPa). Based on field observations and experience, the rock mass failure strengths are considered more realistic when uniaxial compressive strengths closer to 60 MPa are used with the criterion rather than the determined value of 240 MPa.

Table 6 shows the effects of varying *GSI* values on Mohr-Coulomb strengths, and the failure criteria are illustrated in Figure 5. Based on experience a value of *GSI* as low as 25 is judged necessary to produce a credible rock mass strength. However, the engineering geological mapping at Belmont indicates that the sandstone rock mass is characterised by many intersecting defects (*blocky/disturbed)* and smooth surfaces (*fair)* for which a *GSI* value of 45 is more appropriate.

Table 5. Mohr-Coulomb strengths from Hoek-Brown failure criteria for Belmont sandstone with varying uniaxial compressive strength values.

GSI = 45, m_i = 13.5	Minor principal stress σ_3 (MPa) $0 < \sigma_3 < 0.25\sigma_{ci}$		Normal stress σ_n (MPa) $\sigma_n = 1$	
Uniaxial compressive strength σ_{ci} (MPa)	c (MPa)	ϕ (°)	c (MPa)	ϕ (°)
240	9.9	33	1.5	62
120	4.9	33	1.0	57
60	2.5	33	0.7	52

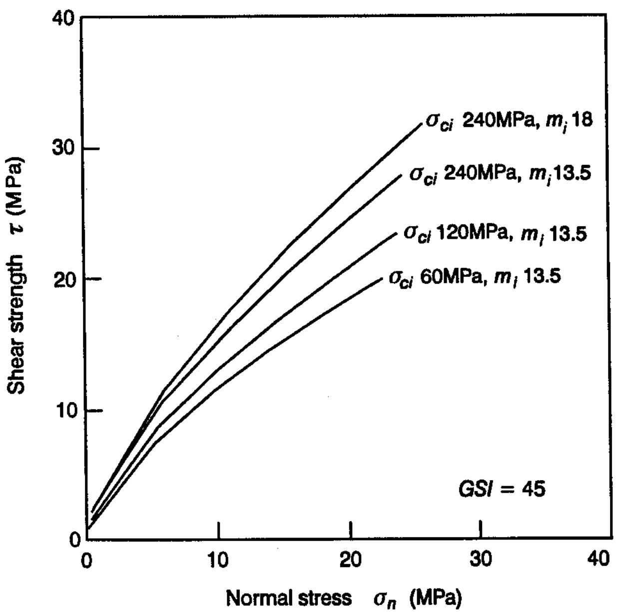

Figure 4. Rock mass failure criteria for Belmont sandstone with varying uniaxial compressive strength (σ_{ci}) and intact rock material constant m_i.

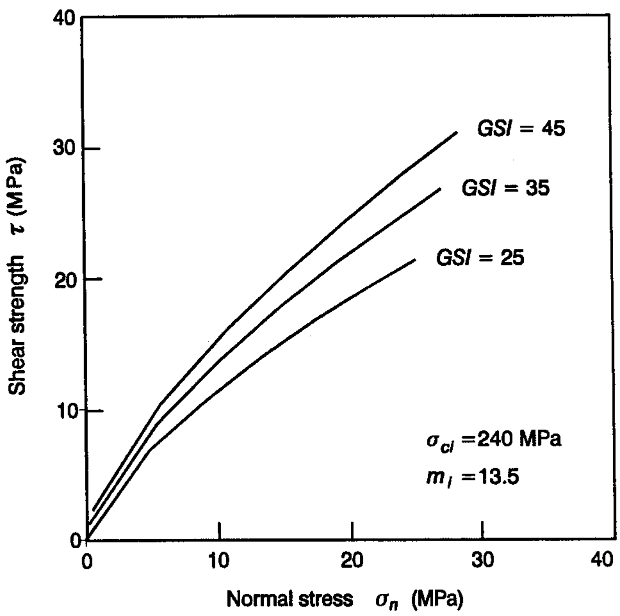

Figure 5. Rock mass failure criteria for Belmont sandstone with varying Geological Strength Index (*GSI*).

Table 6. Mohr-Coulomb strengths from Hoek-Brown failure criterion for Belmont sandstone with varying *GSI* values.

$m_i = 13.5$ $\sigma_{ci} = 240$ MPa	Minor principal stress σ_3 (MPa) $0 < \sigma_3 < 0.25\sigma_{ci}$		Normal stress σ_n (MPa) $\sigma_n = 1$	
Geological Strength Index (*GSI*)	*c* (MPa)	ϕ (°)	*c* (MPa)	ϕ (°)
45	9.9	33	1.5	62
35	8.3	30	1.0	60
25	5.9	27	0.4	53

It is therefore apparent that for rock masses with combinations of very strong intact material and closely-spaced jointing, such as the sandstone at Belmont, the direct use of the measured uniaxial compressive strength leads to unrealistically high predictions of the Hoek-Brown failure criterion. For these rock masses, the surface quality of the defects appears to have a more dominant influence on the rock mass strength.

The greywacke mudstones, with lower intact material strengths, do not appear to have the above problem to the same extent and the Hoek-Brown criterion may be more directly applicable.

The ongoing research programme is further investigating the relative influences of intact rock material and rock mass parameters on greywacke rock mass strength. This includes lower quality greywacke rock masses (i.e. *GSI* <25), for which very low uniaxial strengths have been used outside of New Zealand for the failure criterion (e.g. Hoek et al. 1998).

2.3 *Rock mass deformability*

Serafim & Pereira (1983) empirically correlated rock mass deformability (E_m) with Rock Mass Rating (*RMR*):

$$E_m = 10^{\frac{RMR-10}{40}} \quad (5)$$

Hoek & Brown (1997), while pointing out that *GSI* values > 25 are similar to *RMR* values, modified Equation (5) for rocks with uniaxial compressive strengths of less than 100 MPa:

$$E_m = \sqrt{\frac{\sigma_{ci}}{100}}\,10^{\frac{GSI-10}{40}} \quad (6)$$

Neither equation takes the stiffness of the intact rock material into account. In Figure 6, Equations (5) and (6) have been used to predict values of the rock mass deformability (E_m) at *RMR* = 100 (i.e. intact rock) using the New Zealand data in Figure 2. Predicted values are about three times those of the intact rock material with Equation (5), giving a value of 177 GPa at *RMR* = 100.

Figure 7 shows Equation (5) in relation to the data set used by Serafim & Pereira (1983). However, as the rock mass deformability should not exceed the modulus of the intact material, any empirical method of estimating the deformability should take the intact rock material properties into account. The authors consider that the following equation may be appropriate for correlating rock mass deformability (E_m) and Rock Mass rating (*RMR*):

$$E_m = 0.1\left(\frac{RMR}{10}\right)^3 \quad (7)$$

This equation has a more realistic rock mass deformability value of E_m = 100 GPa at *RMR* = 100. As a check against the intact rock properties, any predicted rock mass deformability value from this equation (E_{calc}) should be normalised by multiplying by the ratio of the intact rock modulus (E_i) to the value of deformability at *RMR* = 100 (E_{100}) as follows:

$$E_m = E_{calc}\frac{E_i}{E_{100}} \quad (8)$$

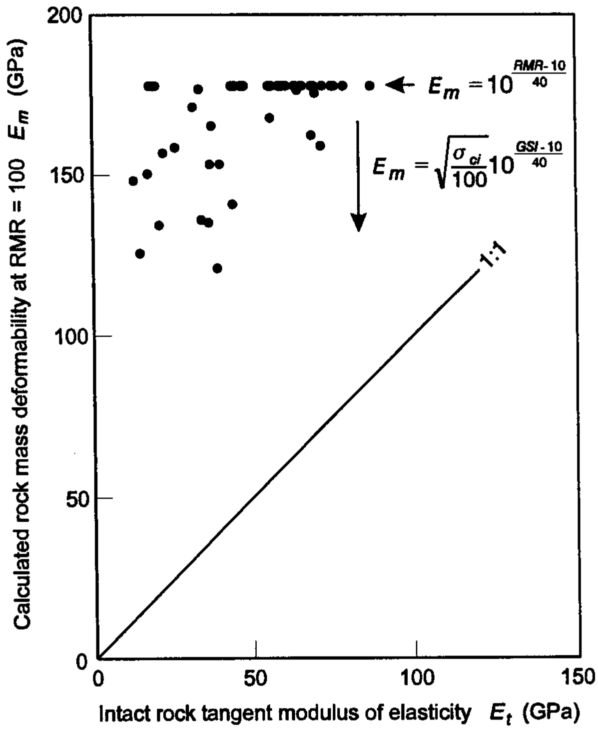

Figure 6. Rock mass deformability for Rock Mass Rating (*RMR*) = 100 estimated from New Zealand data in Figure 2.

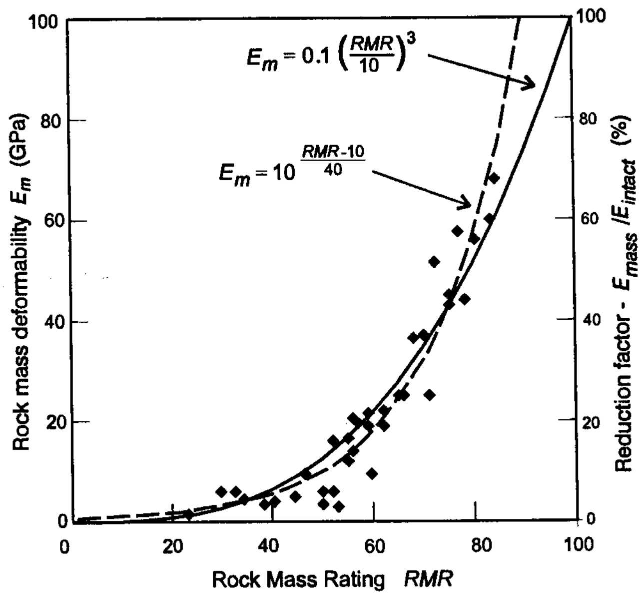

Figure 7. Prediction of rock mass deformability using Rock Mass Rating (*RMR*) or intact rock material modulus (E_i).

The method has the potential to be used either to directly predict the rock mass deformability or to indicate the reduction factor that should be applied to the intact modulus. For example in Figure 7 at *RMR* = 60, either the rock mass deformability is 21.6 GPa or a reduction factor of 21.6% should be applied to the modulus value of the intact rock.

3 CONCLUSIONS

The Hoek-Brown failure criterion has been found to give misleading results for the rock mass strength of New Zealand greywackes, particularly sandstone. Consequently care and judgement must be used in its application.

Laboratory strength testing has determined a typical value of the rock material constant m_i for greywacke sandstone (12) that is lower than the *a priori* value (18) assigned by Hoek & Brown (1997). Determined values for greywacke mudstone accord closely with the Hoek-Brown values for siltstone (9).

The very high strengths of intact greywacke sandstones lead to unrealistically high estimates of the rock mass strength if these are used directly in the Hoek-Brown failure criterion. In these situations, the surface quality of the defects appears to have a relatively more dominant influence on the rock mass strength than the intact material strength. Consequently calibration determined rock mass strength against observed behaviour of the rock mass in the field is considered an integral part in the assessment of the failure criterion.

An alternative approach has been proposed to estimate rock mass deformability (E_m) using a modified empirical correlation with the Rock Mass Rating (*RMR*). It also includes a check between the rock mass deformability at *RMR* = 100 and the modulus of elasticity of the intact rock material.

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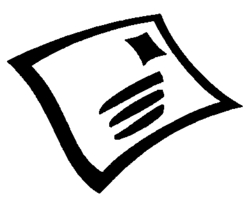

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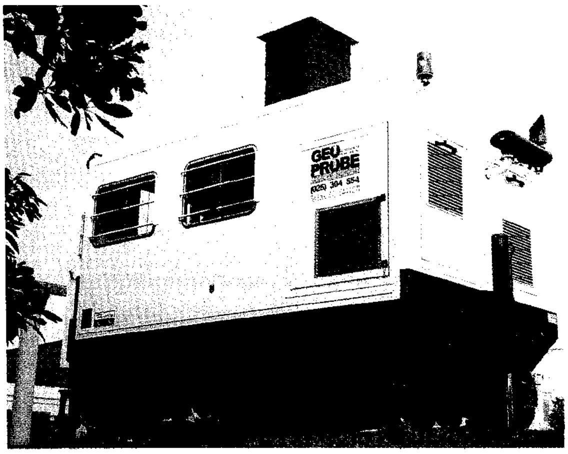

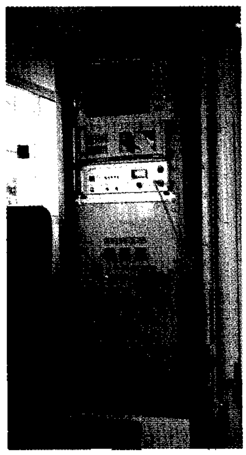

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Relation Between Settlement & Cone Penetration Resistance at Two Sites

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Introduction

This paper gives a brief account of work at two sites attempting to correlate observed settlements with CPT soundings. The idea that soil stiffness, for the prediction of settlement, is related to CPT resistance goes back some time; an important contribution came from Schmertmann (1972) who proposed that the soil modulus at a given point in a sand profile is twice the cone penetration resistance. In this paper data from monitoring the settlement of two embankments, one a preload and the other permanent, are analysed. The soil at the preloaded site was of volcanic origin, at the other mainly recent sediments were encountered.

Senneset et al (1989) suggested the following relation between the net cone penetration resistance and the constrained modulus for the soil:

$$M = \alpha(q_c - \sigma_{vo})$$

where: M is the constrained modulus for the soil (ie modulus for one dimensional compression such as would be measured in the oedometer),
q_c is the cone penetration resistance,
σ_{vo} is the total vertical stress at the depth corresponding the q_c,
α is a dimensionless coefficient.

Settlement estimation commonly uses the constrained modulus even in situations where the deformation is not one-dimensional. In this way only the vertical stress increments due to the surface loading have to be estimated, and, even if the deformation is not truly one dimensional, the constrained modulus allows approximately for the stiffening effect of the lateral stress increments which accompany the vertical stress increments.

Sennesset et al suggest that M falls in the range 5 to 15 for overconsolidated clays and 4 to 8 for normally consolidated clays. Having surveyed a number of case studies Kulhawy and Mayne (1990) propose an "average" value of 8.25 for α. Figure 1 gives the data they used.

