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N.Z. GEOMECHANICS NEWS

No. 33

NOVEMBER 1986

A NEWSLETTER OF THE N.Z. GEOMECHANICS SOCIETY

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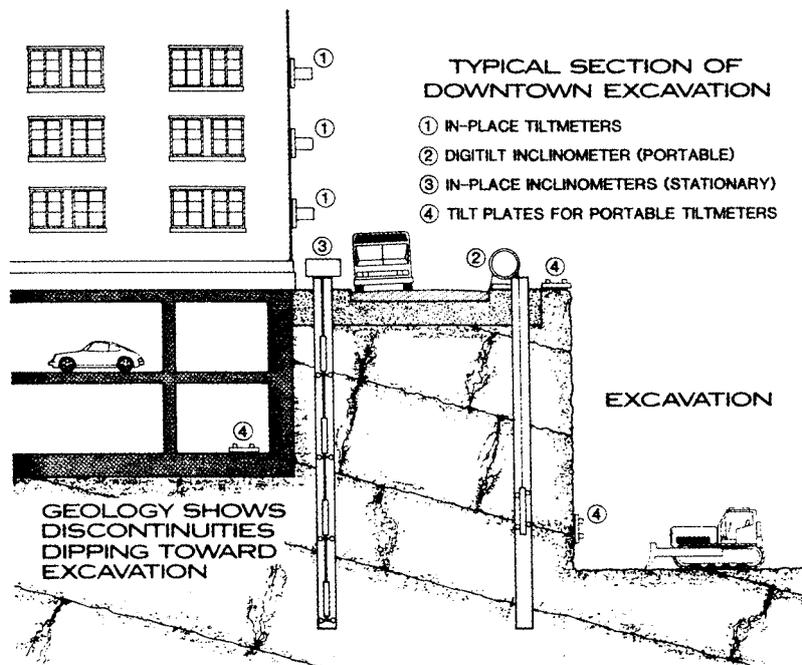
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NZ GEOMECHANICS NEWS

NO. 33, NOVEMBER 1986

A NEWSLETTER OF THE NZ GEOMECHANICS SOCIETY

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THIS IS A RESTRICTED PUBLICATION

"NZ Geomechanics News" is a newsletter issued to members of the NZ Geomechanics Society. It is designed to keep members in touch with recent developments. Authors must be consulted before papers are cited in other publications.

Persons interested in applying for membership of the Society are invited to complete the application form at the back of this newsletter. The basic annual subscription rate is \$20.00 and is supplemented according to which of the international societies, namely Soil Mechanics (\$11.00), Rock Mechanics (\$15.00), or Engineering Geology (\$7.00) the member wishes to be affiliated. Members of the Society are required to affiliate to at least one International Society.

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AUCKLAND
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WELLINGTON NORTH
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EDITORS NOTES

A milestone was reached this year for the New Zealand Geomechanics Society with the first award of Life Membership to Dr Ken Birrell in recognition of his service to the society (see page 3). It is people of such calibre as Dr Birrell who have pioneered and advanced the study and practice of geomechanics in New Zealand and who have provided the wealth of experience and technique which we are fortunate to benefit from today.

The 5th geomechanics lecture is to be presented mid next year by Mr Les Osborne, a man who has given much to the study of geology and in particular who has been instrumental in linking geology and engineering in N.Z. His contributions to geological investigations in the context of hydro schemes and with particular reference to seismotectonics will be known to many. Details of this prestigious lecture are given on page 23.

The 1986 symposium on piled foundations was a notable success, well attended, and a credit to the organisers. A report on the symposium is given on page 18 including Dr Harry Poulos's resume. I hope that his call for increased monitoring performance of piles in order to determine just how accurate our analysis and predictions are will be heeded. We have the example of people such as Dr Birrell and Mr Osborne who have not only developed their fields of study but who have also made their results freely available for others to use. Perhaps it is time that we contributed more to our profession in terms of expanding the pool of knowledge that we work from. Case history papers are one way of achieving this. They need not be a fully developed thesis, but if they contain details of one small facet, then many facets when shared among all can lead to a significant contribution.

A case history paper describing the experience of one of our members in litigation involving geotechnical data in contract documents is presented on page 24. This is a topic arousing much current interest and accounts of other members' experiences in legal interpretation of this matter would be welcomed for the next issue.

It has been gratifying to receive several papers for this issue and a series of three discussion articles have been included. It is hoped that these will be thought provoking and will stimulate further response and discussion.

Seasons greetings to all our members with best wishes for a satisfying and prosperous New Year.

YOLANDA THORP
Editor

1. NEWS FROM THE MANAGEMENT SECRETARY

The Management Committee met on Friday 18 July and Friday 7 November.

2. 1987 IPENZ Conference

The 1987 IPENZ Conference is to be held in Christchurch in conjunction with the University of Canterbury School of Engineering centennial. With the conference being held from Sunday 10 May to Thursday 14 May this will mean a change of timing for the Society AGM. It is intended that the AGM will be held in conjunction with the conference.

Geomechanics Society has arranged for two full sessions on the Tuesday afternoon.

3. Geomechanics Lecture

Mr Les Oborn has accepted an invitation to present the 5th Geomechanics Lecture. The lecture title is Thoughts on the Evolution of Engineering Geology in New Zealand. The lecture will be presented at the IPENZ Conference on Tuesday 12 May 1987 and at other centres.

4. Symposium on Piled Foundations for Engineering Structures

The Symposium held on 5 - 6 September 1986 in Hamilton was most successful. A report is presented elsewhere.

5. Annual General Meeting

The AGM for the Society will be held during the 1987 IPENZ Conference late on Tuesday 12 May following the presentation of Society technical papers.

6. Special General Meeting

At the Special General Meeting held in Hamilton on the 5 September two items of business were dealt with. Dr Ken Birrell was elected to life membership of the Society and is the first person to achieve this status. A Certificate of Life Membership was presented to Dr Berrill at the Management Committee meeting on 7 November in Wellington.

The Chairman presented the inaugural Geomechanics Award to Bernard Hegan and Peter Millar for their paper "Investigations in Soft Rock Terrain" which was presented at the Alexandra Dams and Canals Symposium in 1984. The Society rules for the award were presented in the December 1985 Geomechanics News.

7. Nominations to 1987 Management Committee

Nominations for election of the Management Committee for 1987 were received from:

- R.D. Beetham
- J.R. A Gammon
- C.J. Graham
- W.J. Henderson
- D.N. Jennings
- A.J. Olsen
- N.W. Rogers
- M.J. D Stapleton
- Y.F. Thorp

Eight members are required and an election is to be held. Ballot papers have been circulated to members. The membership will be advised of the outcome of the election by post to enable the Management Committee to meet at the next meeting on Friday 13 February in Wellington.

Of the elected members of the 1986 committee Messrs B R Patterson and T J Kayes and G Ramsey are not standing for re-election. Dr Bell recently stepped down as International Vice President of IAEG and will no longer be ex officio on the committee.

8. New Members

The following new members are welcomed to the Society:

- | | | |
|---------------|-----------------|---------------|
| T. Matushka | K. Stevens) | |
| J.B. Berrill | S.A. Crawford) | |
| A.H. Nelson | K.G. Delanse) | |
| R.R. Thomson | D. Jacquet) | July 1986 |
| R.J. Mathew | C.B. Duncan) | |
| R.J. Peploe | B.W. Fairless) | |
| J.P. Ashby | K.S. Marriott) | |
| | | |
| S. Terzaghi | P.G. Marchant) | |
| P.N. Jacobson | M.J. Bloxham) | |
| G.A. Rogan | F.G. Henderson) | November 1986 |
| D.J. Convery | F.G. Henderson) | |
| G. Kneebone | L.E. Leach) | |
| I.F. McCahon | F.S. Patten) | |
| R.L. Williams | V.P. O'Connor) | |

9. 1988 ANZ Geomechanics Conference

The 1988 ANZ Geomechanics Conference is to be held in Sydney as part of the bicentennial celebration. An organising committee has been established under the chairmanship of Prof. Harry Poulos. Communication has been established between N.Z. Geomechanics Society and the organising committee. While Prof. Poulos was in Hamilton in September the opportunity was taken to discuss the proposed format for the conference. When details are established I will advise members.

The conference will include state of the art topics:

Prediction of Rock Excavatability
Evaluation of Geotechnical Performance
Geomechanical aspects of Earthquake Engineering

Presentation of technical papers is intended to be by general reports and poster sessions.

In addition the organising committee is planning to have some case study predictions. They are interested in suitable problems involving the prediction of geotechnical performance. If you have any suggestions of projects with good base investigations and performance data which are amenable to analysis and prediction which will be available by April 1988 please let me know. I will keep you informed as further information becomes available.

10. Science and Technology Review

Geomechanics Society did not meet the early submission date but is making a brief late submission to ensure members interests and concerns are registered.

11. Overdue Subscriptions

I have been advised that there are still 32 members who have yet to pay their 1986 subscriptions. If you are among these please act promptly as your support is important to the future of the Society.

12. International Society Meeting Representation

At the 7 November meeting the Management Committee discussed the importance of having a delegate to represent the Society at the Council meetings of the International Societies (ISSMFE, IAEG and ISRM). Often a Society member will advise the secretary of his/her intention to attend an international conference where a Council meeting is being held and offer to represent the Society. While it is recognised that it is difficult for a member to be fully aware of the implications of all Council business, direct representation at such meetings is highly desirable. Any short comings can be minimised by briefing from the appropriate vice chairman of the Society and liaison with the Australasian International Vice President at the meeting. A brief report would be appreciated on the events of the meeting.

In recognition of the value of representation at Council meetings the Management Committee has agreed to actively seek the assistance of the membership in achieving this. Some support may be made available to the nominated representative. The experience of attending an International Council meeting would be both interesting and stimulating. If you are attending an international conference and would like to represent NZGS please let the secretary know. As from the next Geomechanics News the venues of Council meetings will be listed.

I hope you have all had a busy and successful year. Merry Christmas and all the best for 1987.

DAVID N. JENNINGS
Management Secretary

INTERNATIONAL CONGRESS ON TUNNELS AND WATER

TECHNICAL PROGRAM

Subject "Tunnels and Water"

WORKING SESSIONS

- A. Water in Design
- B. Water in Construction
- C. Water in Exploitation
- D. Under Water Tunnels
- E. Hydraulic Tunnels

FREE COMMUNICATIONS

Free communications will be admitted concerning the above subjects.

Members intending to submit communications should send to Congress Secretary a resume of contents in any of the official languages of Congress: Spanish, English or French. Last admittance date being July 31 1987.

At the receipt of drafts authors will be informed of requirements for writing and publication afterwards as documents of Congress.

Final communications should arrive at the Congress Secretary before January 31 1988.

ASOCIACION ESPANOLA DE TUNELES Y OBRAS SUBTERRANEAS (AETOS)
Congreso Internacional ESPANA-88
Calle Martinez Izquierdo, 53, 2.º,3.
28028 MADRID (Espana)
Telef. 256 79 45. Telex 48.207-E.

INTERNATIONAL CONFERENCE ON CALCAREOUS SEDIMENTS

PERTH, WESTERN AUSTRALIA

15-18 MARCH 1988

THE CONFERENCE

This Conference is being organised jointly by The Institution of Engineers, Australia and the International Society for Soil Mechanics and Foundation Engineering.

The catalyst for this Conference has been the decision by Woodside Offshore Petroleum Pty Ltd to make generally available their experience gained from the studies during the extensive geotechnical investigations carried out on the North West Shelf of Western Australia.

This call for papers is made to attract papers from relevant studies on these materials in other areas. It is intended that the conference establish a forum for the presentation of and discussion on the challenging problems of designing and building structures on these materials.

Expressions of interest are also sought from organisations or individuals who wish to consider participating in an exhibition associated with the Conference.

PAPERS

Intending authors are invited to submit the titles of proposed papers together with a brief synopsis of not more than 300 words outlining the aims, contents and conclusions of their papers.