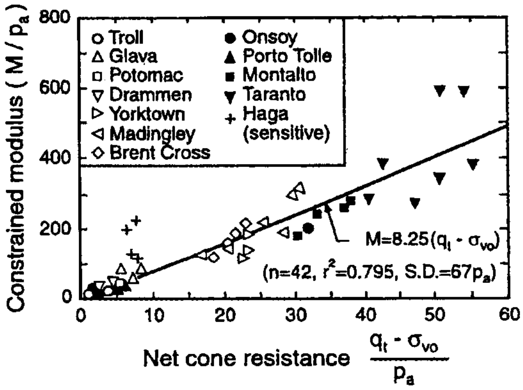

Figure 1. Kulhawy and Mayne data for $\alpha \approx 8.25$

Site and Loading Description

The unit weight of the filling for both sites was assumed to be 17 kN/m^3.

(a) Tank Preload Site

The first case is a preloading for a water storage reservoir at a site consisting of volcanic soil in the Tauranga region. The layout of the preload is shown in Figure 2 and a typical CPT record is given in Figure 4. Some data about the volcanic soils at the site are given by Pender et al (1998). The original ground surface contours are shown in the figure as are the locations of the five CPT soundings and the locations of the five positions at which the settlements were monitored. The upper surface of the preload was at elevation 83.5 m, so that the maximum depth of fill was about 3 m. As this loading was complex, both in plan and in variation of fill thickness across the preload, it was represented as a series of rectangular constant pressure loads.

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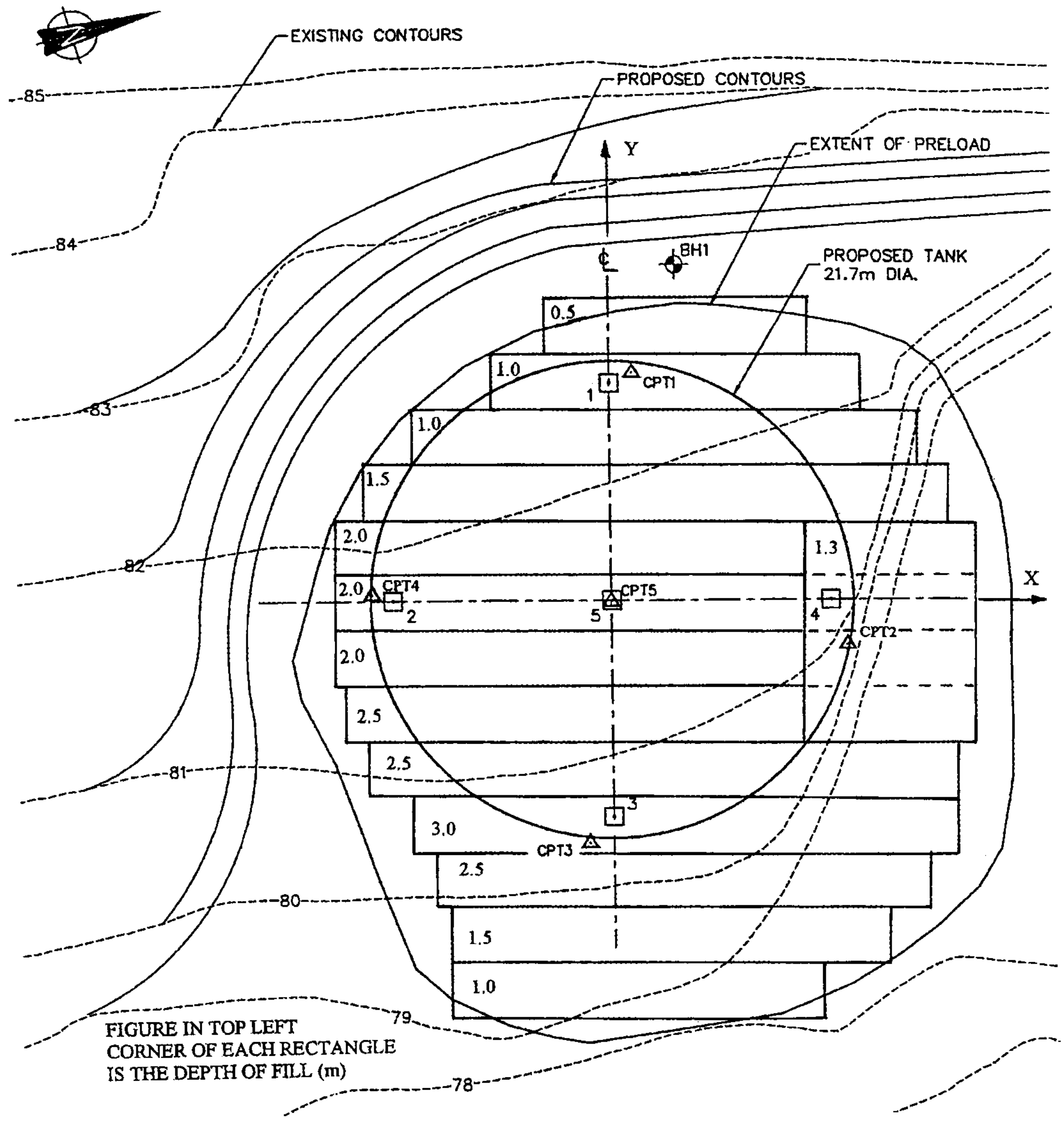

Figure 2. Details of the tank preload and the constant pressure rectangles representing the loading.

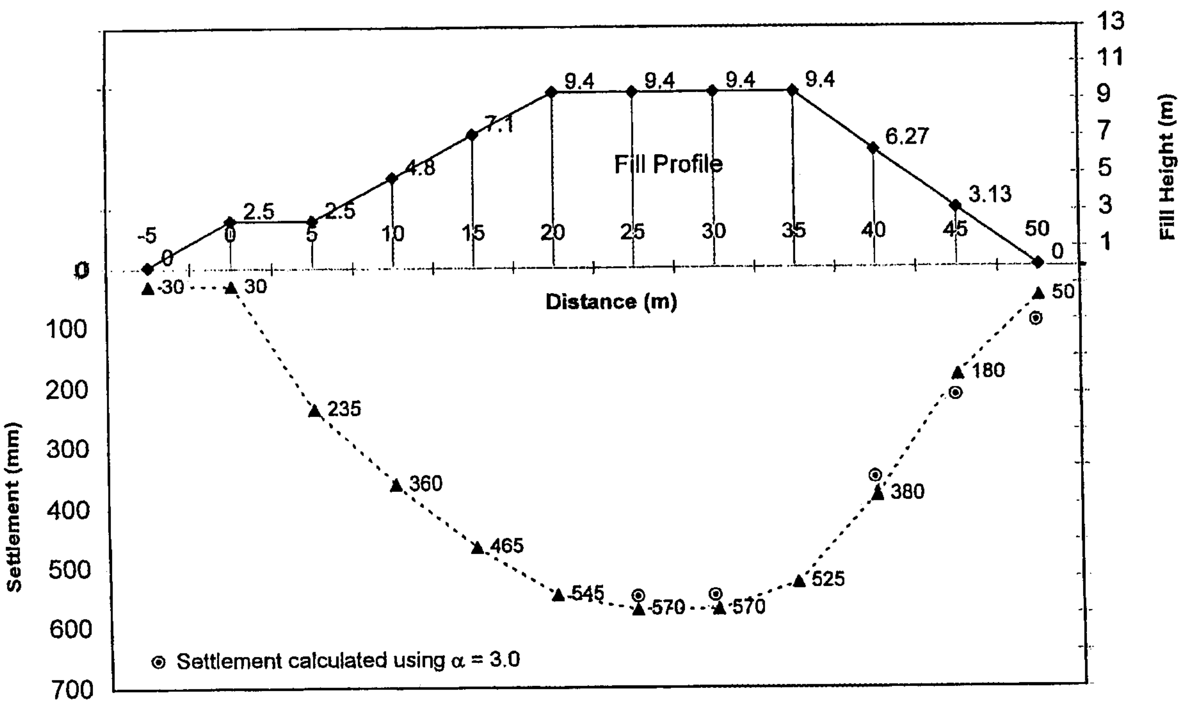

Figure 3. Typical profile of a SH20 embankment with measured and calculated settlements.

The layout of the rectangular loads and the associated fill thickness are shown in Figure 2. The decision about the number, dimension, and position of the rectangular areas was, of course, arbitrary, but attempted to give a reasonable approximation to the fill geometry.

(b) SH20 Motorway Embankment

The embankments were constructed as part of the State Highway 20 Mangere Extension project in South Auckland. Further details are given by Crawford (1998). The fill profile for a typical embankment along with the measured settlement is shown in Figure 3. The CPT record in Figure 4 shows a complex layered profile consisting of a mantle of weathered volcanic ash followed by about 30 m of sub-horizontally layered silty sand, silty clay, and clayey peat, below which was a very dense cemented sand. Continuous settlement profiles were obtained by pulling a very sensitive pressure transducer through a water-filled tube, which had been placed between benchmarks before the filling commenced (see Crawford (1998) for details).

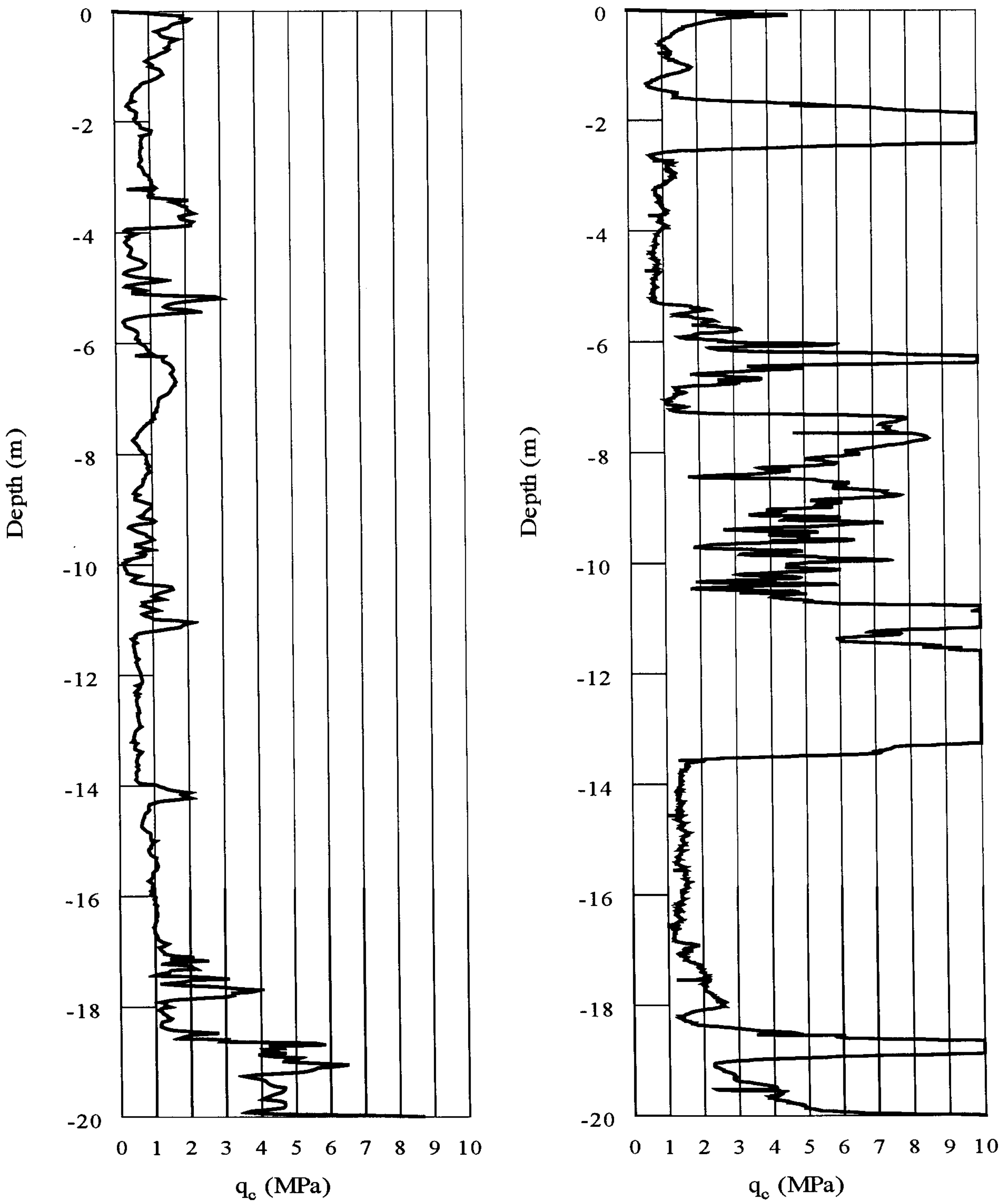

Figure 4. CPT records – left CPT5 from the tank preload, right CP162 from SH20.

Procedure

The vertical settlement at a point on the ground surface is obtained by integrating the vertical strains beneath the point; assuming that at some depth the vertical strains are zero. The soil profile is divided into layers, the vertical strain for each evaluated, and then the surface settlement calculated as the summation of the layer settlements:

$$(\varepsilon_v)_i = \frac{(\Delta\sigma_v)_i}{M_i} \quad \text{settlement for N layers:} \quad S = \sum_{i=1}^{N} (\varepsilon_v)_i \Delta z_i$$

where: i is the layer identifier, ε_v is the vertical strain, Δz the layer thickness, and S the settlement at the surface.

Now the CPT records used for this work are available in electronic form with readings taken every 20 to 40 mm (depending on the particular penetration rig used). For the settlement calculations the layer depth was set to the CPT recording interval, thus a large number of thin layers were used, so avoiding the need to assign "average" values to parts of the CPT record. The calculations were done using the programming facilities in Mathcad. The vertical stress increments were obtained assuming a homogeneous isotropic elastic material and using formulae given by Poulos and Davis (1974). It needs to be remembered that the vertical stress increment beneath a given point is derived not only from the pressure directly above but also from the pressures at adjacent positions; this effect is most significant at depth. The increase in vertical stress generated by the surface loading directly beneath the corner of a rectangular loaded area is given by (equation 3.18a in Poulos and Davis):

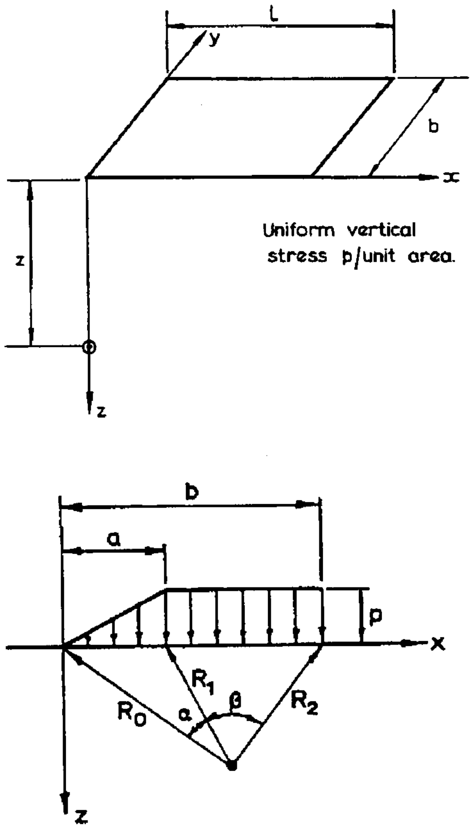

Figure 5. Geometry for vertical stress increment calculation. Upper - rectangular load, lower - embankment loading.

$$\sigma_z = \frac{p}{2\pi}\left[\tan^{-1}\frac{lb}{zR_3} + \frac{lbz}{R_3}\left(\frac{1}{R_1^2} + \frac{1}{R_2^2}\right)\right]$$

where: p, b, l, & z are defined in Figure 5. $R_1 = (l^2 + z^2)^{1/2}$, $R_2 = (b^2 + z^2)^{1/2}$, and $R_3 = (l^2 + b^2 + z^2)^{1/2}$

The vertical stress at a point not beneath the corner of the rectangle is obtained by using the principle of superposition and calculating the stress increments generated by four rectangles, some of which may need to apply a negative pressure, the corners of which are above the point of interest.

In the case of the embankments on the SH20 project we have a two dimensional strip loading. The vertical stress arising from a half embankment is given by (equation 3.9a in Poulos and Davis):

$$\sigma_z = \frac{p}{\pi}\left[\beta + \frac{x\alpha}{a} - \frac{z}{R_2^2}(x-b)\right]$$

where: the various parameters in the equation are defined in Figure 5. The above equation gives the stresses from half the width of the embankment, superposition gives the stresses generated by the other half. More complex loadings can be handled by further superposition.

Results

(a) Tank Preload

Table 1 gives the measured settlements at the five measurement points on the preload, along with the calculated results for two values of α. The results are also plotted in the upper and middle parts of Figure 6. These show

Table 1: Settlement data for the reservoir preload

Settlement point	Measured settlement (mm)	Settlement (mm) $\alpha = 12.5$	Settlement (mm) $\alpha = 25$
1	8	24	12
2	29	46	23
3	91	98	49
4	73	70	35
5	34	77	39

that, although the measured and calculated settlements are not matched with a simple linear relation, the smaller settlements can be predicted with α at 25 and the larger settlements with α at 12.5. In the lower part of Figure 6 the values of α which give a match between the calculated and observed settlements are plotted, from this it is apparent that as the intensity of the surface loading increases there is a nonlinear increase in the settlement. It is tempting to suggest this is to be expected from nonlinear behaviour of the soil but, as the loads applied are small in relation to the expected bearing capacity of the volcanic soil, this may not be correct.

It is apparent from the upper part of Figure 6 that $\alpha = 12.5$ gives a conservative estimate of the settlements. It is notable that such a value for α is at the upper end of the range of values quoted from Senneset et al earlier.

(b) SH20 Embankments

The results of the calculations for one of the several embankment sections monitored along SH20 are shown in Figure 3. The particular section plotted, at settlement measurement line A, was near CPT 162. It appears that $\alpha \approx 3$ models reasonably the observed settlement profile. De Silva (1999) investigated the settlement measured along other monitoring lines and used other CPTs; he found that the value of α required to match the observed settlements ranged between 2.5 and 3.5.

Discussion

The two sites have very different soils and give quite different values of α. Although this is not surprising, one needs to note the SH20 embankments apply much greater pressures to the underlying soils than the reservoir preload, so the smaller α for SH20 could be a consequence of more severe deformation of the soil. Furthermore, there is no small settlement data for the SH20 embankments, all that is available corresponds to pressure loadings beyond those applied by the reservoir preload. On the other hand comparison of the CPT records in Figure 4 gives the misleading impression that the SH20 soil profile is much stiffer than that at the tank preload site.

A careful look at the Kulhawy and Mayne data in Figure 1 is instructive. The settlements at both the SH20 and reservoir preload sites are dominated by CPT values up to about 2 MPa. In this region of Figure 1 there is considerable scatter and the majority of the data points are bracketed by lines with $\alpha \approx 3$ and $\alpha \approx 12.5$.

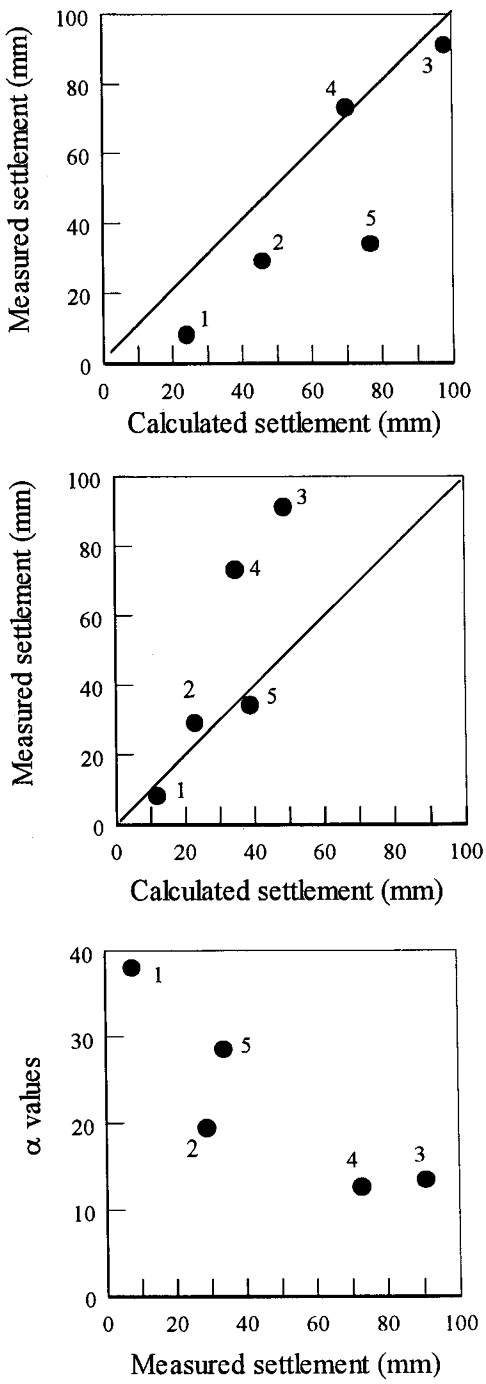

Figure 6. Comparisons of measured and calculated settlements at the tank preload site. Upper $\alpha = 12.5$, middle $\alpha = 25.0$, lower α values required to match the measured settlements.

A number of assumptions have to be made in performing these settlement calculations; these are:

- the elastic vertical stress distribution is valid despite the soil layering,
- the intensity of the pressure loading on the natural ground follows the profile of the fill,
- the settlement is the result of one-dimensional compression of the underlying soil,
- the settlement depends only on vertical stress increments and is independent of the in situ stresses in the soil,
- the CPT test, which is undrained, gives the drained modulus required for settlement estimation,
- all significant settlement occurs within the depth of the penetration records,
- the measured settlements were made after all consolidation settlement had occurred.

Conclusions

The following conclusions can be drawn from these two case studies:

- It appears that there is no constant value of α from which settlement can be predicted. At the site with the volcanic soil, where settlements were obtained for a range of fill thicknesses, it appears that α decreases as the intensity of loading increases; although $\alpha \approx 12$ gives a conservative indication of the measured settlements.

- It is also apparent that the volcanic soil is stiffer than the sedimentary soils along the SH20 embankments, despite the opposite impression gained from comparison of the CPT records in Figure 4. A simple pro-rating of the SH20 settlement based on the ratio of fill heights (ie ≈3m / 10m) gives a predicted settlement of about 170 mm for the tank preload, which is nearly double the maximum settlement measured. This apparent difference in stiffness might be a reflection, though, of the fact that the motorway embankments applied much larger pressures to the underlying soil.

- Once a CPT record is available in electronic form it is convenient to take very thin layers rather than use some averaging process to simplify the profile to a small number of layers.

- Further work relating settlement observations to CPT soundings would supplement the data on α presented in this paper. The author would be pleased to hear from anyone with existing information or possible projects in planning.

Acknowledgements

Dave Jennings and Phil Woodmansey of the Hamilton office of Opus International Consultants Ltd provided the data for the reservoir site. Transit NZ granted access to the data for the SH20 embankments and Steve Crawford, of Tonkin & Taylor Ltd., provided helpful information and insight, whilst Ranjith De Silva processed the settlement and CPT records. Part of this work was funded by the New Zealand Foundation for Research Science and Technology (contracts UOA 604 and UOA 804).

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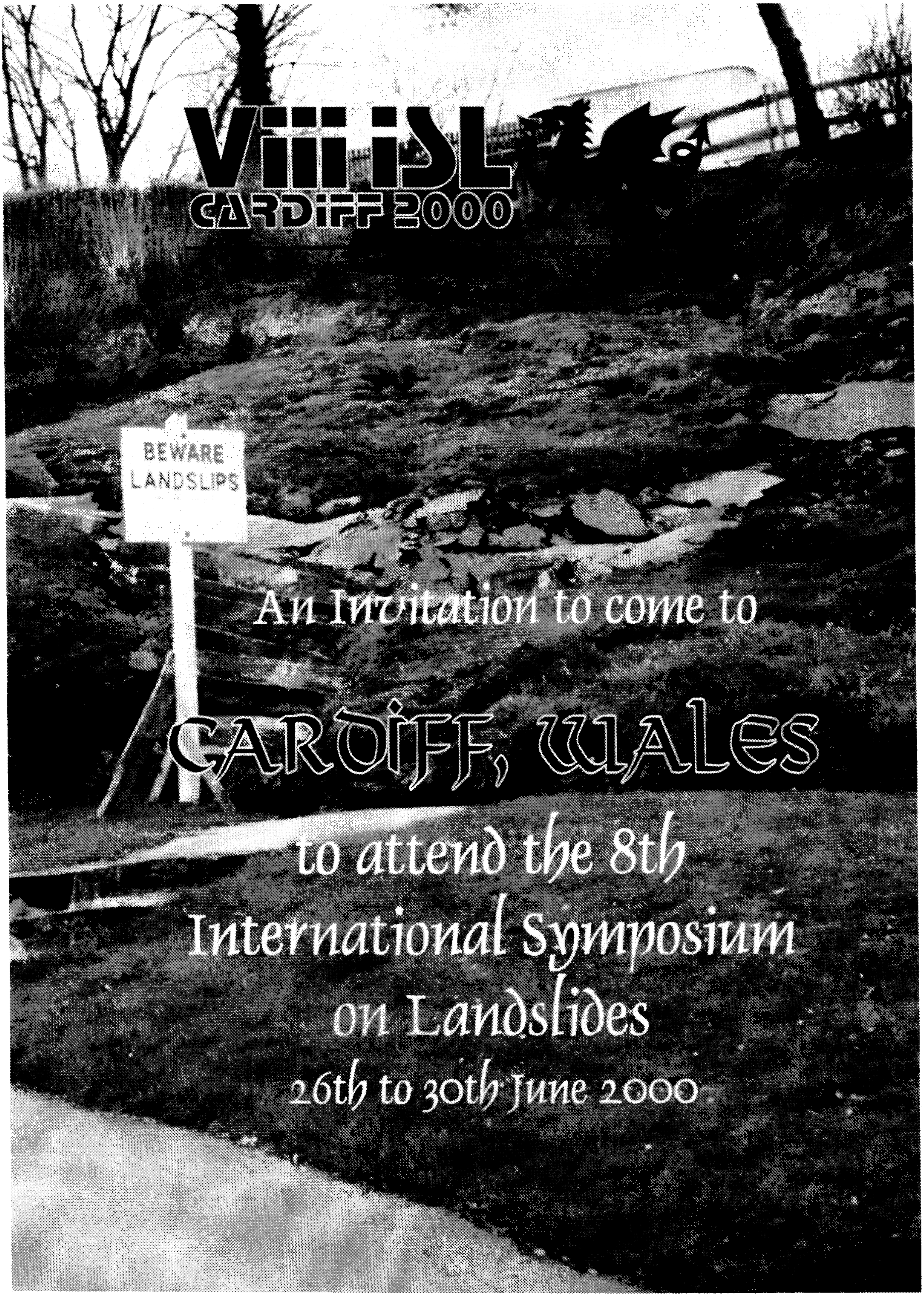
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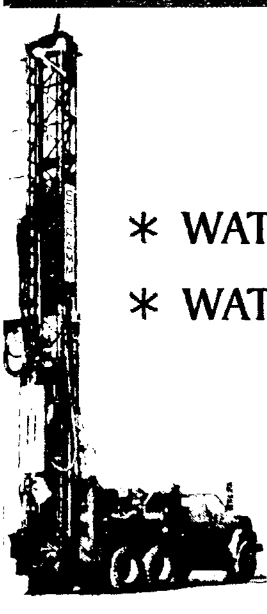

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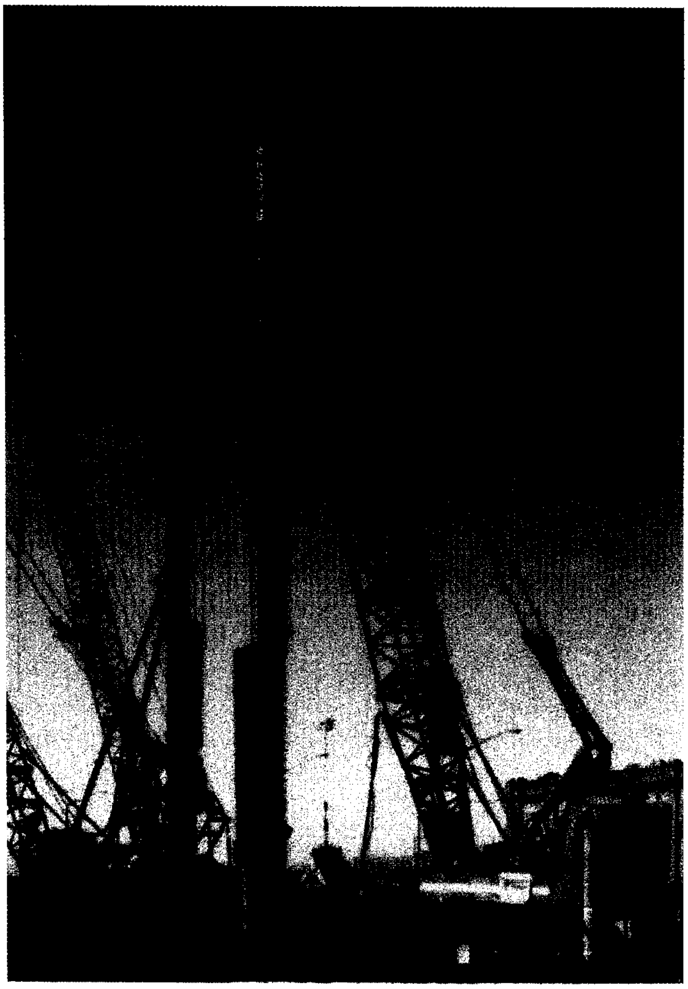

Installation of 700 vibro set, driven cast insitu piles on the Westpac Trust Stadium, Wellington, May 1998.