Papers in the following categories will be particularly welcome:

- Fabric, structure and formation of calcareous deposits
- Sampling and laboratory testing, including laboratory model tests
- Insitu testing, such as CPT and pressuremeter
- Large scale insitu testing
- Foundation design and analysis
- Construction techniques in calcareous deposits.

Final papers should be limited to 6000 words. Manuscript including text, tables and diagrams to be in camera ready copy format as designated by The Institution of Engineers, Australia.

DEADLINES

Intending authors should note the following deadline dates:

Receipt of 300 word synopses	20 December 1986
Notification of acceptance of synopses	30 March 1987
Receipt of full text for final review	30 June 1987

Synopses should be accompanied by a statement of the author's intention to attend the Conference.

Papers received after June 1987 will not be included in the Conference. A final program will be available in October 1987.

ENQUIRIES

All correspondence relating to the Conference and proposals for papers should be addressed to:

The Conference Manager
International Conference on Calcareous Sediments
The Institution of Engineers, Australia
11 National Circuit
Barton ACT 2600
Telex: AA62758
Facsimile: (062) 73 1488
Telephone: (062) 73 3633
Telegrams: ENJOAUST CANBERRA

FORTHCOMING CONFERENCES

- | | |
|------------------------------|--|
| 1 - 9 December 1986 | - Computer Aided Design and Monitoring in Geotechnical Engineering, Bangkok |
| 15 - 20 December 1986 | - The Role of Geology in Urban Development, Geological Society of Hong Kong. |
| 18 - 20 December 1986 | - Indian Geotechnical Conference |
| 23 - 27 March 1987 | - Tunnel Australia 1987 - 13th ITA General Assembly, Melbourne |
| 3 - 7 May 1987 | - The Engineering Geological Environment in Mountainous Areas, Beijing |
| 10 - 13 May 1987 | - 1987 IPENZ Conference, Christchurch |
| 20 - 24 July 1987 | - 8th Asian Regional Conference on SM & FE, Kyoto |
| 1 - 12 August 1987 | - ANZ SLIDE 87 - The Fifth International Conference and Field Workshop on Landslides |
| 12 - 13 August 1987 | - Geotechnical Engineering of Soft Soils, Mexico City |
| 16 - 21 August 1987 | - 8th Pan American Conference on SM & FE, Cartagena |
| 26 - 29 August 1987 | - International Congress on the Geology, Structure, Mineralization and Economics of the Pacific Rim, Queensland. |
| 30 August - 3 September 1987 | - ISRM 6th International Congress on Rock Mechanics, Montreal |
| 31 August - 4 September 1987 | - Groundwater Effects in Geotechnical Engineering, Dublin |
| 1 - 3 December 1987 | - Sixth International Conference on Expansive Soils, New Delhi |
| 15 - 18 March 1988 | - International Conference on Calcareous Sediments, Perth |
| 1 - 5 June 1988 | - Second International Conference on Case Histories in Geotechnical Engineering, St. Louis |
| June 1988 | - International Congress on Tunnels and Water, Madrid |
| 10 - 15 July 1988 | - 4th International Symposium on Landslides, Lausanne |
| 22 - 26 August 1988 | - Fifth ANZ Geomechanics Conference, Sydney |

PUBLICATIONS OF THE SOCIETY

The following publications of the Society are available:

(a) From the Secretary, IPENZ, P.O. Box 12-241, Wellington North:

- Proceedings of the Palmerston North Symposium **Geomechanics in Urban Planning**", April 1981. Price \$20.00
- **"Stability of House Sites and foundations - Advice to Prospective House and Section Owners"**. (Published for the Earthquake and War Damage Commission). Price \$0.50.
- Proceedings of the Third Australia-New Zealand Conference on Geomechanics, Wellington, May 1980. Price \$20.00 for the three volume set to members, \$30.00 to non-members.
- Proceedings of the Second Australia-New Zealand Conference on Geomechanics, Brisbane, July 1975. Price \$25.00.
- Proceedings of the Wanganui Symposium **"Using Geomechanics in Foundation Engineering"**, September 1972. Price \$8.00 to members, \$10.00 to non-members.
- Proceedings of the Christchurch Symposium **"New Zealand Practices in Site Investigations for Building Foundations"**, August 1969. The last copies of a limited reprinting are available at \$8.00 to members, \$10.00 to non-members.
- Proceedings of the Alexandra Symposium **"Engineering for Dams and Canals"**, November 1983. Price \$40.00 to members, \$50.00 to non-members.
- Copies of all back-issues of **"New Zealand Geomechanics News"** are available to members at a nominal price of 50cents per copy plus 50 cents post and packaging per order.
- The following back-issues of the IAEG Bulletin are available. Price \$3.00 to members: volumes 15, 24, 26, 27.

(b) From Government Bookshops and the Secretary IPENZ:

- **"Slope Stability in Urban Development"** (DSIR Information Series No. 122). Price \$2.00. (Also available from Government Bookshops).

The following publications of the Society have been sold out:

- Proceedings of the Nelson Symposium **"Stability of Slopes in Natural Ground"**, 1974.
- Proceedings of the Wellington Workshop **"Lateral Earth Pressures and Retaining Wall Design"**, 1974.
- Proceedings of the Hamilton Symposium **"Tunnelling in New Zealand"**, November 1977.

G. RAMSAY
Publications Officer

REPORT FROM THE ENGINEERING GEOLOGY VICE-CHAIRMAN

1. IAEG Publications

Bulletin No. 32 and Newsletter No. 12 were the latest publications received from IAEG - these were reported in N.Z. Geomechanics news No. 32.

All current members of IAEG should have received a personal copy of the 1985 IAEG Membership List. Please notify the Engineering Geology Vice-Chairman if you have not received your copy.

2. 5th IAEG Buenos Aires Congress October 1986

N.Z. authors have submitted 5 papers for presentation at the Congress the titles of which were reported in N.Z. Geomechanics News No. 32. Two of the authors D.H. Bell and G.T. Hancox will be attending the Congress.

David Bell will be attending his final IAEG Council meeting as Australasian Vice-President, and will also be representing the N.Z. National Group during the meeting.

A report on the business discussed at the Council meeting will be included in the next issue of Geomechanics News.

BRIAN PATERSON
Vice-Chairman Engineering Geology

REPORT FROM THE SOIL MECHANICS VICE-CHAIRMAN

A listing of 25 Technical Committees and their terms of references set up by the International Society of Soil Mechanics and Foundation Engineering is available and will be published in the next issue of the Geomechanics News. In the meantime anyone wishing to obtain further information on these committees should write to the Society.

N.S. LUXFORD

REPORT FROM THE ROCK MECHANICS VICE CHAIRMAN

1. **6th ISRM Congress Montreal 1987**

One paper has been proposed by N.Z. authors for this conference. The abstract for the paper is:

DETERMINATION OF ELASTIC CONSTANTS FOR HIGHLY
ANISOTROPIC (SCHISTOSE) ROCKS AND
THEIR APPLICATION FOR MODELLING

S.A.L. Read+ N.D. Perrin+ and I.R. Brown*

The results from laboratory testing of highly anisotropic (schistose) rocks to determine their uniaxial compressive strength are influenced by the orientation of the foliation with respect to the direction of load application. The Tangent Moduli of Elasticity values determined are not greatly affected by either foliation orientation or the positioning of the deformation measuring points relative to it. The Poisson's Ratio values determined do appear to be affected by these considerations. A modification to the positioning of deformation measuring points is proposed, different to those in published test methods (e.g. ISRM).

The laboratory test results are used to show that schist can be described as a transversely isotropic rock defined by four deformability parameters. The implications for numerical modelling of schist are discussed.

+ NZ Geological Survey, Lower Hutt, New Zealand

* I.R. Brown and Associates, Wellington, New Zealand

2. **ISRM Council Meeting Sweden August 1986**

Dr K. Mills attended this meeting as the New Zealand Geomechanics Society representative. A report on the meeting is expected from Dr Mills and significant matters will be reported in the next issue of Geomechanics News.

G. RAMSAY
Vice-Chairman Rock Mechanics

INTERNATIONAL TUNNELLING ASSOCIATION

We note that the MWD have made arrangements to receive copies of the tunnelling Association of Canada's newsletter "TAC News" and also of the Australian Underground Construction & Tunnelling Association's newsletter. These publications will be held in the MWD library.

TWELFTH ANNUAL MEETING - FIRENZE 1986 - PRESS RELEASE

The International Tunnelling Association held its twelfth annual meeting in Firenze from the 8th to 11th June 1986 in conjunction with the International Congress on Large Underground Openings. The meeting was attended by representatives, delegates, observers and working group members of 26 to 33 member nations of the Association and 2 representatives of international Associations.

MEMBER NATIONS REPRESENTED

South Africa, Algeria, Federal Republic of Germany, Australia, Austria, Belgium, Canada, Columbia, Egypt, Spain, United States of America, Finland, France, Hungary, Iceland, Italy, Japan, Mexico, Netherlands, Norway, Poland, United Kingdom, Sweden, Switzerland, Czechoslovakia and Venezuela.

MEMBER NATIONS NOT REPRESENTED

Brazil, People's Republic of China, Republic of Korea, Denmark, India, Irak and New Zealand.

ORGANISATION

The Association has registered the adhesion of 30 affiliate members: 11 corporate members and 19 individual, which reached a total of 89 members (42 corporate members and 47 individual members);

THE NEW EXECUTIVE COUNCIL IS:

E. BROCH	NORWAY	President	until 1989
J.K. LEMLEY	U.S.A.	Past President	until 1992
G. GIRNAU	F.R.G.	Past President	until 1989
A.M. MUIR WOOD	U.K.	Honorary President	
V. ROISIN	BELGIUM	Vice President	until 1989
C. KIRKLAND	U.K.	Vice President	until 1989
E. TEGIDO NOGUES	SPAIN		until 1987
Y. ONOUCHI	JAPAN		until 1987
GAO QUQING	P.R. CHINA		until 1988
B. PIGORINI	ITALY		until 1988
C. BERENQUIER		Secretary General	until 1987
		(Acting As)	

The Association decided to promote the activities of the working group; as a first step, S.T.U.V.A. (Germany) will act as a "Technical Support Institute" for two working groups: "Health and Safety in Work" and "Maintenance and Repair of Underground Structures". The help can be extended the next year to other working groups.

The activities of the Association will be from now published in the new journal "Tunnelling and Underground Space Technology", created by merging "Underground Space" and "Advance in Tunnelling and Subsurface Use".

The new Journal, published by Pergamon Press is prepared by the Underground Space Centre of University of Minnesota and it is the official journal of I.T.A.

The next I.T.A. annual meeting will be held in Melbourne (Australia) from 23rd to 27th March 1987 in conjunction with the VI Australian Conference of Tunnelling: "Tunnel in Australia BORE OR BLAST", organized by the Australian Group of I.T.A.

I.T.A. Members Nations are requested to participate to the Conference.

WORKING GROUPS

Eight of the ten Working Groups held working sessions during this Congress; Working Group STANDARDISATION did not meet and Working Group SEISMIC EFFECTS finished its work in 1985.

WORKING GROUP CONTRACTUAL SHARING OF RISKS

J. VAN DER LINDEN (Belgique representing W.O. SALTER (U.S.A.))

The meeting of the working group was attended by 14 participants representing 11 countries. On the resignation of the animateur MR J. LEENEY (U.K.), the group elected Mr W.O. Salter (U.S.A.) as animateur and Mr J. Van Der Linden (Belgium) as vice animateur. The participants reviewed seven propositions of which three were presented to the General Assembly for adoption as formal recommendations of I.T.A. These are: "Rights-of-Way and permits" (XVII), "Provision of Plant, Equipment, Services and Materials by the Owner" (XVIII), and "Alternative Tenders" (XIX). The remaining four propositions were continued for further discussion, as follows: "Measurements Problems in Rock", "Protection of Project Surrounds", "Measurement Problems Related to Water" and "Role of the Engineer". As important changes have occurred in the past year, the original recommendation of the "Role of the Engineer" is to be postponed for further investigation in member countries.