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BRIAN PERRY

PDA – Dynamic Testing of Driven & Cast Insitu Piles

Maurice Fraser
Senior Geotechnical Engineer, Tonkin & Taylor Ltd

1.0 INTRODUCTION

In simple terms, pile driving involves hammering a large mass onto the end of a pile to drive it into the ground. Following installation, the pile may be straight, bent or broken. It may have a high capacity or less than required and since most of the pile is buried underground, quality of the finished product is uncertain.

Two aspects are of concern in a completed pile; its ability to perform as a structural element within a foundation system and the load-deformation relationship between the pile and surrounding soils.

The traditional New Zealand design approach has been to rely on driving criteria established via the Hiley formula, together with numerous 'rules of thumb' and industry experience to assess a piles geotechnical capacity[1]. In terms of structural integrity, limits are placed on "overdriving a pile" based on observations of driving equipment performance, soil response (temporary compression or rebound) and damage at the pile top. When soil conditions or pile responses are uncertain, large factors of safety are used to ensure that the pile will perform as intended, although the degree of conservatism is generally unknown.

The above is somewhat of a simplification of current design practice but there is considerable reliance on achieving a "set" during driving that often fails to account for the complexity of soil and pile responses during driving. To our general credit, driven pile foundation failures in New Zealand are relatively rare, reflecting perhaps conservative design, unaccounted for pile setup and experienced piling contractors.

The current revision of the NZ Building Code has removed the Hiley formula as an acceptable means of demonstrating foundation compliance. While not wanting to enter the debate on why the Hiley formula is no longer seen as reliable, (which generally it isn't without large factors of safety), the current situation leaves NZ Design Engineers with limited options to demonstrate foundation capacity. Either the current situation of setting conservative sets and subsequent overdriving can continue, or an alternative method of Pile Dynamic Assessment ('PDA') using stress wave theory can be used.

This technical paper sets out the theory behind stress wave analysis, describes several projects undertaken across the country over the past 12 months and outlines many of the significant cost and construction advantages that PDA testing and associated software analysis offers to Contractors and Design Engineers. Aspects of AS 2159:1995, the Australian Piling Code are also discussed.

2.0 BACKGROUND

2.1 Pile Driving

During installation, pile driving simulates a static load test- a load is applied (hammer mass * drop height) and displacements occur. Soil resistance to driving has both static (set) and dynamic (temporary compression, rebound or quake) components. Pile set relates to the actual work done on the pile – energy required to drive the pile some distance into the ground. Soil rebounds are primarily a function of soil type, and to a lesser extent, the driving system. Typical causes for high rebound include large pore pressures generated by driving into fine-grained soils or quasi-elastic behaviour of soil materials at the base of the pile. Pile elastic response can also markedly affect observed rebound values, especially where stiff piles are driven into a low skin friction profile leading to high rebounds where hard end bearing is encountered.

2.2 Wave Equation Theory

The PDA and associated software analysis is based on wave equation theory to assess and model pile behaviour. Each time a hammer hits the top of a pile, the pile experiences a stress wave generated by the blow as well as being accelerated into the ground.

As the stress wave travels the length of the pile, small compressive reflections are generated along the pile length due to resistance from skin friction. When the stress wave reaches the pile base, additional reflections are generated due to end bearing. Compressive reflections occur with substantial end resistance while tension reflections occur where end resistance is minimal. Compressive 'resistance reflections' cause the force at the top of the pile to increase and the pile

[1] This paper defines 'required geotechnical capacity" as being a piles ultimate structural load increased by a factor to account for geotechnical uncertainties surrounding the driving process, soil capacity and other geological / installation factors.

top to slow down compared with the pile base while tension reflections cause the opposite effects to occur.

Forces and accelerations within the pile are measured by pairs of bolted on strain gauges and accelerometers. Measurements are made on the opposite sides of the pile to account for eccentric impacts and bending, with results being averaged. Gauges are installed at least one, and preferably two pile diameters below the pile top to minimise pile end effects during driving. Data from the gauges are collected and processed by a field portable computer referred to as a pile driving analyser (PDA).

The PDA software converts each stress wave to a force record by dividing stress with the pile impedance[2] and calculates pile velocities and displacements via integration of the acceleration record. The software processes wave reflections due to skin and end resistance from the initial downward travelling force and velocity records. Taking into account the difference between the force and acceleration records, measured pile rebounds and estimates of soil damping response, pile static capacity estimates are made.

As well as static capacity estimates, the field equipment is able to:

- Measure the driving energy impacting the head of the pile, allowing calculations of hammer and driving system efficiency;

- Monitor compressive and tensile stresses within the pile as a result of each blow during driving, minimising the risk of causing pile damage; and,

- Obtain indications of the extent and depth to pile damage through detecting changes in pile impedance due to loss of section area.

PDA testing can be carried out on steel, precast concrete and timber piles, as well as composite piles such as concrete filled driven steel tubes and obviously jointed piles. Drilled, cast insitu concrete piles can also be tested. Modifying the driving system to impact the pile above the gauges can assess the capacity of bottom driven steel tubes. Alternative software is available to analyse bottom driven blows where this is not possible, although accuracy is more limited compared with top driven blows.

For static capacity assessment, piles have to be driven some distance to fully mobilise available skin friction and end bearing capacity. Low sets (typically less than 5mm) may not fully mobilise all available capacity. Where low sets occur due to insufficient driving energy the PDA will indicate what capacity is mobilised but more capacity may be available if greater driving energy (larger set) is available. Typically, piles need to undergo permanent set per blow of 4 to 6 mm to fully mobilise skin friction and in the order of 10% of base diameter to fully mobilise end resistance.

Wave theory also allows predictions of hammer / pile / soil performance to be assessed prior to construction. A program called GRLWEAP ('Wave Equation Analysis of Pile driving') allows relatively detailed modelling of the driving system, pile and soil response to be undertaken. The program allows for optimisation of hammer weight and cushioning details, selection of pile type and estimates of pile penetration vs axial load capacity for a given range of sets to be calculated. Over 600 computer models of driving hammers are currently available, including the majority of diesel, drop and hydraulic hammers available in NZ.

The PDA has onboard software to provide estimates of pile capacity directly in the field (CASE – RMX methods). These methods are based on various damping factors related to soil type and are generally taken as being +/- 20% accurate compared with static load tests at the same displacement. Figure one is taken from the PDA manual[3] and illustrates the close agreement between the CASE analysis and actual static load tests.

Where warranted, further processing using CAPWAP software is undertaken.

2.3 CAPWAP

CAPWAP (CAse Pile Wave Analysis Program) Software is a sophisticated software package that models a driven pile and resulting soil response. CAPWAP combines measured force and velocity data with wave equation analysis procedures to calculate the initially unknown soil resistance forces acting on a pile. The program models the pile as a series of uniform continuous pile segments, with each segment being assigned different soil parameters (skin friction, soil damping, soil rebound) based on the geological model.

The soil model is based on site geotechnical data and takes into account total pile resistance and distribution, soil damping constants, skin friction, pile base loading and unloading factors, plus soil rebounds. Additional modelling for "toe gaps" (eg where a pile is sitting slightly above hard end bearing due to 'bouncing' from a previous blow), or "pile plugs" (eg where the tip of a steel tube or H section pile is "plugged" with soil) can also be modelled.

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2 Impedance is defined as (pile area * elastic modulus), divided by wavespeed, ((E*A)/c)

3 'Appendix H – CAPWAP correlation studies', in PDA – Pile Driving Analyser, Users Manual, April 1997.

The analysis for each hammer impact involves adjusting the computed pile / soil model to approach the field data obtained from the PDA. Continuous adjustment to the soil model is made until a 'best possible match' between actual and modelled pile response is made. The final set of soil parameters is taken as being the best available representation of actual soil conditions.

CAPWAP analyses are usually undertaken on both end of drive and re-drive blows. Differences between each analysis case for the same pile are very common. Re-drive tests will normally record pile setup components, which are significant in some softer soil profiles. Occasionally, pile relaxation will occur, such as when large displacement piles are driven in dense silty sands where dilation under driving induces negative soil suction forces.

Experience in Auckland currently indicates that large displacement piles generate significant positive pore pressures in weathered Waitemata Group sandstone sequences during driving. Substantial setup on redrive is also indicated in the same materials.

CAPWAP analyses between different pile types in the same soil profile can show quite different pile behaviour, especially in terms of skin friction / end bearing distribution. One project in the upper North Island has CAPWAP results indicating that steel section piles driven into dense sands at depth is causing liquefaction at the pile toe during driving (no base resistance at all). This was contrary to the design assumptions used to establish driving criteria (increasing resistance with depth due to better end bearing). In comparison, precast concrete piles founding in the same geological unit met virtual refusal on an adjacent project.

Figure Two is also taken from the PDA manual and illustrates close agreement between CAPWAP analysed piles and static load testing. CAPWAP is generally taken as being ± 10% accurate compared to static load tests and tends to marginally under-predict higher pile capacities above 2000kN to 2500kN.

3.0 CASE STUDIES

3.1 Westpac Trust Stadium, Wellington – April-June 1998

Brian Perry Ltd was subcontracted to Fletcher Construction Ltd to install some 700 piles on the WestpacTrust Stadium Site in Wellington. Ground conditions on the site were quite variable consisting of several metres thickness of reclamation filling overlying deposits of sand and beach gravels. Underlying materials included weathered gravels and silty sands.

Structural factored pile loads ranged from 500 kN to greater than 2600 kN, with required geotechnical ultimate pile capacities initially set at 2 x factored pile loads (ϕ_g of 0.5).

Pile construction comprised a vibroset driven cast in-situ steel casing system using a sacrificial steel shoe at the pile toe and a large vibratory hammer to extract the casing following rebar placement and concreting. The driving system comprised Junttan 7 tonne and 9 tonne hydraulically accelerated hammers mounted on 40 and 80 tonne crane carriers.

Pile lengths were estimated to be in the order of 7 to 12 metres with required end of drive set and rebound values calculated via the Hiley formula.

After the first few days driving it was apparent that problems were occurring on site including the following:

- Overdriving of casing was occurring to obtain required end of drive sets – piles estimated at 12 metres length were being driven to 17 to 18 metres depth with subsequent cost and production time delays.

- Very high pile rebounds were occurring, due to soil conditions and in part to the high energy being transferred into the pile from the hydraulic hammers.

- Pile casing left to set-up overnight or in some cases over several days to obtain required driving sets were subsequently difficult to extract and several had to be left in place at a cost of over $7,000 per casing.

- Hiley formula did not take into account the high efficiency of the Junttan hammers. Typical hydraulic hammer efficiencies measured on site were in the order of 75% to greater than 95% dependant on ground conditions and driving system performance. Mid range values were grouped around 85%.

- No mechanism was available within the acceptance criteria to account for pile set-up over time, as well as capacity improvements following concreting and casing extraction.

Initial PDA testing was undertaken in late April to establish hammer performance and monitor the load capacity of steel casings during end of drive and redrive conditions.

CAPWAP analysis of the steel casings indicated that virtually all end of drive resistance was coming from end bearing on the oversize shoe with significant side friction developing as a result of pile set-up on redrive. Initial driving formula modifications were undertaken to better model the hammer efficiencies and allow some benefit from pile set-up to be taken in to account.

In mid May and mid June, PDA testing of actual cast in-situ piles was undertaken to better assess pile set-up and the effect of concreting. CAPWAP analysis of concreted piles returned capacities ranging between 2110 kN to over 9040 kN, the latter occurring on a 16.2 metre long pile originally driven to a Hiley capacity of 4447 kN, requiring a structural factored load of 2665 kN. In practice the PDA and CAPWAP analyses were able to demonstrate substantial over capacity of all piles tested.

As a result of these tests the driving formula was calibrated to on-site conditions, the original ϕg of 0.5 increased progressively to 0.67 and then 0.74 and a set-up/concreting factor of 1.3 incorporated to better model actual on-site pile performance.

Production was dramatically increased with virtually no requirement to wait for pile set-up to occur. Pile lengths were reduced to values close to original design. Considerably better information was made available to the consultant concerning pile capacity and greater confidence in the long-term performance of the piles was obtained.

3.2 ALX Building Development, Christchurch Polytech, September 1998

Daniel Smith Industries were engaged to drive some 100, 275-mm square, pre-cast concrete piles on the ALX building site at the Christchurch Polytech in Madras Street, Christchurch.

Ground conditions consisted of 6 plus metres of soft/loose silts and sandy silts overlying dense sands and gravels at depth. Loose materials in the top 6 metres were considered to be prone to liquefaction under moderate earthquake shaking. Static load testing was originally specified for the site.