WORKING GROUP RESEARCH J.F. BOUGARD (FRANCE)

The meeting of the working group gathered 15 persons representing 11 nations. Five other nations have sent contributions. Five studies are underway:

- Tunnel boring machines in hard rock: new questionnaires will be sent to member nations.
- Slurry shield and earth pressure balance shield: a draft of the final report will be submitted at the next annual meeting.
- Water problems during and after the construction of tunnels.
- Single shell in situ concrete lining.
- Noise and vibrations: an intermediary report will be presented in Melbourne meeting.

Besides, a general survey related to current and future national activities in terms of research is being made.

WORKING GROUP MAINTENANCE AND REPAIR OF UNDERGROUND STRUCTURES

A.J. HOWARD (U.K.)

Fourteen persons attended the Group meeting, representing ten nations. the group's present study topic is "the corrosive effects of water and other liquids, entering tunnels during their working life". A number of case histories had been prepared and there was a full discussion of these and of similar problems known to those attending.

During the coming year the group will continue to seek case histories of water problems. The group will be supported by the West German research agency S.T.U.V.A., who will study the case histories and prepare a report for discussion at the next meeting.

WORKING GROUP CATALOGUE OF WORKS. S. KUWAHARA

Specific information of tunnel projects in different stages (i.e. completed, in progress, under planning) were reported in total 816 numbers of projects by member nations during 1985.

This information has been memorized in the computer as original data file.

Applying the computer data processing, the participants of the working group are able to classify whole the tunnel projects by the kind of tunnel, by the project scale, by the construction method and by the cost etc. In order to grasp the world wide trend and tendency of the tunnel construction activities, the participants discussed modification and improvement of the data processing programme. Though the co-operation of member nations, more useful output data and figures will be obtained by the next I.T.A. congress.

WORKING GROUP HEALTH AND SAFETY IN WORK N. LAGRANGE (BELGIQUE) representing N. KRIGE (SOUTH AFRICA)

The recommendations concerning the "Use of Tunnel boring Machines and Road Headers", "Dealing with Water" and "Underground Communications" have been discussed. The final project will be proposed at the end of 1986 to I.T.A. In all, the group already approved recommendations for 11 different aspects concerning underground works. The group is glad to have, next year the support of the West German research agency of S.T.U.V.A.

WORKING GROUP SUBSURFACE PLANNING M. BARKER (U.S.A.)

The working group on Subsurface Planning is dedicated to the wise use and management of underground space. By publishing case studies the working group hopes to encourage the use of the subsurface. In Florence eight national delegation had representation at the W.G. meeting. Papers presented covered underground utilities, nuclear research laboratories, and transportation facilities. The working group decided to make a special effort to study the legal implications of underground use.

WORKING GROUP COST BENEFIT FOR UNDERGROUND URBAN PUBLIC TRANSPORTATION
M. BLENNEMANN (F.R.G.)

The main task of the Group "Cost Benefits for Underground Urban Public Transportation" in the Florence meeting was to discuss methods of cost benefit analyses for transport facilities. It was shown in contribution from several countries, that there exists great differences in how this problem is handled in various countries.

The group will prepare a report with descriptions of different methods and a comparison of the various approvals. An evolution of the reports will show the necessity of cost benefit methods on the one hand but the limits of their application on the other.

WORKING GROUP GENERAL APPROACHES TO THE DESIGN OF TUNNEL H. DUDDECK

A first draft of the "Guide Lines for the Design of Tunnels" has been presented by the animateur of the working group. It has been extensively discussed at the two meetings. It covers a general outline of the designing procedure, which should be a consistent unity of the evaluation of site investigation of the analysis by appropriate design models, and of the interpretation of in situ monitoring. The second draft should be ready late this year, so that publication of the ITA Guidelines may be possible after the next year meeting in Melbourne.

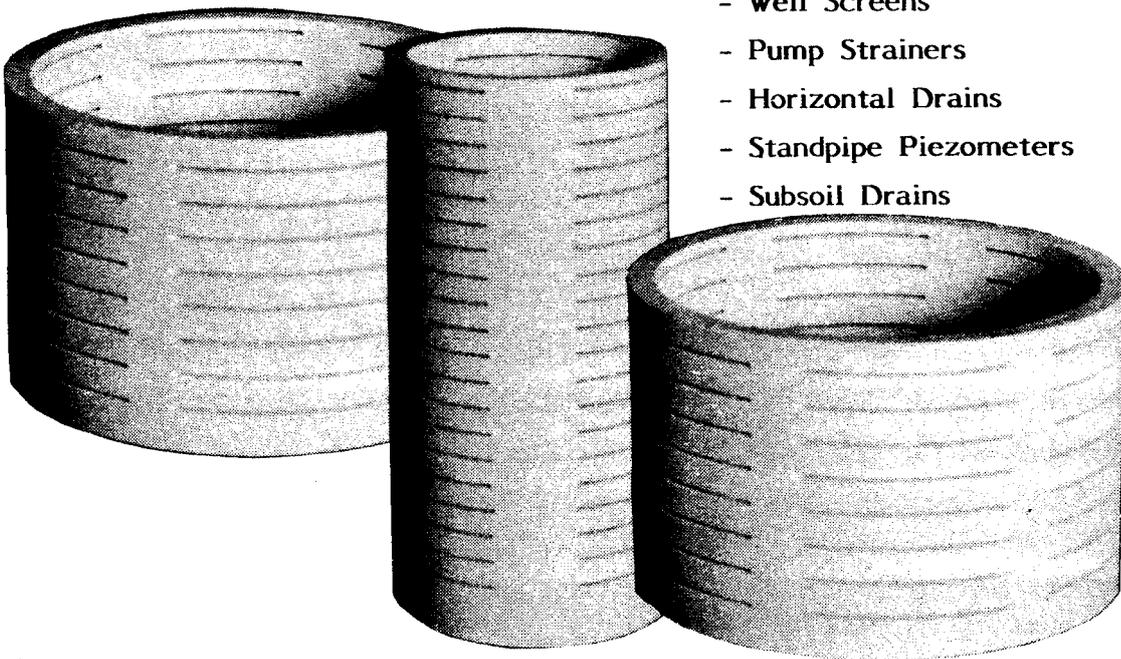


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LOCAL GROUP ACTIVITIES

OTAGO SOUTHLAND GROUP

A meeting attended by 30 people was held on 14 July on the "Cromwell Gorge Roding" when speakers addressed various aspects of the major works being undertaken as part of the Clutha River development and in particular as a consequence of the construction of the Clyde Dam.

The speakers were:-

Roydon Thomson	- "Geological Setting and Preliminary Investigations"
Graham Salt	- "Establishing Strength Parameters"
Jeff Bryant	- "Design Investigations and Construction"
Murray Petherick	- "Contractural Aspects"

The speakers gave a most interesting insight into the problems encountered on the project which is constructed in difficult terrain. Many of the experiences highlighted the need for adequate investigations before design and construction, and also demonstrated that in variable country even extensive investigations do not always reveal all the problems. These factors and others resulted in some interesting construction and contractural problems from which a number of lessons have been learnt.

AUCKLAND GROUP ACTIVITIES

There have been no meetings during the second half of the year for the Auckland Group but an end of year meeting is planned for the 20th November 1986.

WELLINGTON AND CHRISTCHURCH GROUP ACTIVITIES

There have been no meetings during the second half of the year for the Wellington and Christchurch groups.

Instrumentation

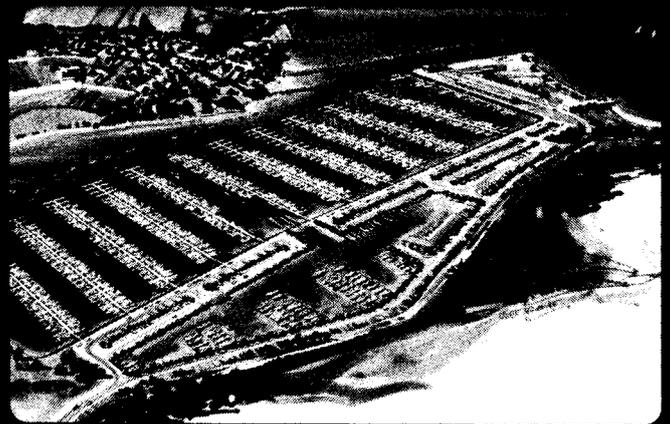
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Below: Aquatic Centre — Rotorua (Under construction)



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- ★ WATER SAMPLERS
- ★ TEMPERATURE SENSORS
- ★ TRANSDUCER METERS
- ★ DIP METERS
- ★ TENSIO METERS — SOIL SUCTION PROBES

Non-destructive testing

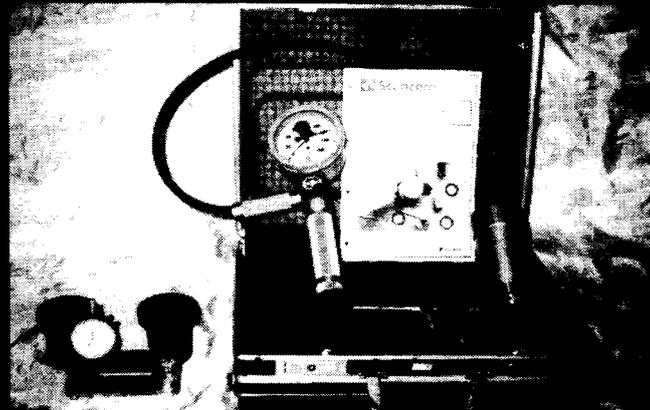
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*Above: Troxler Gauge in use on motorway construction
Left: Scancem Break-Off Tester*

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SYMPOSIUM ON PILE FOUNDATIONS FOR ENGINEERING STRUCTURES

Hamilton, 5 - 6 September 1986

A most successful Symposium was held at the University of Waikato, attended by 128 registrants (including four from Australia). Papers by selected authors addressed foundation evaluation, design, testing and included a number of case histories. (Refer NZ GEOMECHANICS NEWS No. 32, July 1986).

Professor Henry Poulos of Sydney University set the tone of the Symposium with an authoritative review of some of the recent advances in the analysis and design of pile foundations. In addition to this address and other contributions to discussion, Professor Poulos provided a perspective of the Symposium in a concluding overview, which is reproduced below. The organising committee comprising Peter McGibbon, Peter Wilcox, Peter North and Mark Mitchell, under the chairmanship of Dave Jennings, are to be congratulated on a most stimulating occasion, including a memorable and uproarious after dinner address by Pat Shannon. Proceedings of the Symposium will be available from IPENZ in the new year.

"I have found these last two days to be extremely interesting and out of these two days I sense three major themes that came out.

The first was in relation to your Pile Foundation Design Notes, the famed CDP 812/B, which I was unaware of, but I note with interest, was very widely used. It was interesting to hear people's comments on this and to hear of suggestions where this particular document might be modified.

I mentioned yesterday the presence of the Australian Standard Piling Code and of another pile code that has been brought out by the state road authorities in Australia (NAASRA). I think that if your Society, or the MWD, is in a position to modify this document, they could perhaps look at those other documents in so doing. I believe that there is scope for placing less reliance on purely empirical methods and perhaps more reliance on methods that have a sounder analytical background. I guess I have been pushing this barrow for some time, but you can't get away from empiricism, it is a fact of life in geotechnical engineering. However, I believe we can remove one facet of the empiricism by using at least well-founded theory and leaving the empiricism to the question of getting the parameters. So if there was to be some modification in that set of design notes, I hope it might go that way.

The second major theme that came through was related to pile load testing. It became apparent to me that this was a fairly rare event in this country, but when it was done, it was extremely useful. I think of the relatively small amount of data that is available on rock socketed piles, for example, the Mick Pender's analysis indicated how useful it could be. It's not cheap, certainly, but I think that we must remember that we are professional engineers, and too often I suspect we tend to pander to the whims of clients who don't want to spend money. We say "OK, we'll try and do what we can and give you a cheap job". As professionals we should try and resist this, and where appropriate we should insist on doing load testing where the job is sufficiently large to carry the cost.

I would also support what Peter Millar said in terms of instrumenting piles. For a relatively small marginal cost increase, one can get a great deal of information about the way in which load is transferred to the soil through the pile by instrumenting it. If you propose to use a pile load test to back figure some parameters or subsequent pile designs, instrumenting it gives you more measurements to fit to the theory. The more measurements you have, the less assumptions you have to make, the closer you may get to reality. I do believe that certainly instrumenting piles, if only to get the load distribution along the pile, is an extremely important addendum and one that should be seriously considered.

The third major theme of this Symposium, I felt, was the presentation of a large number of case histories in which there was a wealth of experience presented. It was, I think, a very useful part of this Symposium and highlighted the need to document the experience of engineers in this area. I think in almost all cases there was, in fact, one factor lacking and that is the question of instrumentation - the measurement of the performance of the structure after the design had been done. I think without instrumentation, without evaluation, of how the structure or how the foundation has performed, we will still be in a difficult situation in trying to evaluate how well we can actually design and analyse pile foundations.

So they were the three major themes that I felt came through the Conference.

There were other aspects I think that are worth mentioning. I think Don Taylor's tabulation of the relative costs of foundations in proportion to the total project cost was very useful. I think the discussion of whether or not to rake piles was extremely valuable. I, myself, have some serious doubt as to whether rake piles are really worth the trouble. They cost more to construct, they are more of a hassle, they are much more prone to damage by soil movements, and they do develop much larger bending moments at the pile cap. I'm not totally convinced that the increase in the lateral stiffness from rake piles is all that great. There was some emphasis also on rock socketted piles, and again I think Mick Pender brought out the uncertainty as to whether or not we are dealing with drained or undrained capacity. There was certainly a very strong inference regarding the importance of roughness of the sockets, and I think there were one or two cases that were represented that indicated that socket roughness is certainly a major factor. It's not a new finding, but it's nice to know it is being repeated here as well as elsewhere.

There was another interesting feature of this morning's discussions, I think, with the Hiley Formula and wave equation analysis being discussed, and I think it is true to say that wave equation analysis in some parts of the world is certainly now a standard type of analysis that people do in trying to assess drivability. There is some suspicion about the ability of any dynamic analysis, whether it be a driving formula or a wave equation analysis, to predict load capacity. Is it adequate to drive to the design penetration? The use of that sort of analysis is extremely useful. I should just mention that you can buy packages such as WEEP, off the shelf, but it is not a particularly difficult thing to programme. If one is concerned about the need to have a large main

frame, there is a gentleman in Israel who has programmed it for an HP calculator. If you have got sufficient time to wait a few minutes, it beavers away and gives you the results on your little hand-held calculator. So I don't think the excuse perhaps that it's only available on main frames is really a valid one.

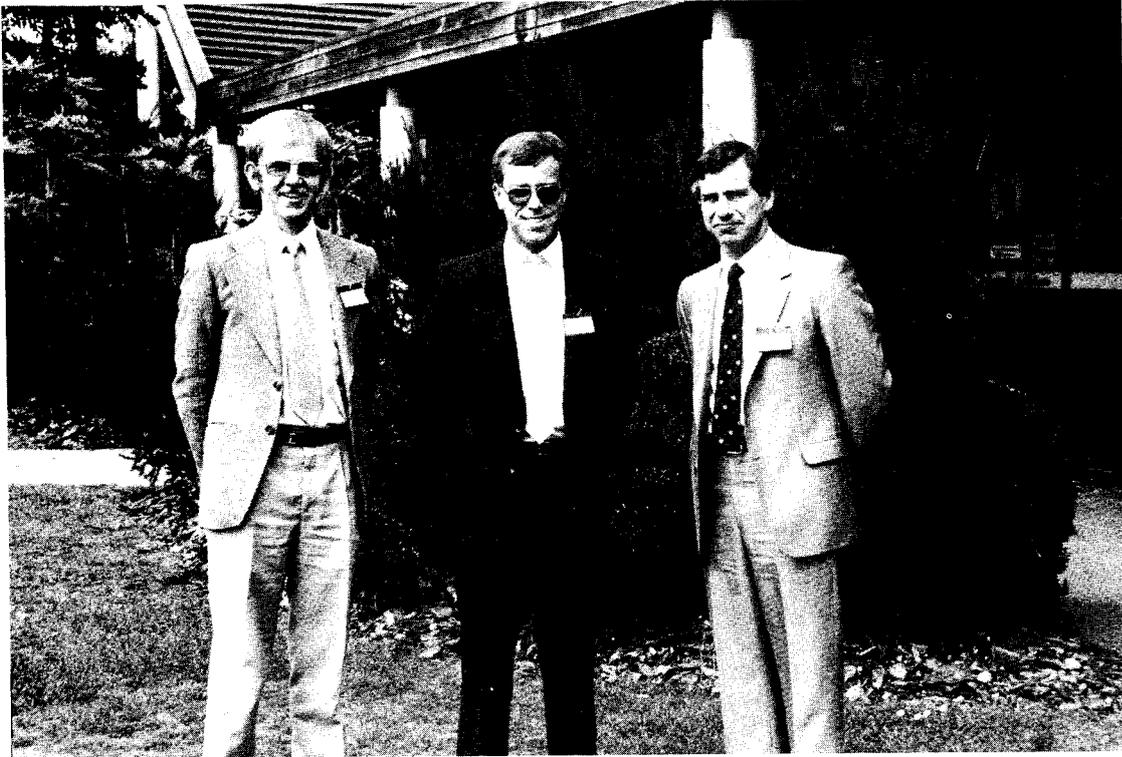
The final thing I might mention is the question of tower foundations. This came out in this afternoon's discussion. Certainly an important part of geotechnical engineering and one about which relatively little is known. I noticed with some interest the sense of uncertainty as to how we should calculate the uplift capacity of piles. This is not only a problem here, it is a problem elsewhere. You may be interested to know that in the United States there are similar concerns about how to design tower foundations. Over there of course, they think in terms of large amounts of money, and I know that at Cornell University, Professor Kulhawy has just got a \$2.5 million contract to look at transmission tower foundations, so I guess something will come out of that in the next 2 or 3 years. I certainly would hope so, and perhaps that will filter through to Australia and New Zealand, and we will all be in a better situation to design these types of foundations.

So in conclusion, ladies and gentlemen, I feel that this has been an excellent Symposium. The large attendance here is proof of the value of such a Symposium. I think specialised topics discussed in this way are of much more value in a technical sense than are the perhaps much larger panned subject conferences that we've been used to having. This Symposium has indicated certain areas where concentrated effort is required to advance the state of practice in pile design. The three major areas that I mentioned before are areas where some progress can be made and these are your pile design notes, the question of increasing the amount of pile load testing, and the reporting of case histories including performance data.

I've been very pleased to be a part of this Symposium. I've learnt much about piling in New Zealand and piling in general, and I would certainly like to thank the organisers for inviting me here. I'm almost embarrassed to accept their hospitality in view of the fact that in about an hour or so the Bledisloe Cup will be going back to Australis!"

T.J. KAYES
Director Tonkin & Taylor

PHOTOS FROM THE SYMPOSIUM ON PILED FOUNDATIONS



From Left Dave Jennings - Symposium Chairman, Andy Olsen - Chairman NZGS,
Prof. Harry Poulos - Keynote Speaker



Close encounters with a dutch cone penetrometer during the field demonstration

1987 IPENZ CONFERENCE

The 1987 IPENZ Conference, to be held in Christchurch in May, promises to be an important event for the Society. Members are urged to support this event by attending the sessions organised jointly with the National Society for Earthquake Engineering as well as other activities.

An effort has been made to organise a series of papers on a theme which is currently being developed in N.Z., and hence should be of interest to practising engineers and geologists. A number of other activities including presentation of the Geomechanics Lecture are also being arranged on the same day as the technical sessions. These are outlined below:

A. TECHNICAL SESSIONS TUESDAY 12th MAY

1) 2.00 p.m. - 3.15 p.m.

- (a) Dr Warwick D. Smith, Seismological Observatory, DSIR
"Earthquake Hazard Estimates: Where are we going?"
- (b) Mr Kelvin R. Berryman, N.Z. Geological Survey, DSIR and
Ms Sarah Beanland
"The Nature and Rate of Present-day Tectonism in New Zealand"

2) 3.45 p.m. - 5.00 p.m.

- (c) Dr Ian R. Brown, Ian R. Brown Associates Ltd and
Mr Graham T. Hancox, N.Z. Geological Survey, DSIR
"The Evaluation of Earthquake Hazards"
- (d) Dr John B. Berrill, Department of Civil Engineering, University of
Canterbury
"Implementation of Seismic Hazard Evaluation in Design Codes"

B. N.Z. GEOMECHANICS SOCIETY AGM - 5.00 p.m.

C. BUFFET DINNER 6.30 p.m. - 7.45 p.m.

Jointly sponsored by Geomechanics Society, National Society for Earthquake Engineering, Society on Large Dams and Technical Group on Water.

D. GEOMECHANICS LECTURE 8.00 p.m.

Mr L.E. Oborn (consultant - previously Chief Engineering Geologist, N.Z. Geological Survey, DSIR
"Thoughts on the Evolution of Engineering Geology in New Zealand"

Further details about the Geomechanics Lecture are presented in this issue of Geomechanics News. Additional information will also be included in the IPENZ Conference Supplement which will accompany the February issue of New Zealand Engineering.

BRIAN PATERSON - Local Convenor, Christchurch

5TH GEOMECHANICS LECTURE

The 5th Geomechanics Lecture entitled:

"Thoughts on the Evolution of Engineering Geology in New Zealand"

will be presented by Mr Les Oborn in the School of Engineering, University of Canterbury on Tuesday 12th May 1987 at 8 p.m.

Mr Oborn is a consultant engineering geologist based in Lower Hutt who, before retirement in 1983, held the position of Chief Engineering Geologist with NZ Geological Survey, DSIR. As a founding member of engineering geology in New Zealand, he is particularly well qualified to speak on this subject.

During 33 years employment with DSIR Mr Oborn carried out a wide range of activities including geological mapping and groundwater investigations. However he is better known for his later involvement in engineering geology during which he worked on a number of major engineering projects including the Upper Waitaki Hydro Development, Lyttelton road tunnel and Tongariro Power Development.

He was responsible for the formation of the Engineering Geology Section of NZ Geological Survey. In addition to his responsibilities as Chief Engineering Geologist, he conducted various research activities, notably the introduction of seismotectonics to the New Zealand scene, and in particular its application to major engineering projects.

In his address Mr Oborn will trace the development of engineering geology in New Zealand, and will then lead on to a general discussion of seismotectonics.

This address will be of wide interest to the general public as well as to the engineering and geological professions.

B.R. PATERSON

GEOTECHNICAL DATA IN CONTRACT DOCUMENTS - A CASE RECORD

Laurie Wesley - Senior Lecturer, Auckland University

The following case record provides some interesting insight into the question of geotechnical data supplied to contractors as part of tender documents, and the extent to which such data should, or should not be interpreted by the Engineer. The writer was recently asked to appear as a witness (for the defendant) in a contract arbitration hearing where the contractor was claiming a large "extra" because of unforeseeable soil and meteorological conditions.

The case involved the design and construction of a hydro-electric project in the island state of Mauritius. The project was designed by the Snowy Mountains Engineering Corporation (SMEC) of Australia for the Mauritius government, and was constructed by a joint Italian-French contracting consortium (Cogefar/Spie Batignolles). The project was successfully completed, and in due course the contractor submitted a huge claim for extra costs, which among other things included a large "Clause 12" claim, that is a claim on the ground that he encountered soil and weather conditions "which could not have been reasonably foreseen by an experienced contractor.