Four piles were tested at end of drive conditions. Redrive testing was undertaken the following morning. Ideally, more time should have been left for piles to set-up, however due to the relatively cohesionless soil profile, significant set-up did occur.

Pile 80 was driven to 7.9 metres depth and recorded a CAPWAP capacity of 1050 kN at end of drive. This capacity was made up of 750 kN in end bearing, 300 kN side friction with minimal to no side friction indicated over the depth interval where liquefaction was predicted to occur from the geotechnical report. During driving, the pile virtually fell under its own weight over the same liquefiable interval once several driving blows had disturbed the material.

Pile 3B recorded similar end of drive performance to pile 80, however overnight set-up on this pile indicated that pile capacity had increased from 800 kN to 1200 kN. CAPWAP analysis of redrive blows indicated that capacity improvement was virtually all in side friction (approximately 50% within the liquefiable layer) with end bearing in the dense gravels below 6 metres similar to pile 80 at 700 kN.

Based on these results and the other piles tested, the foundation pile layout was changed (removal of central piles in a series of 5 pile groups under several columns) leading to a nett saving of nine piles from that originally scheduled. Considerable time was saved in the programme and the contractor obtained very good information on the efficiency of his hydraulic hammer, information subsequently used on further projects in the Christchurch area.

As an aside, the seismic capacity of the foundation piles was taken as being that available in end bearing under end of drive conditions (approx 700kN). The assumption was made that driving induced liquefaction and ground disturbance during pile installation approximately modelled pile capacity under seismic shaking. A current Masters research project at the University of Canterbury is exploring this approach further.

4.0 AS 2159:1995: Piling – Design & Installation

Australian Standard 2159:1995 allows for a range of geotechnical strength reduction factors (ϕ_g), depending on the method of assessing the ultimate geotechnical capacity of a pile. The standard was originally proposed to be a joint Australian – New Zealand Piling Standard however substantial issues over seismic design were not resolved during the review process in 1993-94 and the Australians proceeded to issue it independently on their own.

Table 4.1 and 4.2 relating to geotechnical capacity reduction are repeated from the standard and reproduced as Figure Three. Note that structural ultimate loads should be divided by the values outlined to calculate required pile geotechnical ultimate capacity.

As one would expect, 'static load tests taken to failure' attract a relatively small amount of geotechnical uncertainty at 0.7 to 0.9. Dynamic pile testing with PDA / CAPWAP analysis has uncertainty values comparable to static load tests at 0.65 to 0.85 with slightly higher values being applicable when the CASE-RMX methods are used with CAPWAP correlation.

Methods using driving formulae such as Hiley have considerably greater uncertainty factors (0.45 to 0.55 and possibly greater). Table 4.1 has additional cautionary notes relating to piles driven into clay. As an aside and an illustration, design tables in Appendix D within NZS 3604:1990 relating to short driven timber piles have an underlying assumption of a factor of safety of 5 within the Hiley formula calculation to ensure piles have sufficient axial capacity. This was based on work carried out in the late 1970's on driven piles in the Auckland area[4].

Current Tonkin & Taylor pile design practice is based on AS: 2159:1995, especially in terms of assessing suitable geotechnical pile capacities. For small domestic type dwellings up to lightweight commercial buildings, conventional driving formulae with suitable uncertainty factors calibrated to site conditions are used. Where projects are larger or piling requirements more complex, PDA / Dynamic pile analysis is used both as a design tool, and in field control.

As a contribution to the current debate over removal of the Hiley formula from the NZ Building Code, we would suggest that the Australian Standard offers reasonable guidance to the Design Engineer in terms of assessing geotechnical pile capacity. The real advantage with the Australian code is it does allow for lower geotechnical ultimate pile capacities to be acceptable, provided more sophisticated testing methodologies are adopted.

Based on the PDA testing undertaken to date in New Zealand, significant cost, construction and time advantages have accrued to projects where PDA has been used.

NOTE: Additional data relating to the two case studies mentioned, copies of papers relating to CAPWAP/static load correlation studies and / or for any queries, please contact Maurice Fraser at Tonkin & Taylor Ltd, 19 Morgan Street, Newmarket, telephone (09)355-6000, 021-378 399 or via e-mail on mfraser@tonkin.co.nz.

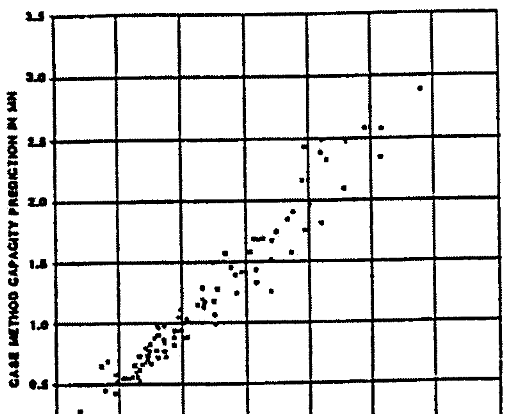
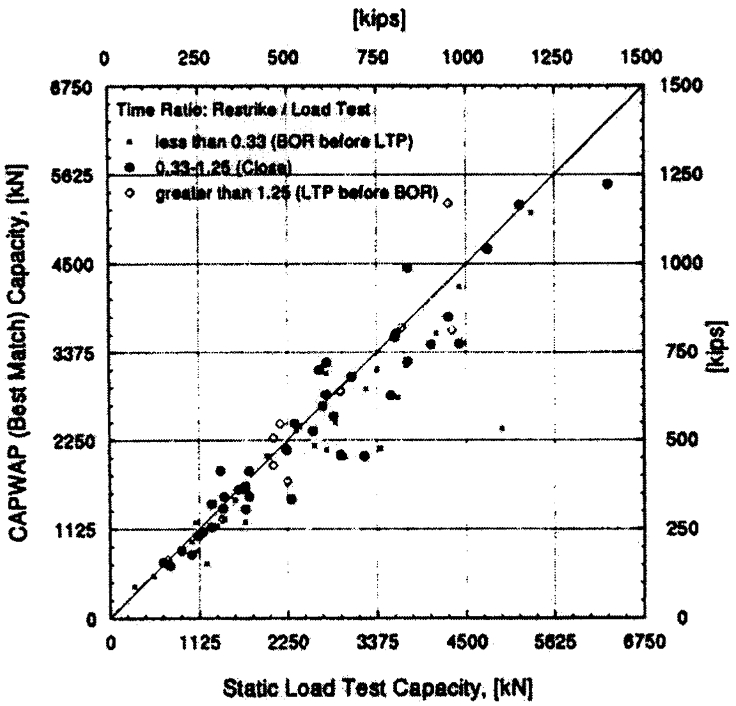
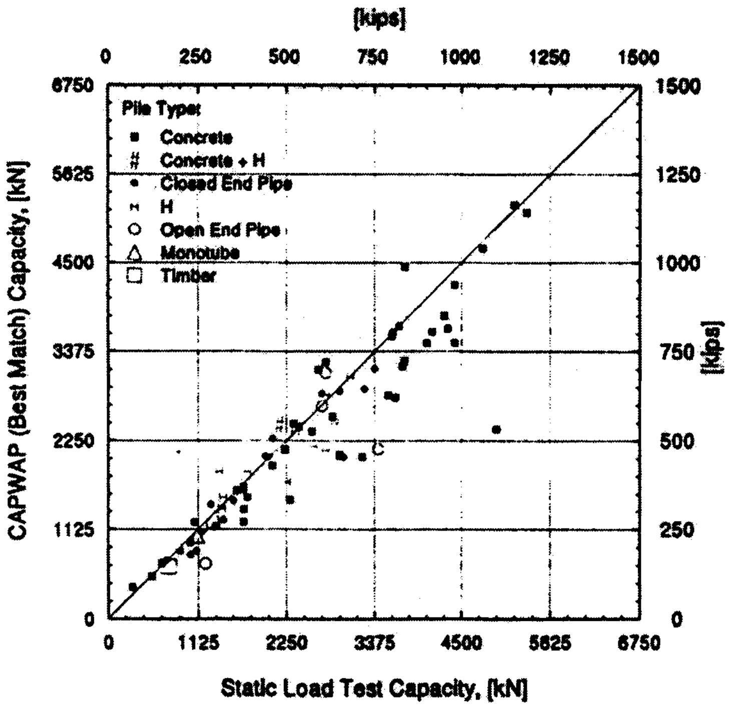


FIGURE 1
Correlation of ultimate static pile bearing capacity by the Case Method and static load tests.

[4] R. Melville-Smith, Foundation Engineering Ltd, personal communications.



CAPWAP (Best Match) Capacity *vs* Static Load Test Capacity
Showing different Time Ratios



CAPWAP (Best match) Capacity *vs* Static Load Test Capacity
showing Different Pile Types

Figure 2 - CAPWAP *vs* Static Load test Capacity Results

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
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

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Back analysis of strength parameters for landslide control works

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ABSTRACT: Two different back analysis methods are presented for strength parameters of a prescribed surface of rupture in a landslide slope. The YUSA method is used for homogeneous slopes and is based on a very simple idea. YUOA, however, is rather a complicated approach based on an optimization technique but has the advantage of dealing with heterogeneous layered slopes as well.

1. INTRODUCTION

Back calculations of strength parameters are often required in remedial works for failed slopes, such as landslides. This paper is concerned with two different techniques for the back analysis of strength parameters along the prescribed surface of rupture in a landslide slope. The first is valid only for homogeneous slopes but is based on a very simple idea (Yamagami and Ueta 1989). Although the second consists of a more complicated procedure with a nonlinear optimization technique, it has the merit of dealing with landslides in inhomogeneous, layered soils (Yamagami and Ueta 1987, 1990); in this paper we call the former YUSA(i.e. Yamagami and Ueta's Simplified Approach) and the latter YUOA(i.e. Yamagami and Ueta's Optimization Approach). The procedure of formulating the back calculation process in the text is described mainly on the basis of YUSA due to space limitation.

In spite of its mathematically simple formulation, YUSA can be applied to noncircular slip surface situations. However, different treatments are necessary according to the factor of safety equation used. Here we choose to employ the Janbu method to show the treatment of the back analysis. This is followed by some practical applications to illustrate that the methods proposed will be very effective tools in estimating strength parameters of landslides when performing the control works.

2. BACK ANALYSIS PROCEDURE

The strength parameters are back analyzed by applying a factor of safety equation in both approaches presented in this paper. We first represent the factor of safety equation in a general form as

$$F = f\ (c,\ \phi,\ M,\ F) \tag{2-1}$$

where F denotes a value of the factor of safety for an assumed slip surface, c and ϕ are strength parameters, and M signifies the self-weight of the sliding mass, external forces acting on the ground surface, the effect of pore water pressure and so forth. The above equation indicates that the safety factor function f consists of c, ϕ, M and the factor of safety itself.

Now, suppose that in Fig.1 the curve AOB is the surface of rupture (SR) and its factor of safety being F_0. A value of between 1.0 and 0.9 is usually assigned to F_0 according to the situation. It should be noted that F_0 must be a minimum value for the factor of safety of the slope because the curve AOB is SR. Once the value of F_0 has been determined, the

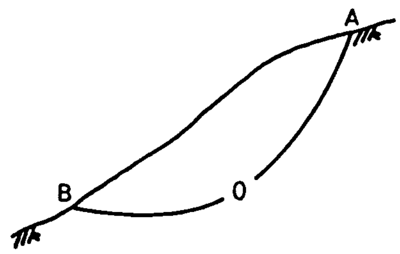


Fig.1 Given surface of rupture

following expression can then be written from Eq.(2-1) for the given surface AOB:

$$F_0 = f\ (c,\ \phi,\ M,\ F_0) \qquad (2\text{-}2)$$

For example, when the Janbu method is employed, Eq,(2-2) becomes:

$$F_0 = \frac{1}{\Sigma (p+t)\Delta x \tan\alpha} \times \Sigma \frac{\{c+(p+t-u)\tan\phi\}\ \Delta x\ (1+\tan^2\alpha)}{1 + \tan\alpha \tan\phi / F_0} \qquad (2\text{-}3)$$

At this stage we must emphasize that any satisfactory back analysis method must meet the following two essential requirements:

a) The strength parameters to be back analyzed must satisfy the factor of safety equation used such as Eq.(2-3).

b) The factor of safety F_0 for SR must be a minimum value of the slope under consideration.

2-1. YUSA for Slopes Consisting of Homogeneous Soil

Basic idea: The only unknowns are a pair of strength parameters c_0 and ϕ_0 where the slope is assumed to be homogeneous in strength. In this case we can obtain the so called c-tanϕ relationship as shown schematically in Fig.2 where c_{max} and ϕ_{max} denote the maxima of all the respective values of c and ϕ that satisfy Eq.(2-2). In other words, the strength parameters c_0 and ϕ_0 to be back analyzed must lie somewhere on the c-tanϕ curve.

The basic idea of YUSA, whose purpose is to obtain c_0 and ϕ_0, is illustrated in Figs.3 and 4. Fig.3 represents a three dimensional space with the factor of safety and the strength parameters. The curve PIQ in Fig.3 schematically illustrates a possible existing range of c and tanϕ corresponding to the factor of safety F_0 for SR(AOB) given in Fig.4. The strength parameters c_0 and $\tan\phi_0$ should therefore exist somewhere on the curve PIQ (see Requirement a) above). Fig.4 shows SR and an appropriate number of trial slip surfaces assumed above and below it. Here we will develop the back analysis procedure by fixing the end points A and B of the slip surfaces for computational convenience.

Let the factor of safety F be expressed as follows for a trial slip surface other than SR:

$$F = f\ (c,\ \phi,\ \underline{M},\ F) \qquad (2\text{-}4)$$

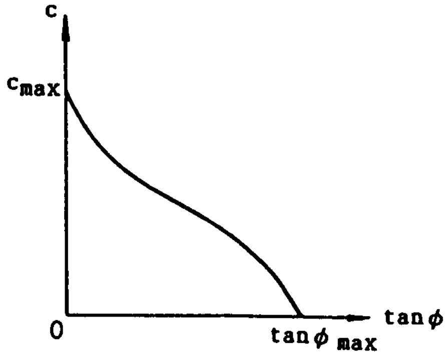


Fig.2 c -tanϕ relationship

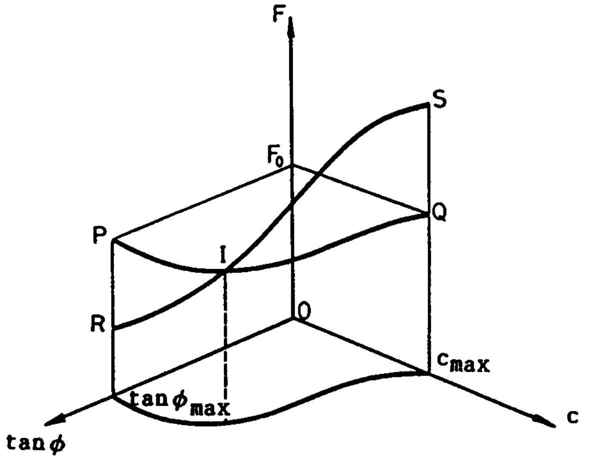


Fig.3 (F,c,tanϕ) three-dimensional space

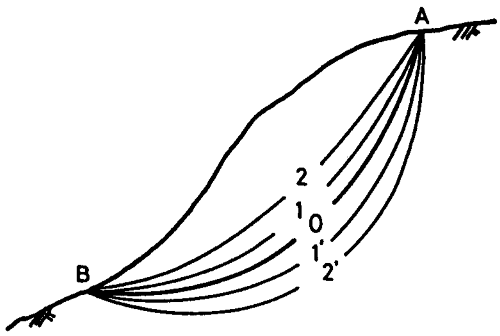
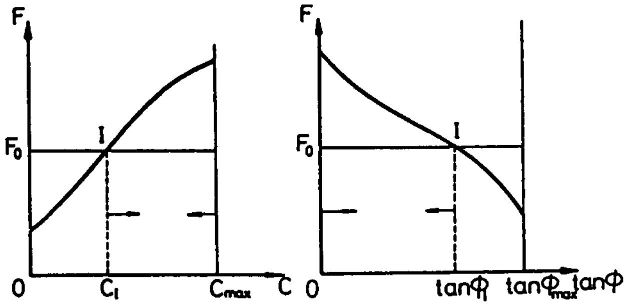


Fig.4 surface of rupture and trial slip surfaces

where the underlined notation $\underline{M}$ indicates the amount evaluated from the trial slip surface. It is anticipated that the distribution of F produces a curve such as RIS in Fig.3 when c and tanϕ in Eq.(2-4) are varied along the curve PIQ, i.e. varied in a way that Requirement a) above is satisfied. Fig.5 illustrates this idea in two dimensional F-c and F-tanϕ coordinate systems. However, according to Requirement b) we must always satisfy the following inequality:

$$F \geq F_0 \qquad (2\text{-}5)$$



(a)restriction based on F-c relationship
(b)restriction based on F-tanϕ relationship

Fig.5 Restriction of ranges within which the strength parameters should exist

The above equation suggests that with reference to Fig.5 the correct parameters c_0 and $\tan\phi_0$ should exist within the following ranges, respectively:

$$\left.\begin{array}{l} c_l \leqq c_0 \leqq c_{max} \\ 0 \leqq \tan\phi_0 \leqq \tan\phi_l \end{array}\right\} \qquad (2\text{-}6)$$

If the inclination of the curve representing the distribution of F is the reverse to that of Fig.3 or 5, then the ranges within which c_0 and $\tan\phi_0$ should exist will become

$$\left.\begin{array}{l} 0 \leqq c_0 \leqq c_l \\ \tan\phi_l \leqq \tan\phi_0 \leqq \tan\phi_{max} \end{array}\right\} \qquad (2\text{-}7)$$

In any case it is expected that from one trial slip surface, the existing ranges of c_0 and $\tan\phi_0$ will be considerably restricted by virtue of Eq.(2-5). Hence, ranges within which c_0 and $\tan\phi_0$ must exist will be extremely restricted by assuming several trial slip surfaces above, below and adjacent to SR as shown in Fig.4. By applying Eqs.(2-6) or (2-7) to each of the slip surfaces, we are then able to identify the correct strength parameters c_0 and $\tan\phi_0$.

These are the fundamental principles of YUSA. Although varying treatments are required according to the factor of safety equation employed for Eq.(2-1), in each case it has been ascertained that the back analysis is successfully performed when based on the central concepts above.

Using the Janbu method: In YUSA, it is essential that we have to automatically locate the trial slip surfaces adjacent to the SR. However, in the case of noncircular slip surfaces, this is not easy as there are numerous possible slip surface configurations. Accordingly, we have set up the following three guidelines in order to locate the trial slip surfaces:

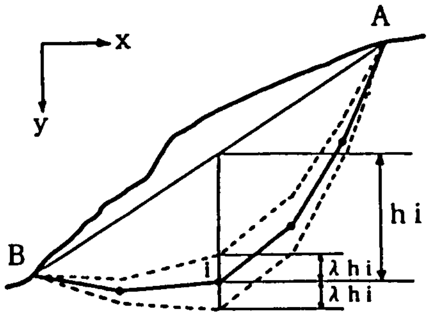


Fig.6 Determination of trial slip surfaces

- A slip surface consists of a chain of linear segments.
- The shape of a trial slip surface is quite similar to that of SR.
- Trial slip surfaces are created by vertically shifting all the nodes up and down except for the end points on SR (see Fig.6).

With these assumptions in mind the procedure to determine the trial slip surfaces runs as follows:

In Fig.6 the solid curve represents SR. First, end points A and B are connected by a straight line. Next, let the vertical distance be h_i between each node on SR and the straight line $\overline{AB}$. All the nodes on SR except for the end points are then moved vertically by an amount of λh_i, by introducing a small arbitrary constant λ. This operation will produce a trial slip surface similar in shape and in close proximity to the original SR as shown schematically in Fig.6. The trial slip surface is located below SR when $\lambda>0$ and, conversely, above when $\lambda<0$.

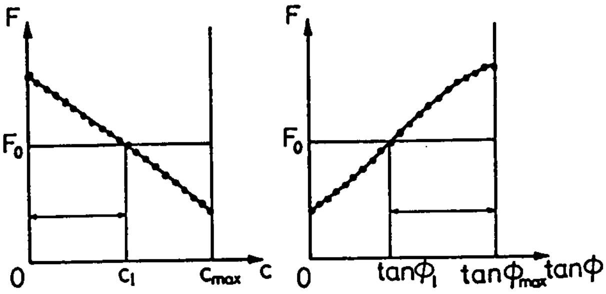
Now, let us suppose that the factor of safety F is expressed in terms of the Janbu method for an arbitrary trial slip surface as below:

$$F=\frac{1}{\Sigma\,(\underline{p}+\underline{t})\,\Delta\underline{x}\,\tan\underline{\alpha}} \times \Sigma \frac{\{c+(\underline{p}+\underline{t}-\underline{u})\tan\phi\}\;\Delta\underline{x}\;(1+\tan^2\underline{\alpha})}{1+\tan\underline{\alpha}\tan\phi/F} \qquad (2\text{-}8)$$

where, as stated earlier, the underlined notations denote values obtained from a trial slip surface other than SR. The restriction of existing ranges c_0 and ϕ_0 based on Eqs.(2-3) and (2-8) thus leads to the schematic diagrams shown in Fig.7. As is evident from Eq.(2-8), the variation of F with c or $\tan\phi$ which satisfies the c-$\tan\phi$ relationship, Eq.(2-3), cannot be expressed explicitly as it can be in the case of the ordinary method of slices (Yamagami and Ueta 1989). Thus the curves shown in Fig.7 must be obtained by substituting the coordinates,(tanϕ, c), of many points satisfying the c-tanϕ relationship Eq.(2-3), into Eq.(2-8), and then by determining the corresponding values for F.

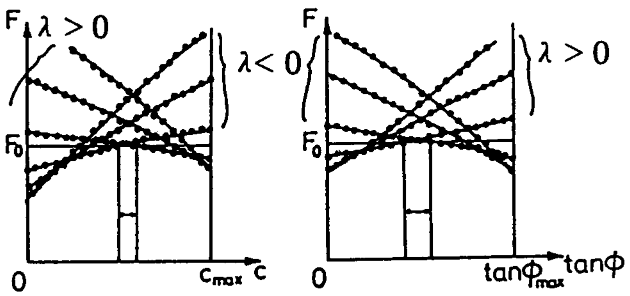
Fig.8 schematically illustrates a situation in which the existing ranges of the strength parameters to be obtained have been restricted by applying the procedure shown in Fig.7 to several trial slip surfaces adjacent to SR.

A more efficient and systematic approach for determining a unique pair of solutions is to utilize Fig.9. The dots ● indicate the relationships between the value of λ and the maximum or minimum values of the possible ranges for c_0 and $\tan\phi_0$ corresponding to each trial slip surface (see Fig.8). These maximum or minimum values should be read from Fig.8. Consequently, the abscissa of the intersection of a smooth curve connecting the dots and the straight line: λ=0 will yield the solution as shown in Fig.9.



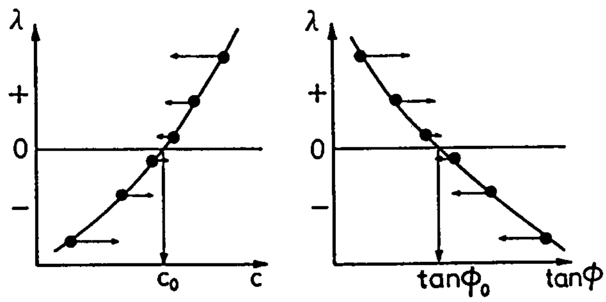
(a)restriction based on F-c relationship (b)restriction based on F-tanϕ relationship

Fig.7 Restriction of ranges within which the strength parameters should exist



(a)determination based on F-c relationship (b)determination based on F-tanϕ relationship

Fig.8 Final determination of existing ranges for the strength parameters



(a)back calculation procedure based on λ -c relationship (b)back calculation procedure based on λ -tanϕ relationship

Fig.9 More efficient and systematic back calculation procedure

2-2. YUOA for Inhomogeneous Layered Soils

In YUOA the back analysis problems are formulated by the use of nonlinear programming as an optimization process with mixed equality and inequality constraints. The basic idea of YUOA and its formulation where slip surfaces are assumed to be circular in shape, have already been published elsewhere (Yamagami and Ueta 1987, 1990). A brief discussion has also been given on the back analysis formulation even for noncircular slip surface situations in previous papers. However, what we propose is a more simplified formulation as an optimization problem for landslides with noncircular slip surfaces.

Let the solid line AOB in Fig.10 represent a given surface of rupture (SR) as before, F_0 being its factor of safety. Suppose that the search for the strength parameters are performed assuming that the end points A and B are fixed. Also, let c_0 and ϕ_0 denote the true strength parameters to be back analyzed, and c and ϕ a pair of strength parameters other than c_0 and ϕ_0. Then, given c and ϕ, the slope must have a minimum factor of safety along a curve different from AOB, for example the dashed line AO'B. Here, the search for the location of the line AO'B, i.e. the critical slip surface corresponding to the strength parameters c and ϕ, is made with the following procedure:

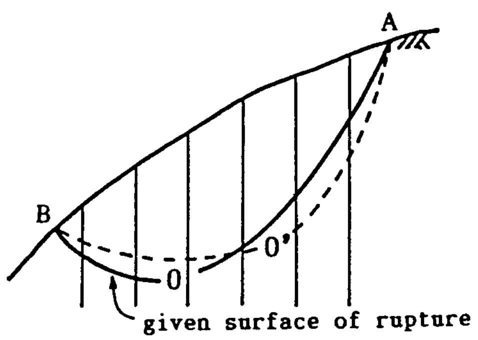


Fig.10 Explanation of fundamental concepts

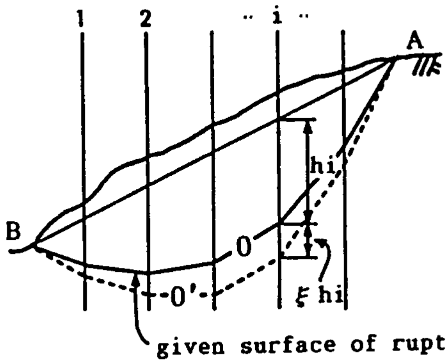


Fig.11 Schematic diagram illustrating the one-dimensional search problem

First, as seen in Fig.11 the sliding soil mass under consideration is divided into an appropriate number of slices with vertical slice lines. Then, let each vertical distance be h_i between SR and the straight line connecting the two points A and B. With the introduction of an arbitrary variable ξ, a trial slip surface can now be located at a distance of ξh_i from SR on each slice line. Consequently, the factor of safety for the trial slip surface so obtained may be regarded as a function of ξ. Thus, the curve AO'B can be determined by searching for a value of ξ which minimizes the factor of safety. This process can be considered a one-dimensional search problem in which the objective function is the factor of safety and where the dependent variable is ξ. Hence we have employed the golden section method to solve the search problem.

Note that the curve AO'B obtained with the procedure mentioned above does not necessarily represent the real critical slip surface corresponding to the strength parameters **c** and ϕ. This is because the curve AO'B is restricted to either side of SR and does not intersect it. Despite this restriction, however, we have confirmed the effectiveness of the proposed method.

The value for ξ corresponding to the curve AO'B or the strength parameters **c** and ϕ other than c_0 and ϕ_0 will never be zero; it will become zero only when the strength parameters **c** and ϕ coincide with c_0 and ϕ_0, respectively. Therefore we can define the following optimization problem for the back analysis of strength parameters:

$$\text{minimize} \quad U(c, \phi) = \xi^2 \qquad (2\text{-}9)$$

subject to
equality constraint

$$F = F_0 \qquad (2\text{-}10)$$

and
Inequality constraints

$$\left.\begin{array}{l} 0 \leq c \leq c_{max} \\ 0 \leq \tan\phi \leq \tan\phi_{max} \end{array}\right\} \qquad (2\text{-}11)$$

It is clear from the discussion above that the proposed optimization approach can deal with landslide slopes consisting of inhomogeneous, layered soils.

3. APPLICATIONS

Applications of YUSA to two actual landslides are given here; in these two cases, box shear tests have been conducted. We have already ascertained that YUOA provides satisfactory results in actual cases of homogeneous slopes and in hypothetical cases of inhomogeneous layered slopes. However YUOA has not yet been applied to real inhomogeneous slopes, because there exists no reliable and accurate information on such cases at hand.