The soil at the site consisted predominantly of a clay or silty clay residual soil, derived from the weathering (in wet tropical conditions) of basalt rocks. The principal works items involving this soil were the excavation of a large cut to form the spillway bay, and the transport and compaction of the soil to form an earth dam. The main grounds for the contractors claim can be summarised as:

- (a) the soil was of high sensitivity, and suffered severe loss of strength when handled or trafficked, making earthwork operations very difficult
- (b) the soil required considerable drying to bring it to a suitable condition for compaction, and the climate of Mauritius was too wet to allow such drying to take place.

The soils reports prepared by SMEC, and supplied to tenderers, were fairly typical of reports prepared for such projects; they were prepared primarily for the general purpose of designing the project, and the test data they contained was relevant to the designers needs. Because of the "stages" involved in the planning of such projects (prefeasibility, feasibility, design, etc) several reports were prepared and the "data" was "scattered" throughout the several reports.

My "brief" was essentially to read the reports and to prepare and present a statement as to how I would expect the soil to behave in the light of the data given in the reports.

The essential property of the soil which could have resulted in the contractors' alleged difficulties was its sensitivity, and hence the most important issue appeared to be whether this property and its ramifications could be deduced from the soils reports. It is clear from the data given in the reports that some of the soil at least was highly sensitive, because of natural water content values equal to, or above, the liquid limit. The term "sensitive" is not used in the text, and no values for sensitivity are given, but the likely "sensitive" behaviour is clearly described in the report text. It is stated for example that;

- "the most striking feature of all the laboratory tests is their high field moisture content in relation to their liquid limit"

- "the soil breaks down in the hands and exudes water when squeezed"
- "water held in the relic structure is released on crushing the soil to turn it into a soft plastic state"

It seemed to me, from the above statements, that any geotechnical engineer or contractor with reasonable experience would have recognised the difficulties likely to be encountered in handling with compacting such a soil. However, there is no mention in the reports of likely handling difficulties with the soils, and no warning is given to tenders to alert them of the danger of such difficulties.

In his claim documentation, the contractor presents a repeated argument to the effect that while a well qualified and experienced geotechnical engineer might have been able to foresee how the soil would behave, it was unreasonable to expect a contractor to be able to do the same, because he would lack the necessary expertise. One of the contractors' technical witnesses (an American) supported this argument on the basis of his knowledge of American contractors.

So how did the arbitration tribunal rule? Firstly, they state a number of "findings", the most significant of which can be summarised as follows:

- (a) the experts called by both parties were in the end generally agreed that all of the relevant information necessary to assess the behaviour of the soils was contained in the reports, though not necessarily presented in a clear and concise manner
- (b) the descriptive sections of the report texts made clear the expected properties of the soil, especially with respect to loss of strength when remoulded
- (c) the contractors invited to tender for the job were specifically pre-selected, and were all well versed and experienced in the type of work involved. They could be expected to have on their staff skilled technical personnel of similar expertise and experience to those employed by Consultants.

The tribunal then ruled that:

- (a) An experienced contractor could be expected to exhibit the same level of expertise as an engineer (i.e. Consultant) and therefore be able to foresee the conditions encountered to the same extent that an engineer would foresee them.
- (b) In this case "the physical conditions encountered were in fact more difficult than were foreseen by the engineer; the level of difficulty was underestimated by 20%".
- (c) "It follows from the above that the claimants are therefore entitled to be reimbursed at the rate of 20% of their full additional expenses to be assessed in due course".

To the writer the judgement seems somewhat puzzling, though I understand that the defending solicitors were quite pleased with it. There doesn't seem to be much logic between (b) and (c) above, and there is the very difficult question of how the claimants "additional expenses" are to be determined. It was understood at the time of the above hearing that this would be a matter for a further hearing.

The important lesson from the ruling is that the engineer was "judged" not to have accurately foreseen the conditions encountered during construction. How could he have avoided this judgement?

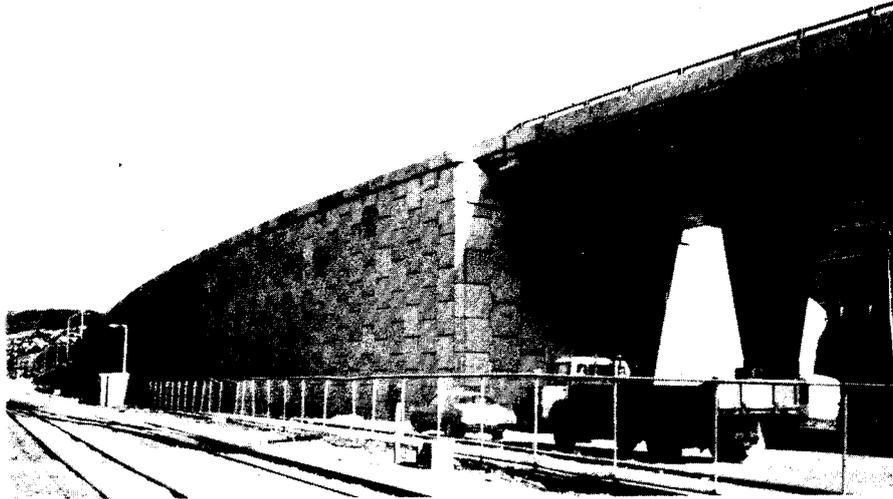
It seems to me that the only way he might have avoided the judgement would have been to explicitly state in the soils reports (or elsewhere in the tender documents) that the contractor could expect difficulties in excavating and handling the soil, as well as in drying it for compaction. In other words, he should go further than simply presenting the data on soil properties; he should "interpret" it as far as its relevance to the contractor is concerned, and not leave this to the contractor. It would appear to me that had explicit statements been made in this case about handling difficulties etc., it would have been very difficult for the tribunal to find in the way it did.

This need for "interpretation" goes against a commonly held view that the engineers role is simply to obtain the data and leave it to tenderers to interpret it.

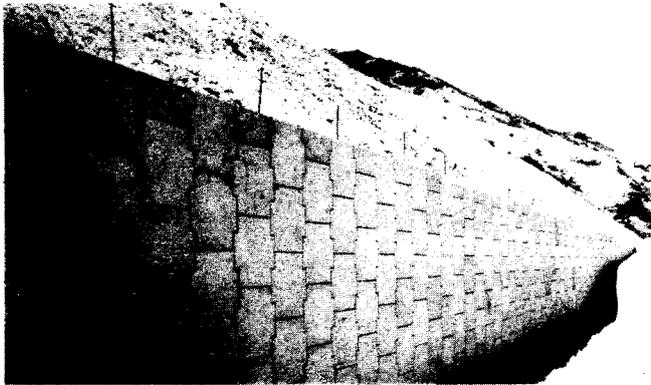
As a matter of interest, the arbitration hearing was held in London over a period of two weeks in February 1986. The tribunal consisted of a panel of three people, a Swiss Lawyer, a French Lawyer, and a British Civil Engineer; the hearings were conducted under the jurisdiction of the International Chamber of Commerce Court of Arbitration. Apart from preparation of statements some six months prior to the hearing, the writers role involved 1 1/2 hours in the witness box presenting evidence, and about 3 hours under cross-examination.

A final comment which should be made relates to the manner in which soils data is given to tenderers. In this case, the reports given to tenderers were those prepared during the feasibility and design stage and were intended primarily for the project designers. As already mentioned the data was scattered throughout several volumes, and it was not easy to locate any particular piece of information. Fortunately the tribunal ruled that all of the relevant information was in the reports and the engineer was not penalised because of its poor presentation. However, the case highlights the need for engineers to give thought to the manner in which soils data is given to tenderers, and to make sure it is presented in a systematic and coherent manner.

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Top: Ngauranga Gorge Interchange, Wellington

Retaining wall supporting the Southbound motorway lanes.

Above left: State Highway 8 reconstruction, Cromwell

A 123m long x up to 9m high retaining wall (741m²) erected in just 15 working days.

Left: Clyde Dam Abutment

A 100m long x 15m high retaining wall to provide a toe buttress platform.

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THAMES MUNICIPALITY RIVER INTAKE STATION

GEOPHYSICAL SURVEY OF RIVER BED AND SUBSOIL CONDITIONS

by A.H. Nelson, Geotechnical Section
Murray-North Partners Limited
and P.J. Dolden, Subsea Surveys Ltd

The municipal water supply to the Thames urban area is partially served by an intake located on the north bank of the Kauaeranga river. Heavy floods during 1985 led to severe erosion on both banks and left the intake structure and pumphouse in a precarious position and subject to silting. Remedial work proposals included river entrainment measures designed to stabilize the banks and restore the river to its original alignment. A combination of anchored sheetpile walls and rock filled wire baskets was considered the most cost effective and environmentally sensitive solution.

The Kauaeranga river has its source in the Coromandel State Forest Park and drains a catchment area of approximately 13,000ha (Tait, 1978). It is a normally placid river but swiftly and frequently becomes a raging torrent during periods of heavy rainfall on the western flank of the Coromandel Range. Boulders up to 1.5m in diameter together with heavy sand and silt loads have been deposited in the lower reaches during these flash floods. The intake structure is located on alluvial sands and silts, while directly opposite the intake andesite bedrock outcrops on the southern bank of the river. Rapids both upstream and downstream of the intake are formed of rounded andesite boulders deposited under previous flood conditions. The bed itself is considered to be formed largely of alluvial silts and fine gravels, with the possibility of large boulders and buried tree trunks.

Remedial work proposals called for 10m long anchored sheetpile sections driven to a minimum depth of 4m below the present bed level on both banks of the river. These were to be further protected against scour by a system of rock filled wire baskets and mattresses. Concern as to the practicality of installing sheetpiling to the required depths in the light of the size and possible distribution of the hard rock boulders and the depth of the underlying bedrock led to a proposal to undertake a geotechnical investigation of the river bottom. Standard methods of site investigation such as borehole drilling or penetrometer probing were ruled out due to the high cost of site mobilization and the poor coverage afforded by these methods. Following discussions with Subsea Surveys Limited a geophysical survey using seismic refraction techniques was commissioned and undertaken as follows:-

A sub-bottom profiling system consisting of an O.R.E. Transducer Array coupled to a Hydroproducts Model 410 Transceiver and an E.P.C. Graphic Recorder was used. Power for the system was supplied by a portable generator. All equipment was mounted on or in two 3.6m aluminium dinghys lashed together and powered by a single outboard motor. The transducer array was just submerged from a gantry mounted between the two aluminium dinghys.

A Raytheon Echo Sounder was used concurrently mounted next to the transducer array. This produced records of the water depth and these were annotated with "fix" marks simultaneously with the profiling system. Positioning during the survey was determined by a surveyor on the bank using observed angles to the boats and distances from known reference points. The results obtained were carefully scrutinised and a series of interpretive drawings constructed. The record quality in shallow water (less than 3 metres) was not good due to noise present in the top two metres of water. The noise was caused by the boats

propeller, hull friction and the transducer array which caused a shallow column of water to resonate with it's pulses.

The records consist of strip charts on which are burnt traces corresponding to returns of the signal transmitted. The first return is usually the riverbed and may be followed by others originating beneath the riverbed. The sound signal is reflected back by changes in the medium in which the sound is travelling. Hence the interface between two differing materials causes the signal to be reflected back. These interfaces have often been shown to coincide with the interfaces between sedimentary deposits or the differences on either side of a fault plane. With skilled interpretation this system can be used to establish the configuration of the stratigraphy of sedimentary deposits.

The depth of penetration achieved is a function of the materials encountered. The records themselves do not give an indication of the properties and/or nature of the materials. As with all geophysical methods correlation with other site investigation data is required. This was achieved by physical inspection of the bedrock, hand auger holes into the alluvium deposits and inspection of the records made during construction of the pumphouse and intake structure.

The records were examined and the reflections taken to have originated from the underlying "basement" rock identified. A cross-section for each of the lines run was constructed and the depths reduced in terms of site datum. From these sections it was possible to identify the elevations below site datum. These positions were translated on to a plan of the river and joined to like elevations creating a contoured plan of the "basement" rock. (Figure 1).