<u>The first example</u>: Fig.12 shows the landslide caused by an excavation at site A; detailed investigations were made in a shaft to identify the location of the surface of rupture, and so forth. In situ and laboratory direct-shear tests were also conducted on several samples, resulting in c' = 0.08 ~ 0.38kgf/cm^2 and ϕ' =15° ~ 17°.

We have performed the back analysis assuming that the location of SR and the value of F_0 (=1.0) are known, but the strength parameters are not.

The results are given in Fig.13 on the c-tanϕ relationship.

<u>The second example problem</u>: Fig.14 shows the landslide occurred in a cut-slope in the Izumi Group Shale. A laboratory direct shear test was conducted on block sample specimens obtained near the SR; the results were c' = 0.0tf/m^2; ϕ' = 16.5°.

The back analysis was performed under the same condition as the first example, and the results are given in Fig.15.

As seen in the two examples above, the predicted values correspond relatively well with

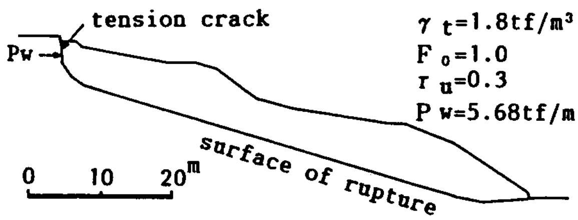


Fig.12 First example problem

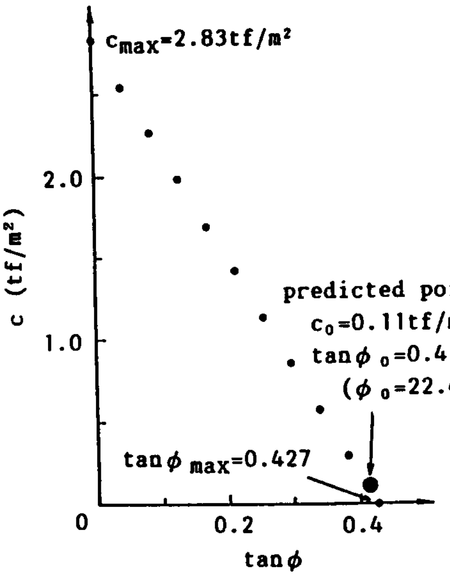


Fig.13 c -tanϕ relationship

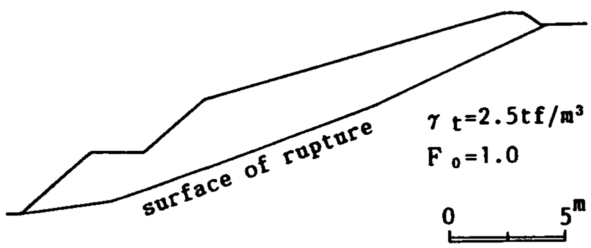


Fig.14 Second example problem

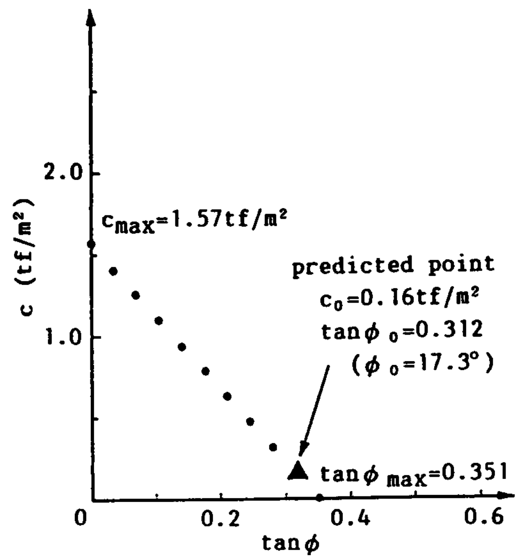


Fig.15 c -tanϕ relationship

the experimental ones. It is however not necessarily essential that the back-analyzed and experimental values coincide with each other, as the main purpose of YUSA is to yield an average pair of c and ϕ along SR as design parameters.

4. CONCLUSIONS

Two back analysis procedures, YUSA and YUOA, have been presented which are very effective for landslide remedial or control works. The function of YUSA is to back analyze a unique pair of c and ϕ along the prescribed surface of rupture (SR). The strength parameters obtained from YUSA are thus considered to represent an average value of c and ϕ respectively, for SR. However, we believe that the average values are still sufficient for the designs of most landslide control works. Applications to practical problems have revealed that YUSA gives quick and reliable solutions with the aid of personal computers.

In theory, YUOA can be applied to inhomogeneous strength parameters along SR, though examples have not been demonstrated in this paper due to lack of the observed data on a landslide consisting of inhomogeneous soils. Another possibility and an attractive point to this approach is that it may be extended to form a back analysis method for pore water pressure distribution along SR. This will be presented on another occasion.

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Development of Laboratory Loadman Test for Determination of the Elastic Modulus of Road Pavement Materials

R Peploe

1.0 INTRODUCTION

The AUSTROADS pavement design procedures were adopted by Transit New Zealand in 1995. These procedures involve modelling prospective pavement structures using multi-layer analysis software to determine the magnitude of strains that occur at critical locations within the pavement. This has created a demand for cost-effective methods of establishing the elastic parameters for pavement materials. The Loadman Portable Falling Weight Deflectometer (FWD), which has been introduced to the New Zealand market in relatively recent times (1996), shows significant potential as a very useful tool for the pavement engineering practitioner to obtain these parameters. Also when compared with other test procedures the Loadman is relatively inexpensive and is quick and simple to operate.

1.1 The Loadman

The Loadman Portable FWD basically comprises an 800-mm long by 132 mm diameter aluminium tube with a 10 kg weight inside it. The tube is closed at both ends with an electromagnet at the top and a circular steel plate at the bottom. A schematic representation of the Loadman is presented in Figure 1. When the Loadman is placed on a pavement material and activated, the weight falls onto the base plate and imparts a dynamic load to the material beneath the device. The Loadman is supplied with an additional base plate, 200 mm in diameter, that can be bolted to the bottom of the device. The acceleration - time history of the device is monitored during the test using an internal accelerometer. This acceleration is integrated to a deflection which is then analysed to determine the material's elastic modulus parameter.

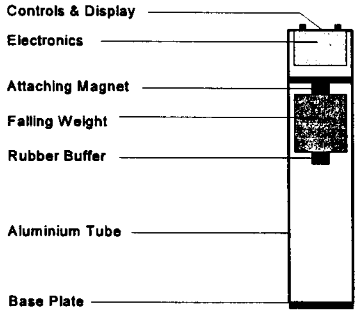

Figure 1 Schematic Loadman cross section.

One of the drawbacks of the Loadman is that it is programmed to calculate elastic modulus values using the Boussinesq theoretical model. The Boussinesq model comprises a single layer of isotropic material, but pavements are generally made up of multiple material layers. In addition, the AUSTROADS pavement design procedures specify that granular and subgrade materials be treated as having anisotropic elastic parameters.

Another source of error in the Loadman testing procedure is the approximation of a constant dynamic load for all tests. It is recognised that the load applied by the Loadman varies depending on the response of the test material. However there is no experimental data available at this time to establish the exact loading magnitude for each test. It is understood that a new Loadman apparatus is being planned and it includes a load cell to measure the applied dynamic load.

2.0 LABORATORY TEST

Research, carried out by Bartley Consultants Ltd (1998), involved the development of a laboratory testing procedure to maximise the utilisation of the Loadman and overcome the limitations imposed by the assumption of the Boussinesq model in the analysis.

The laboratory Loadman test uses a back-calculation technique to establish the elastic properties of the test material. Specimens are prepared in a 0.5 m x 0.5 m x 0.5 m steel box and the Loadman is then used to both impose the dynamic load and to measure the elastic response of the specimen. A multi-layer elastic computer program, CIRCLY, is used to back-calculate the anisotropic (or isotropic) elastic modulus.

The widespread availability and user friendliness of the computer program CIRCLY means that it is desirable to use it for the back-calculation analysis of laboratory Loadman tests. Unfortunately, CIRCLY models the implied load as a uniform pressure and hence non-uniform displacement but the rigidity of the Loadman's base plate results in uniform displacement and hence non-uniform pressure. Therefore, an analysis was undertaken to verify that the CIRCLY program could model the Loadman test conditions in a representative fashion.

2.1 Analysis of CIRLCY Model

The analysis simply compared predicted deflection bowl shapes from CIRCLY to those obtained using the finite element modelling program LUSAS. Three pavement structures were modeled using CIRCLY and LUSAS. Each model comprised a 250 mm thick layer of linear elastic material with an underlying rigid base. The elastic modulus values for the soil layer were 30 MPa, 50 MPa and 100 MPa for the three structures respectively. In addition, the soil layer was treated as being anisotropic with a degree of anisotropy of 2.0. A Poisson's ratio value of 0.35 was assigned to the soil layer in each model.

For each pavement model, three loading situations were considered, one using the CIRCLY model and two using the LUSAS models. In the CIRCLY model a vertical stress of 1.61 MPa was applied uniformly over a circular area with a diameter of 132 mm. In the LUSAS models two loading cases were used. One applied the 1.61 MPa stress to the model via a 132 mm diameter circular steel plate with an elastic modulus of 210 GPa. A second analysis applied the stress to the model via a rigid plate. The actual situation with the Loadman is considered to be somewhere between the steel and the rigid plate situations. This is because the Loadman base plate is bolted to the cylindrical body of the device giving it a degree of fixity that has not otherwise been considered in the analysis. In both the LUSAS models the load was transferred to the base plate only over the central 80 mm of the plate. This corresponds approximately to the dimensions of the rubber buffer that cushions the impact between the falling weight and the Loadman base plate.

Figures 2, 3 and 4 show plots of vertical deflection at the top of the models versus lateral offset for models with (vertical) elastic moduli of 30 MPa, 50 MPa and 100 MPa respectively. Note that the zero offset corresponds to the centre of the Loadman base plate. The vertical dashed line in each plot represents the edge of the base plate.

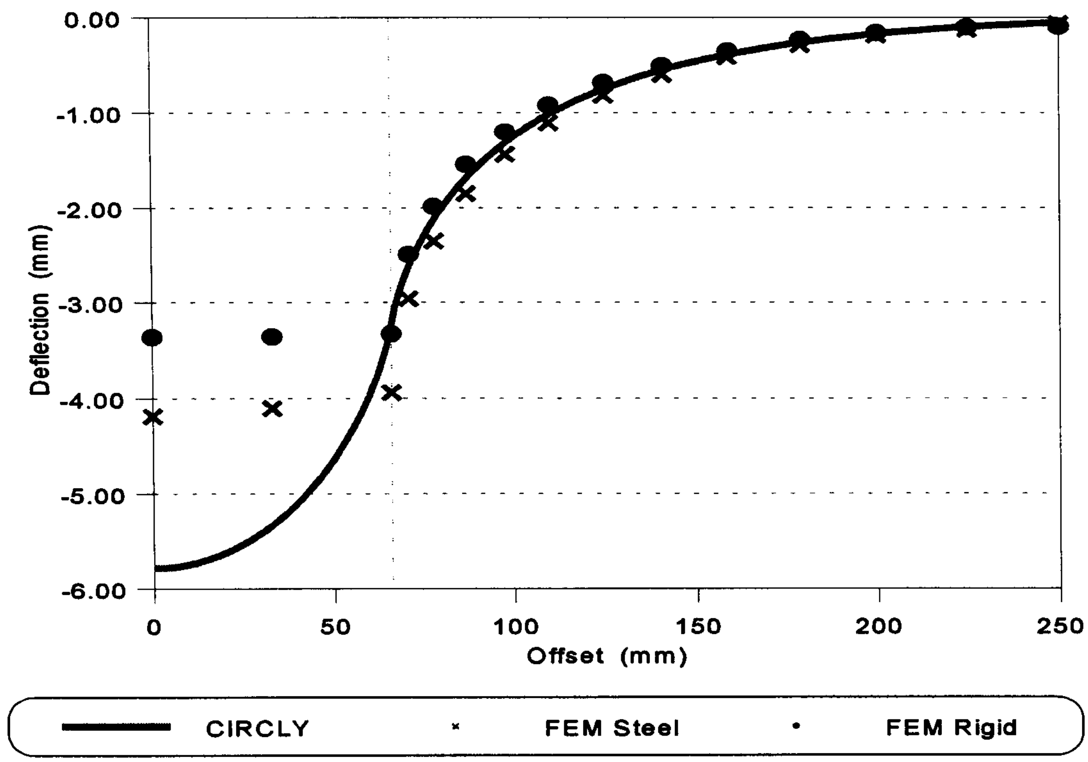

Figure 3 FEM / CIRCLY Loadman analyses (Ev = 30 MPa).

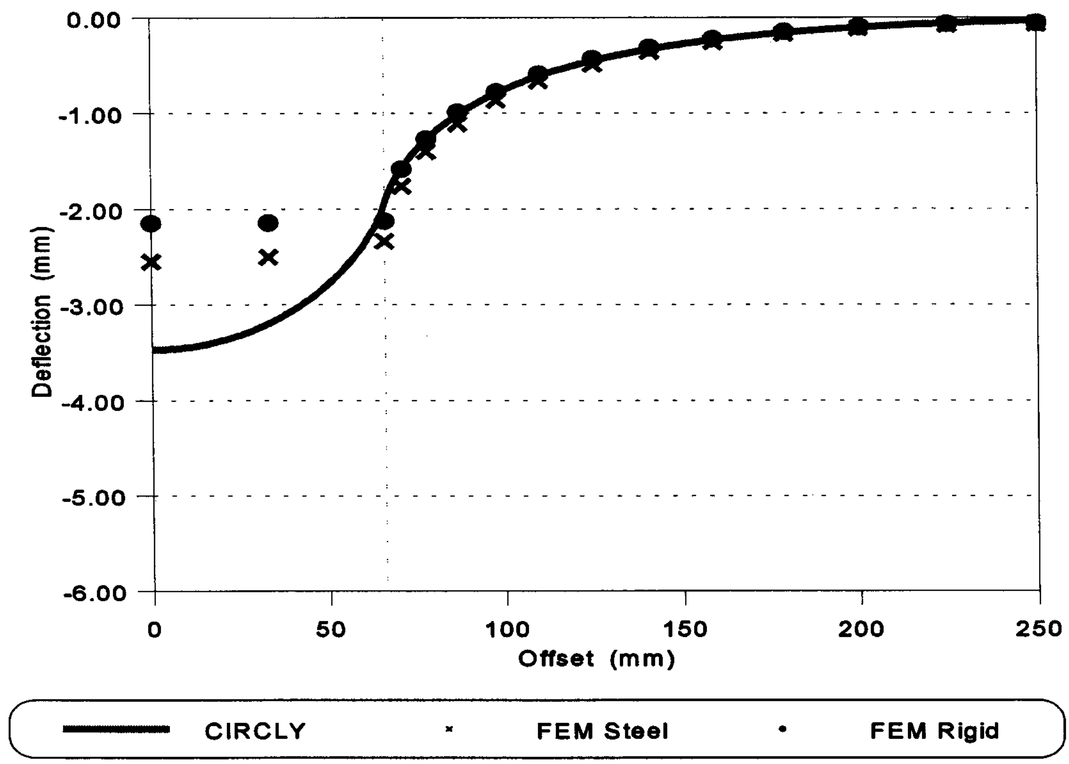

Figure 2 FEM / CIRCLY Loadman analyses (Ev = 50 MPa).

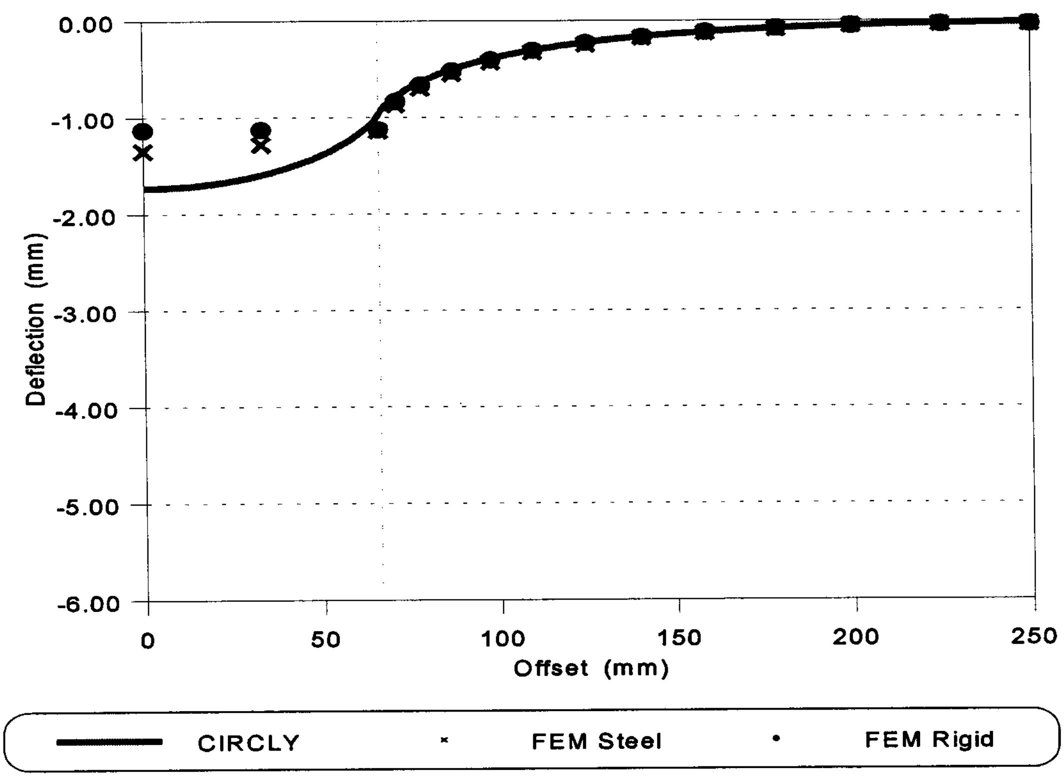

Figure 4 FEM / CIRCLY Loadman analyses (Ev = 100 MPa).

The result that is significant from Figures 2, 3 and 4 is that the deflections for the rigid base plate and the flexible CIRCLY base plate coincide very well at the edge of the plate. As this is the deflection that is measured in a Loadman test it is reasonable to conclude that CIRCLY is a suitable program for back-analysing Loadman deflections as long as the base plate edge deflection is used in the CIRCLY model and not the central deflection.

3.0 COMPARATIVE STUDY

To further evaluate the Loadman's reliability in determining the elastic parameters of a pavement material, a comparative study was carried out on laboratory Loadman test results and data obtained using repeated load triaxial (RLT) tests. The RLT test is the recognised standard test for determining the elastic modulus of pavement materials.

3.1 The Materials

Three aggregate specimens and two subgrade soil specimens were selected on the basis that RLT test results were available for these materials for comparative purposes. The materials used in the testing were as follows:

- Waitemata Group silty clay recovered from the Albany area of North Shore City;
- pumice soil recovered from Kaingaroa Forest;
- AP25 andesite aggregate from Winstones Flat Top Quarry;
- AP65 greywacke aggregate from Stevenson's Drury Quarry; and
- AP65 stabilised greywacke aggregate from Stevenson's Drury Quarry.

In general terms, the Waitemata Group and pumice soils represent common sedimentary and volcanic subgrade soils respectively. With respect to the aggregates, both the Flat Top AP25 and the untreated Drury AP65 represent a good quality subbase. The stabilised AP65 represents a subbase material suitable for heavily loaded pavement applications.

3.2 Testing and Results

All specimens were tested using both the 132 mm diameter and the 200 mm diameter Loadman base plates.

To provide a realistic comparison between the results of the laboratory Loadman tests and the RLT tests the state of stress occurring in the laboratory Loadman test must be considered. This has been achieved using CIRCLY to calculate the prevailing deviator stress or mean normal stress as required. The location within the Loadman specimen that has been used to characterise the state of stress has been taken as the point at which the vertical stress is expected to reduce to one half of the magnitude of the stress applied at the surface. Lambe and Whitman (1979) provide influence charts that establish the appropriate depth to be approximately 1.3 times the radius of the loaded area. Therefore, the state of stress for the 132 mm diameter Loadman base plate has been determined at a depth of approximately 86 mm. On the same basis, the state of stress for the 200 mm diameter Loadman base plate has been determined at a depth of approximately 130 mm.

Comparisons of resilient modulus values from the laboratory Loadman and RLT tests are presented in Figures 5 to 9. Note that the configurations of the RLT tests are not entirely consistent as the results were obtained from a range of projects that were carried out prior to the current project. When viewing Figures 5 to 9 the reader should be aware that resilient modulus test results are generally plotted on a log scale. The results shown below have been plotted on arithmetic scales so that the magnitude and variation of the data is not masked by non-linear scales.

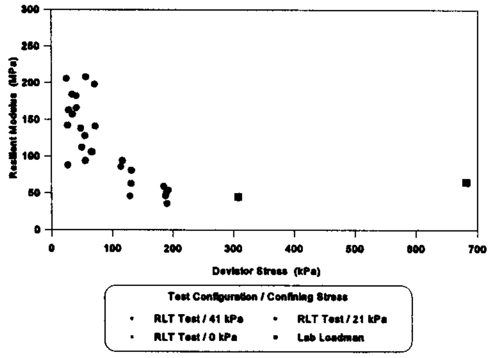

Figure 5 RLT test and laboratory Loadman test results - Waitemata Group soil specimens.

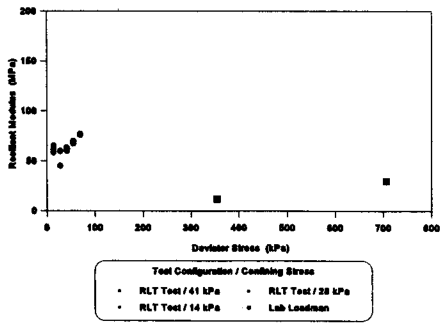

Figure 6 RLT test and laboratory Loadman test results - Kaingaroa pumice soil specimens.

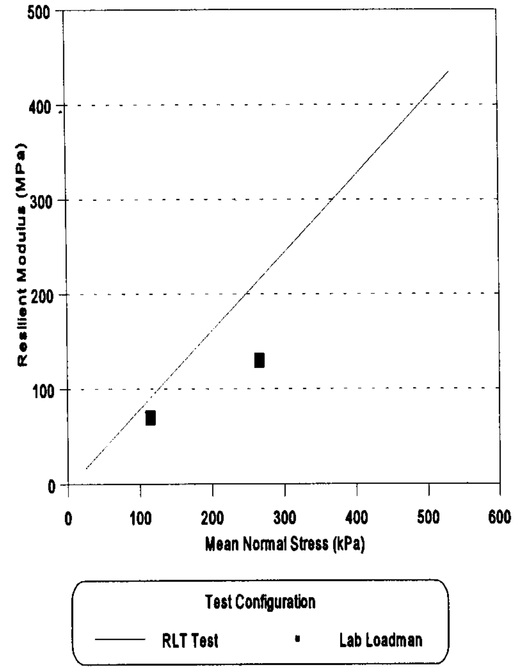

Figure 7 RLT test and laboratory Loadman test results - AP25 Flat Top aggregate.

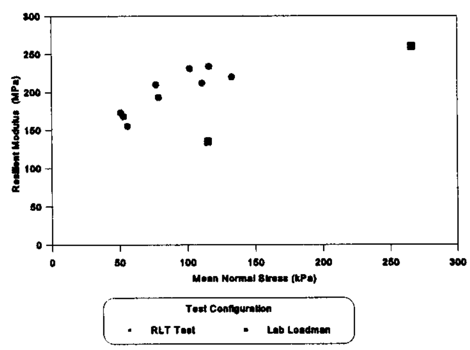

Figure 8 RLT test and laboratory Loadman test results - Drury AP65 (untreated) aggregate specimens.

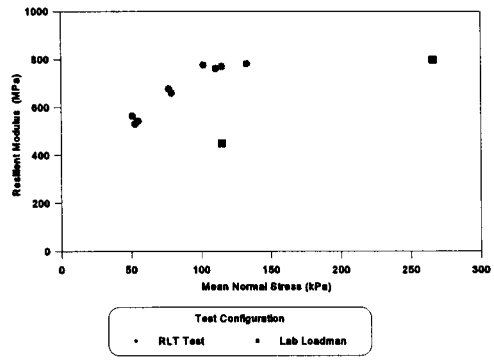

Figure 9 RLT test and laboratory Loadman test results - Drury AP65 (stabilised) aggregate specimens.

3.3 Discussion

The data shown in Figures 5 to 9 indicates that the Laboratory Loadman tests provide resilient modulus values that correlate reasonably well with the corresponding values obtained using RLT tests. In general, if the Loadman results differ from the RLT results, the Loadman elastic moduli are slightly lower.

An inconsistency between the laboratory Loadman results and the RLT test results that should be noted is with respect to anisotropy. In the analysis of the Loadman tests the material can be modeled as an anisotropic material whereas typical RLT tests do not include sufficient measurements to allow anisotropy to be considered. However, including anisotropy in the resilient modulus results causes the vertical resilient modulus value to increase, which is opposite to the trend shown in Figures 5 to 9.

Another important observation derived from Figures 5 to 9 is that the stresses imposed by the Loadman are significantly higher than those applied during typical RLT tests. This is especially true for subgrade soils which experience relatively low in-service stresses because of the low elevation in the pavement. This could be resolved by increasing the size of the Loadman base plates.

4.0 CONCLUSION

The advantage of the proposed laboratory Loadman test over tests such as the RLT is that it is quick and simple to perform, it requires very little equipment, and representative specimens can be tested. This means, for example, that aggregates with particle top sizes of 65 mm or even 100 mm can be tested, with no need to "scalp' the larger particles (i.e. removing particles retained on the 19 mm sieve) as is required for most repeated load triaxial (RLT) tests. However the level of stress that is imposed by the Loadman may be greater than that occurring in-service, especially for subgrade soils.

The Loadman could be developed so that it incorporates an internal load cell to measure the applied dynamic load then the results would be even more favourable. This would result in a new method of determining anisotropic elastic modulus data that is simple, quick and cost-effective.

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Bartley Consultants Ltd (1998)	*Loadman Portable Falling Weight Deflecometer : Development of Testing Procedures*, Transfund New Zealand Research Report No. 125, Transfund New Zealand.
Lambe, T.W. & Whitman, R.V. (1979)	*"Soil Mechanics : SI Version"*, Series in Civil Engineering, John Wiley & Sons Ltd, USA.

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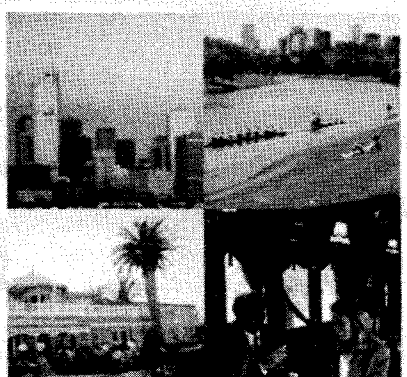

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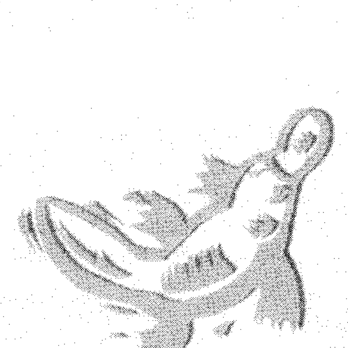

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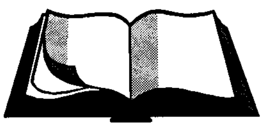

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NEW ZEALAND GEOTECHNICAL SOCIETY

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OBJECTS

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

MEMBERSHIP

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

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

ANNUAL SUBSCRIPTION

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

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

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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*
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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

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

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

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

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

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

(for office use only)

Received by the Society..

Recommended by the Management Committee of the Society..

Approved by the Council of the Institution..

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 (cm)
Black & White Back Cover	$275	19.5 wide x 27 high
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Full Page Internal	$200	19.5 wide x 27 high
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*Note:
1. All rates are excluding GST
2. Space is subject to availability
3. All margins are 10 mm

The deadline for advertising copy for the next issue is 1 November 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:

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