The records were further examined for the occurrence of boulders and/or other phenomena in the sediments overlying the "basement" rock. The near surface records were confused and difficult to interpret but no evidence of obstructions was identifiable above the "basement" rock. Hence it was concluded that the river bed consists largely of silts and sands overlying bedrock and that no major obstructions are likely to be encountered during driving of the sheetpiles. The investigation thus provided adequate coverage of the entire site for the cost of the equivalent of approximately three boreholes.

A word of caution:- This system can be affected by a large number of variables and no guarantee of achieving useful results can be given prior to completion of the survey.

References:

Tait, M.B. 1978. Hauraki Catchment Board and Regional Water Board, Waihou Valley Scheme. Kauaeranga River Flood Control Scheme, Hydraulic Design Standards. Unpublished report, Te Aroha, June 1978.

Permission from the Thames Coromandel District Council to publish this article is gratefully acknowledged.

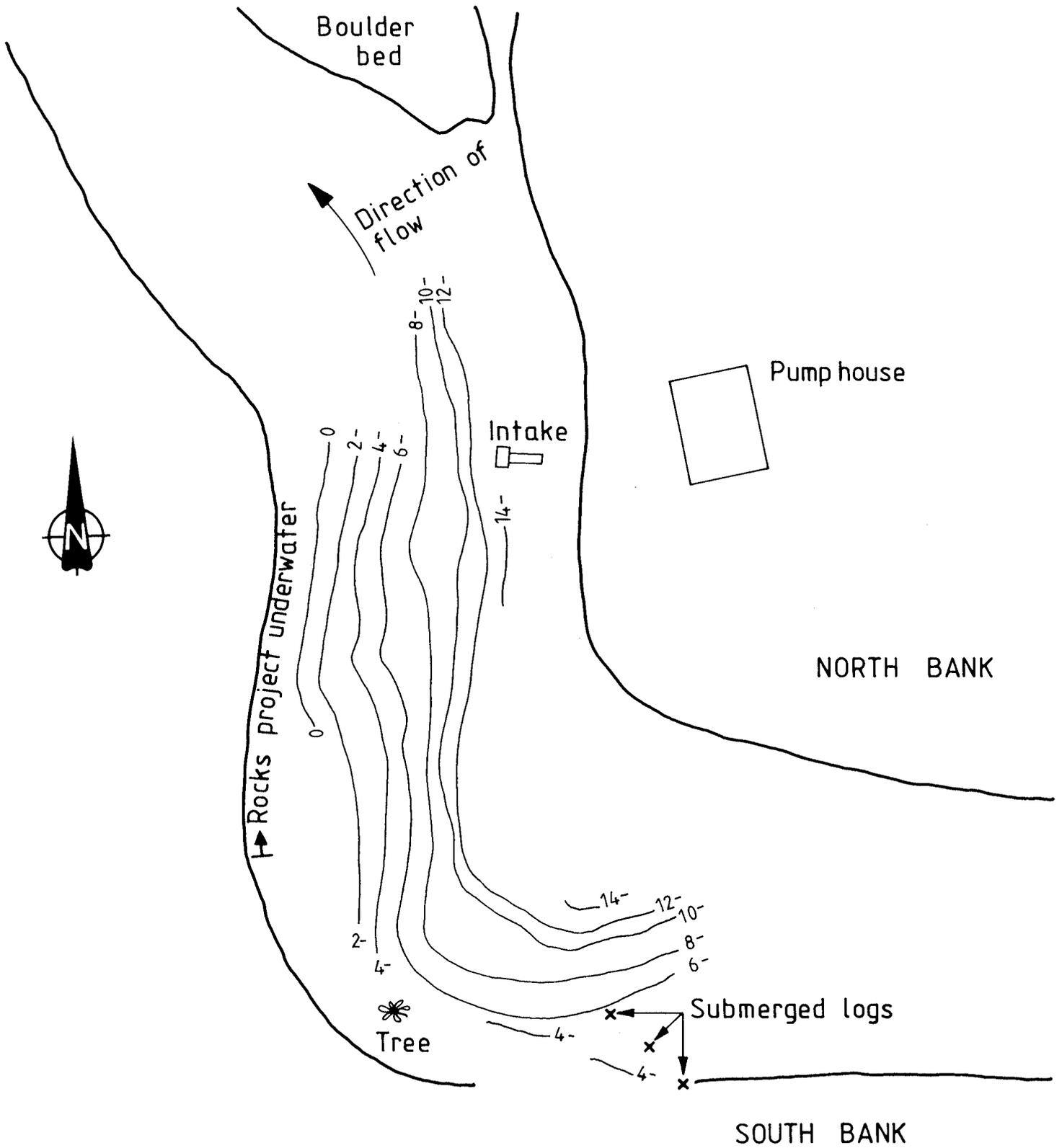


FIGURE 1

"Basement" rock contours below site datum
(2m contour intervals)

ONE APPROACH TO RISK IDENTIFICATION OF
INSTABILITY IN GREYWACKE CUTSLOPES OF THE WELLINGTON AREA

by A.G. Mahoney - Brickell, Moss and Partners.

In line with part of the intention expressed by the Presidents Task Committee on Professional Practice and Risk - Geotec. No. 29 1984, I also believe it is important that clients are made aware of risk involved particularly in cut slope design and construction. The problem I have found in endeavouring to advise clients of potential risk involved in respect to cut slopes, is the lack of some practical numerical scale by which various slope angles can be compared. Generally, subjective terms such as extreme, high, moderate or low risk are very useful and sufficient, however, in some cases, where space is short and land values are very high, i.e. commercial properties/development, a more definitive scale of comparison is often advantageous.

Even with the aid of borings and a reasonably good idea of the general rock conditions likely to be encountered in a cut within Wellington greywackes, the variables within the normally complex defect patterns largely determine its behaviour. I have never, personally, been sold on a typical soils analysis type approach in this material. Additionally, I am also all too aware of the difficulty of inferring internal structure from outcrops, borings etc., with any great degree of reliability prior to cutting, as is typically done in a rock mechanics type of approach.

My experience in greywackes is that 'the proof of the pudding is in the eating', i.e. you do not really know what you have until you cut! Because of the recognised severe limitations attached to practically assessing the inherent risk of instability, I have endeavoured to develop a graphical presentation to risk definition.

Based purely upon my own general experience of observing numerous cut faces in the district with their many vagaries and surprises, a plot of likely failure risk against slope angle for cuts made into the highly weathered to moderately weathered greywacke for vertical heights of up to 15 metres, has been compiled. This plot is based upon the material type assumption that there is no specific reason to suspect the cut material is likely to be other than typical of the 'normal' or average conditions. In addition stormwater from above is unlikely to be a major problem.

The order of risk is plotted on a semi log graph with the risk plotted on the vertical axis and the batter slope on the horizontal. A simple straight line plot of inferred risk for a given slope angle can be manufactured by fixing two points.

At a higher risk end of the scale one could reasonably infer with virtual certainty of incurring significant failure (i.e. a failure greater than many 10's cubic metres) even for the most optimistic case, on a vertical cut face. A more pessimistic case could be nearer 1/3 horizontal to 1 vertical. This range of sure failure, i.e. 1 in 1 chance of failure, is in my opinion entirely reasonable.

Establishing a lower limit risk is more difficult. Taking a 1 on 1 (45 deg) slope as being a currently accepted 'safe' slope for normal design purposes in the area, I have arbitrarily assigned a 1 chance in 100 risk. Again, based purely upon subjective observations of slopes in the area and being aware of occasional very rare failures on 45 deg slopes, this implied order of 1:100 risk is considered reasonable. The 1:100 risk for a 45 degree slope could be considered as a reasonably pessimistic case.

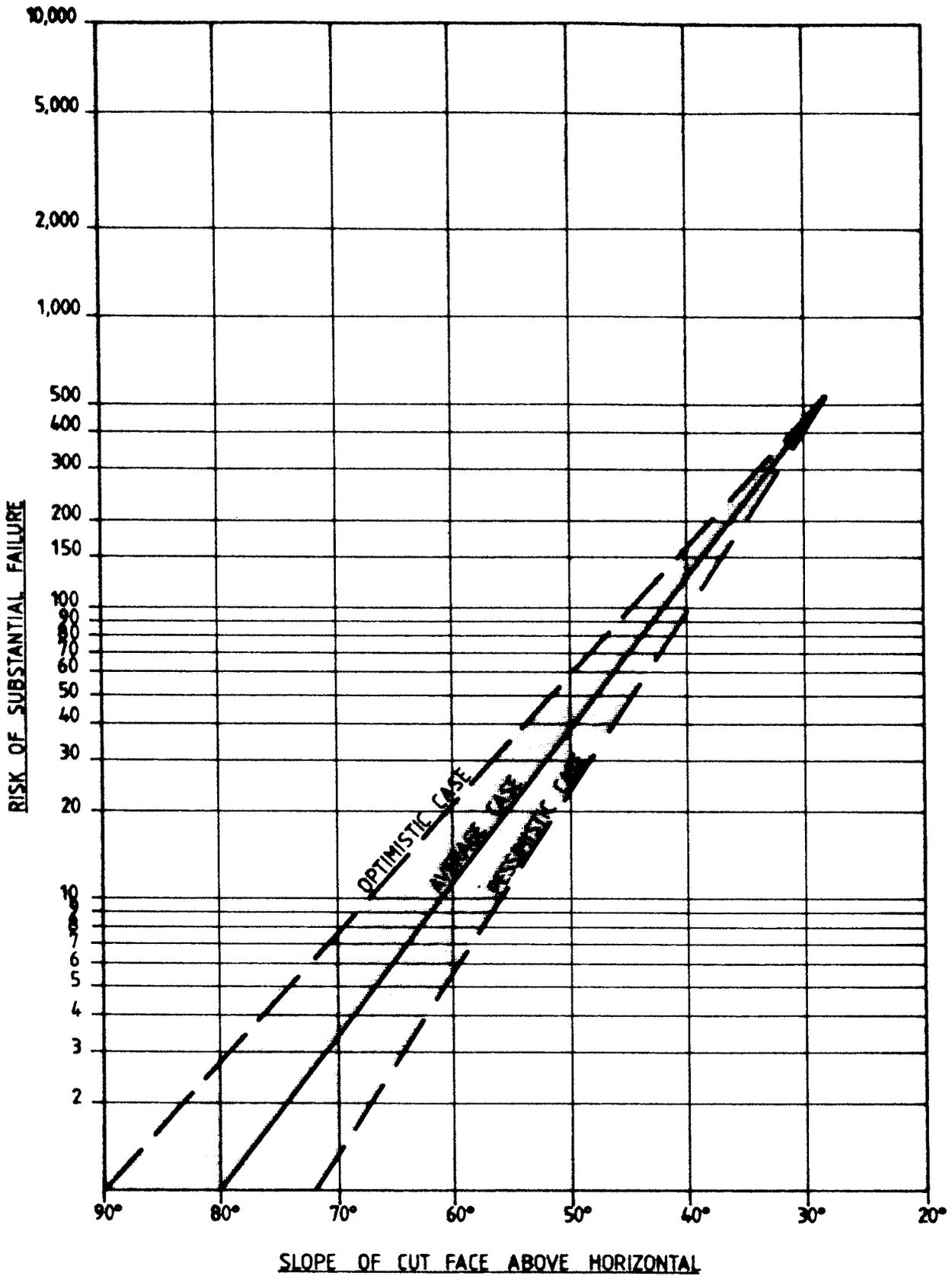
The straight line plot thereby resulting from establishing such order of risk for the 1 in 1 chance of failure to the 1:100 chance cases, provides a basis for comparative risk of substantial failure.

"Fine", you may say! - but as it is all based upon completely unsubstantiated (i.e. not scientifically deduced) assumptions, what real use is it?

Answer:- Probably no more or less use than most soils/rock stability analysis which are generally based upon a few discrete measured geometric and geotechnical properties. These properties are then by necessity inferred to be representative of the whole mass! Using the above suggested graphical approach certainly is limited in its usefulness but then so really are most systems.

One principal use of such a simplified approach as outlined above, is to provide a number which is the only language understood by many clients such as Structural Engineers, Accountants, Developers, Tax Collectors, Share Brokers, Town Planners and others of similar ilk.

The real value I see in using such an approach, is being able to communicate in a simplified, quantifiable manner the reality of risk and the degree of risk relative to the various options being considered. While we are all aware that there is no absolutes in the stability game, it is essential that the geotechnical engineer gets this clearly across to the client. The client then is in a better position to assess the level of risk he/she is prepared to accept and is able to compare the cost effectiveness of other options which will probably be available to improve the stability of the cut slope (determined by the space available), if the risk is considered unacceptable.



RELATIVE RISK OF SUBSTANTIAL FAILURE OF SLOPE CUT
IN H.W. GRADING M.W. INSITU GREYWACKE

THE USEFULNESS OF GEOPHYSICS IN SITE INVESTIGATIONS

BY
CHRIS BROMLEY and CHRIS PEARSON
KRTA LIMITED

1. INTRODUCTION

Geotechnical engineers are frequently called upon to make an assessment of subsurface ground conditions when investigating a site for future development, or when designing foundations. Geophysics can greatly enhance the quality of this assessment by providing in-situ measurements of various subsurface properties, without disturbing the site. Examples of geophysical methods frequently employed include : seismic refraction profiling, resistivity soundings and ground-probing radar. Other methods, employed less frequently, include gravity and magnetic field surveys. All these methods use surface measurements which are then interpreted using subsurface models of lateral or vertical changes in soil and rock properties. For seismic surveys this property is acoustic velocity; for resistivity, it is electrical conductivity; for gravity it is density; and for magnetic field surveys, the property is usually magnetic susceptibility.

Experience has shown that many soil and rock properties are inter-related. changes in the geophysically measured properties, such as velocity, density, and resistivity, can often be related to changes in porosity, water saturation, clay content, and clay type, as well as to more fundamental differences in lithology. These changes, in turn, can be related to changes in the engineering properties of the subsurface material, such as its shear strength and stability.

This paper discusses the advantages, limitations and some basic principles behind various geophysical surveys used in site investigations, and then illustrates them by means of a few case histories. It is hoped that this approach will assist the geotechnical engineer, or the engineering geologist, to make informed and practical decisions, in specific cases, on suitable geophysical techniques to employ.

2. SEISMIC REFRACTION SURVEYS

Seismic refraction profiling is a rapid and inexpensive means of obtaining information on the depth to a competent basement, which is of importance in the design of foundations for civil engineering structures and buildings. Measurement of seismic velocities is also used to help estimate subsurface rippability for road cuts or excavations, and to facilitate calculations of deformation in the abutments of dams (Atkins, 1980). There are numerous other applications of the method, including the location of buried faults, as revealed by vertical displacements of a high velocity basement, and calculation of the thickness of unconsolidated fill, or the depth of weathering, for assessing potential excavations or for evaluating slope stability.

The seismic refraction equipment consists of : a seismograph, comprising a high speed multichannel photographic or digital recorder, a geophone for each channel of the recorder (usually 12 or 24), a multicore cable to connect the geophones to the recorder, and an electronic blaster for use with explosives, or alternatively a hammer and plate, with a signal stacking device, for use in sensitive built-up areas. The geophones are placed along a selected profile line, usually at spacings of 5 to 12 metres for shallow engineering seismic refraction studies. The optimum geophone spacing and length of profile line can be calculated using preliminary estimates of the layer velocities and the expected depth to a refracting (high velocity) basement. In a typical case, with a high velocity refractor at about 10m depth, one would probably use about 120m of profile line to achieve over-lapping coverage, and therefore accurate depth calculations, for the central part of the profile. Ideally, explosive charges are detonated at the end-points of the line, at several intermediate points, and at 'offset' locations, several tens of metres behind each end. If a hammer-and-plate source is used, the signals from a large number of hammer blows at each shot point are combined (or "stacked") to improve the signal-to-noise ratio.

"Picking" of the seismograph records involves measurement of the time difference between shot instant and the arrival of the first wave at each geophone. These are then plotted on a time-distance graph. Velocities and layer depths for two or three cases can be calculated using the "plus-minus" technique (see Cummings, 1979). The plus time is defined as the sum of two arrival times at a geophone, using shots fired from opposite directions, minus the total travel time between shot points, and is used to calculate the depth to the refractor. The minus time is just the difference between these arrival times and is used to calculate the true velocity of this refractor.

Limitations on the accuracy of interpreted velocity sections using the refraction technique are caused by the following:

1. steep boundaries,
2. small velocity contrasts,
3. high velocity layers overlying low velocity layers,
4. surface topography,
5. background vibration noise.

Some of these limitations can be reduced or removed by careful selection of the profile line locations, the shot timing, and the weight of explosives. To some extent, the limitation of steep boundaries is caused by a small angle assumption in the algorithm used by the plus-minus method. In severe cases, it is possible to alleviate the problem by using a different iterating algorithm that does not rely on this small angle assumption.

Because changes in acoustic velocity can be caused by a variety of physical property changes, it is helpful to pass at least one profile line across a point where the lithology is well known, such as a borehole if one exists, or a surface outcrop of the high velocity basement.

Correlations between the observed lithologies and the interpreted velocity section can then be made, and predictions of the lateral extent and thickness of the overburden can be determined with greater reliability. If the seismic refraction survey is conducted at an early stage of a site investigation, then subsequent borehole drilling or augering can be programmed more effectively and economically using the velocity sections as a guide.

An example of the successful application of the seismic refraction method to site investigations is a survey conducted recently on a site in South Auckland. This site includes a volcanic crater, approximately 100m in diameter, which is filled in with organic silts and peat. The developer wanted an accurate and detailed profile of the thickness of the peat/organic silt in order to plan his development strategy for the site. Several boreholes in the crater revealed a maximum thickness near the centre of about 20m. Three profiles across the crater were shot, with geophone spacings of 5 or 10m, and array lengths of about 150m. The results revealed a strong velocity contrast between the peat/organic silt (0.3 km/sec), and the underlying volcanic colluvium or bedrock (1.0 to 3.0 km/sec). The depths to this interface were calculated using the plus-minus method, and plotted on velocity profiles. (See Figure 1). At the intersection of these profiles with boreholes, the calculated depths accurately matched the observed depths of the high strength basement, determined from undrained shear strength measurements, and lithologic identification of cores. In between the boreholes, the velocity profiles revealed an interface that was significantly different from that assumed by just linear extrapolation of borehole information. Therefore, a much more accurate estimate could be made of the total volume of peat and organic silt in-filling the crater.

3. RESISTIVITY SURVEYS

The resistivity method is most commonly employed in the search for groundwater aquifers, geothermal exploration, and mineral prospecting. The method detects lateral or vertical variations in the resistivity (or conversely the conductivity) of the subsurface formations and their saturating fluids. Resistivity has been shown to depend principally on porosity, fluid saturation and salinity, clay content and type, and temperature. In relatively shallow engineering investigations, all these properties, with the exception of temperature, are likely to vary significantly.

In the absence of conductive clays, the resistivity of a saturated formation is dominated by its porosity and the resistivity of the saturating fluid, which is simply related to its salinity. This relationship is known as Archie's Law and is frequently presented in the form : $R = A R_w \phi^{-n}$ where " R_w " is water resistivity " ϕ " is porosity, " A " is a constant, usually varying between 0.6 and 1.6, and " n " is also a constant, often equal to 2. The water resistivity is temperature dependent, but at ambient temperatures it is approximately $0.1/C$ where " C " is the ionic molar concentration of the water. In practice, the effective water resistivity reaches a maximum of about 10 ohm-m for fresh water because of the contribution of free ions loosely attached to the walls of joints and pores. The presence of conductive

clay minerals also adds to the total conductance of a formation (i.e. lowers the resistivity), because of the contribution of sorbed ions through ion exchange reactions on the clay surfaces.

The standard procedure for conducting a resistivity sounding, using the Schlumberger array, involves passing high voltage direct current through a pair of electrodes that are spaced at increasing intervals in order to get information from progressively deeper levels. The ground potential voltage is measured at the centre of the array, and the resistivity calculated from the ratio of the potential voltage to the current. This resistivity is plotted against half the current electrode spacing to produce the sounding curve, which is interpreted in terms of layers. It should be noted that a maximum electrode spacing of at least four times the depth to an interface is required for adequate detection, and there should not be significant lateral contrasts in resistivity between the electrodes for a layered interpretation to be valid.

Examples of the application of the resistivity method to specific engineering problems or site investigations include the following:

- (a) Ground water flow direction and velocity have been measured in shallow gravels on the Canterbury plains, by pumping a slug of salt water into a well and monitoring resistivity as a function of time (White, 1986). This has assisted calculation of the recharge by rivers to the ground water system that is tapped for horticultural purposes.

Identification and mapping of significant ground water aquifers is also an important aspect of dewatering studies for deep excavations. The resistivity method provides a useful and inexpensive means of locating aquifers, particularly those within bedded gravel and sands. (KRTA Ltd., 1984).

An environmental study of the extent of shallow ground water contamination by hazardous fluids (from dump sites, for example), can benefit from a resistivity survey, if the polluting fluids contain sufficient ions to create a strong resistivity contrast with the surrounding fresh water.

- (b) In the Auckland region, identification of variations in clay content and composition, within the weathering profile of the Waitemata Group sediments, can be very important when considering the probability of slope failure at a particular site. Building foundation failures caused by shrinkage or swelling of particular clays, are quite common. (Harvey et al, 1982). In principle, the various layers within a weathering profile, as distinguished by the relative abundance of clay minerals such as kaolinite, illite and montmorillinite, can be successfully modelled using resistivity methods, because these clays have variable ion exchange capacities. Montmorillinites are highly conductive, illite has intermediate conductivity, and kaolinite is a relatively poor conductor, by virtue of its low ion exchange capacity. In practice, interpretation of resistivity data over clays also needs to take into account variations in porosity and water saturation because of the important effect of the pore fluid, particularly if it contains ions.

The contact between soft, saturated, porous clays, rich in montmorillinite, and an underlying basement of hard siltstone or sandstone, is frequently responsible for some of the more spectacular slope failures. Such a discontinuity will probably have a very large resistivity contrast which can readily be mapped using resistivity soundings.

- (c) The resistivity method also has an application in the evaluation of potential quarry sites because it can model the depth of weathering overlying fresh competent rock. In cases where this interface is gradational, interpretation control may be needed using a surface rock exposure or a nearby borehole.
- (d) A recent example of the application of the resistivity method to geotechnical investigations of foundation conditions involved the assessment of possible geothermal power plant sites. As described by Layugan (1983), a resistivity survey contributed to the feasibility study of the Bacon Manito Geothermal Project in the Philippines (KRTA, 1983). A vertical electrical sounding located over one potential power station site, (Palayang Bayan Crater) revealed a layered geometry comprising : 1 metre of dry soil, 7.8 metres of saturated soil, 30 metres of a soft grey organic clay (deposited in a lake environment), followed by a resistive fresh volcanic horizon (see Figure 2). The organic clay was sampled and shown to be highly compressible and unsuitable for building foundations.

4. GROUND RADAR

Subsurface interface radar (SIR) uses the reflections of electro-magnetic wave pulses (100-1000 MHz) to map subsurface interfaces in the earth. The pulses are reflected at discontinuities of electro-magnetic wave velocity in the earth, and recorded using detectors on the surface in a manner similar to seismic waves. Because of the much higher frequencies, SIR is ideally suited to detailed investigation of shallow interfaces.

There are two problems in interpreting SIR data. The first is determining the appropriate velocity to be used in depth calculations. This can be accomplished by calibration against known depths from borehole data (in which case the mean velocity is just twice the known depth divided by the travel time), or by the wide angle sounding mode, where the velocity is calculated from the "moveout" as the source detector distance is increased.

The second problem with SIR surveys is the limited depth of penetration. Because SIR pulses use electro-magnetic radiation, their amplitudes decay exponentially as the pulse travels into the earth. The rate of decay can be calculated from the skin depth equation. Attenuation of electro-magnetic waves is caused by conductivity in the substratum:

$$E(z) / E(o) = \exp (-2 \times 10^3 Z/\text{SQR} [F/R])$$

where F is the frequency, R is the resistivity, and Z is the depth. The "skin depth" is the depth at which the signal is reduced by a factor of $1/e$ or 37%. this is given by:

$$Z = 500 * \text{SQR} (R/f) \text{ metres.}$$

Note that two-way passage to the skin depth will reduce the signal by a factor of $1/e^2$, or to 14% of its original value. Because the instrument is capable of recording very small signals, this equation should not be used to estimate the maximum depth penetration of SIR systems, but it does illustrate the fundamental problem. In practice, the depth of penetration ranges from one kilometre or more in glacial ice, to a few centimetres in gravels saturated with seawater. Typical depths of penetration in peat (resistivity 100 ohm.m) are about 10m, and in sandy clay (5-20 ohm-m), 3 to 5m.

4. GRAVITY & MAGNETICS

Both of these techniques are a relatively inexpensive and rapid means of obtaining information on changes in subsurface density and magnetic properties. They have frequently been used in detailed archeological investigations and searches for buried objects. The gravity method is also used to map basement depth, when the overlying formations have a different density. This can be particularly useful in situations where the seismic refraction methods fails because of excessive depths or because of a surface layer of higher velocity.

Interpretation is hampered in both methods by the fundamental problem that all model solutions are non-unique, without some additional constraints such as borehole information, because the anomalous response at the surface depends on the size of the buried body, its magnetic or density contrast, and its depth of burial. In the case of gravity surveys, accurate elevation control is also necessary, because an error in station elevation of 10m can result in an apparent gravity anomaly of about 2 mgals, which is often comparable to the anomaly of interest.

Ground magnetic surveys are often used to locate buried contacts between volcanic formations such as lava flows (with high magnetic susceptibility) and sedimentary sequences (with typically weak magnetisation). This could prove useful in a building site investigation over buried lava flow contacts, especially if the flow is considered suitable for bearing foundation loads.

SUMMARY

Geophysics can often assist the geotechnical engineer and the engineering geologist with assessments of subsurface conditions when investigating a potential development site. Knowledge of the methods available, their limitations and advantages, and their cost effectiveness relative to borehole drilling and coring, can prove highly advantageous in today's competitive world of engineering consultancy.

This paper has attempted to briefly describe the main surface geophysical methods that are employed in site investigations, while placing an emphasis on practical applications and case studies, in order to provide a background for informed decisions on the most suitable method to use. The methods described have included seismic refraction, resistivity, ground radar, gravity and magnetics.

In some situations a combination of techniques may be advantageous. Their usefulness is based on the observed inter-relationships between soil and rock properties such as velocity, density, resistivity, magnetisation, porosity, clay content and shear strength.

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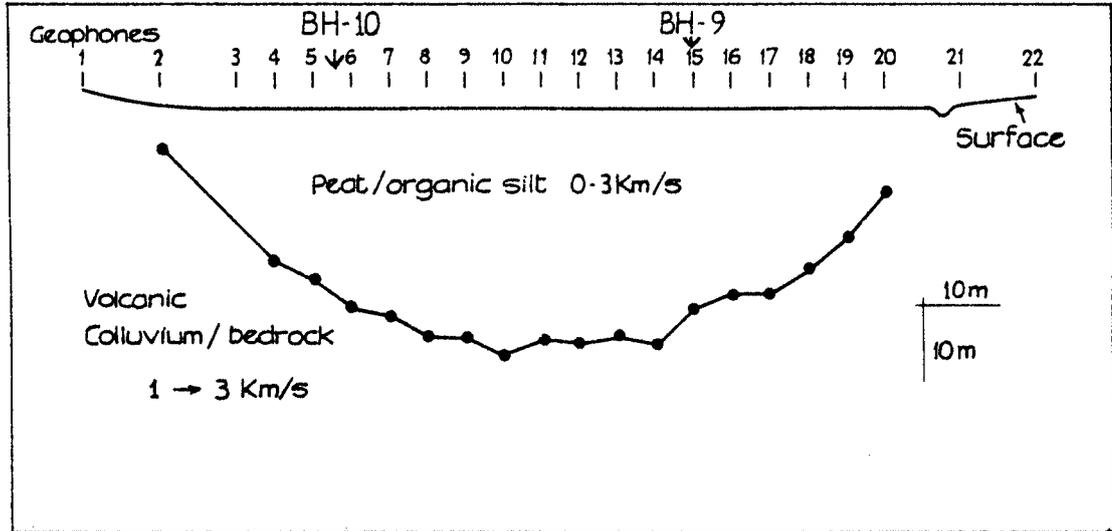


Fig 1 INTERPRETED VELOCITY CROSS-SECTION

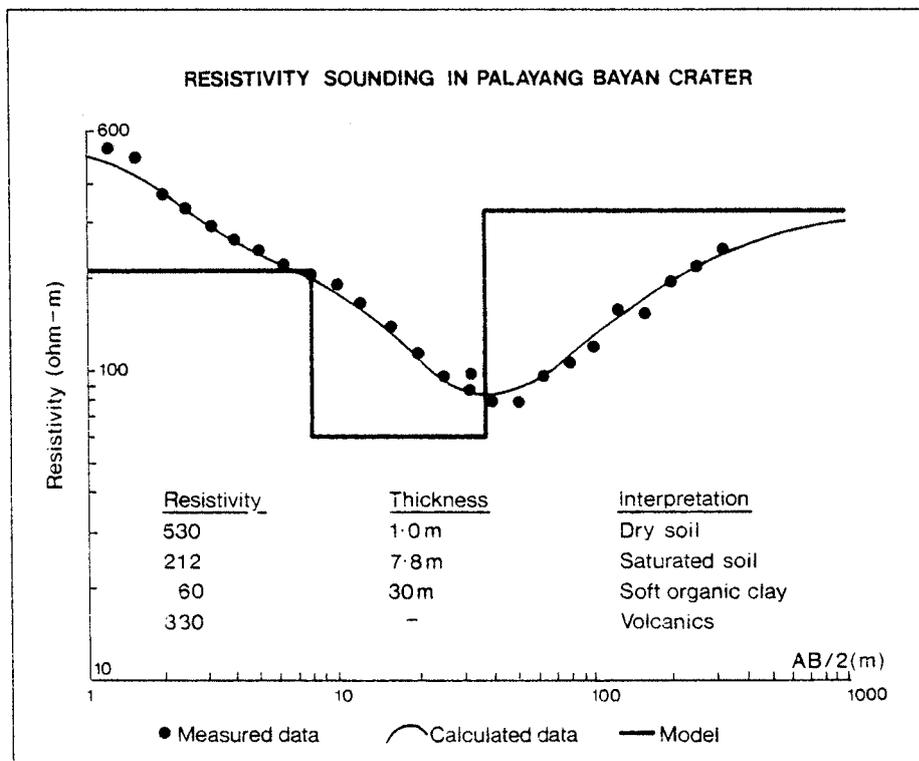


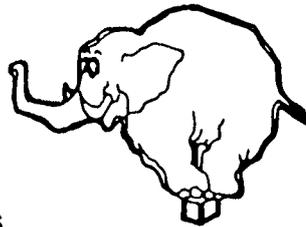
Fig 2

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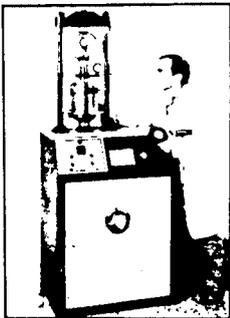
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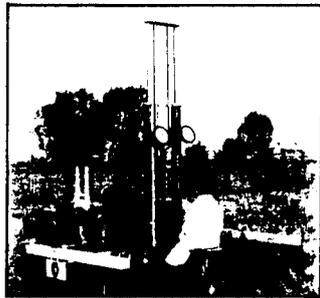
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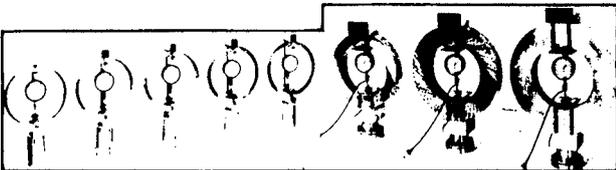
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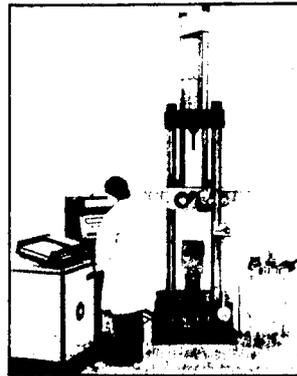
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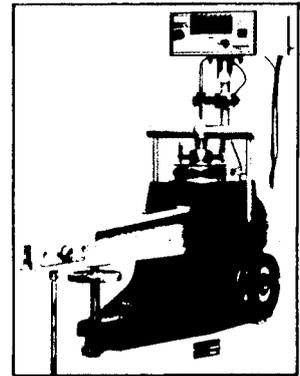
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LONG TERM MONITORING OF GROUND ANCHORS
USED IN RETAINING WALL CONSTRUCTION
BY G. C. ARCHER

1.0 INTRODUCTION

It is notable that for the first time in Wellington, a permanent multi anchored retaining wall constructed for a private commercial development, is to be performance monitored during construction and throughout its expected functional life. Various forms of monitoring have been implemented during the construction phase of anchored retaining walls within the past 10 years and this monitoring has been mainly face position assessments. The relevance and accuracy of some of these monitoring programmes which have been implemented in the past are somewhat questionable and analysis of the face deformation data was in most cases generally not conclusive.

There has been a reluctance by principal advisers to specify performance monitoring of the ground anchored system after construction, and this is mainly due to the complexity of responsibility and payment for this work. Similarly, there is a tendency to be somewhat complacent about the long term integrity and functional capacity of permanent ground anchored retaining walls.

This situation has arisen in part from a general absence of technical evaluation on failed ground anchors and systems despite failures having occurred both locally and internationally. There has been a level of confidence on the part of principal designers that permanently ground anchored retaining walls are no more vulnerable or different from other forms of foundation and retaining wall construction.

It is becoming more generally accepted that permanently ground anchored structures are more vulnerable to decaying long term performance than foundations, cantilevered retaining walls, etc. It is now recognised that, because permanent ground anchored retaining walls associated with inner city developments are usually integral with the structure, the consequence of partial or total collapse of the anchored system on the economic returns of the development are in most cases significant.

2.0 EXISTING CODES OF PRACTICE

There are a number of international codes of practice for the design, construction and monitoring of ground anchors. While design and construction aspects of ground anchors are generally treated in some detail, post construction monitoring is discussed in very general terms. As a result the author became involved in assisting with the preparation of a procedure for the long term monitoring of existing permanent ground anchored slopes for a development in Hong Kong. The basic procedures arising out of that study together with specific suggestions for the Wellington conditions are given below.

3.0 GENERAL PROCEDURES FOR LONG TERM MONITORING

The extent and type of long term monitoring of permanently ground anchored retaining walls should take into account the following:-

- (a) the dependence of the retaining wall or structure on the anchor support;
- (b) the risk of failure of either partial or total collapse and the consequential danger to property and the general public;
- (c) the usage of the surrounding land and in particular the intensity of development adjacent to the ground anchored retaining wall;

The highest standards of long term monitoring should be for permanent ground anchored structures where the failure of only a few strategic ground anchors could lead to partial or total failure of the overall ground anchored system.

3.1 Importance of the Ground Anchored Retaining Walls

The extent and type of long term monitoring of permanent ground anchored retaining walls are dependent on the importance of the overall retaining wall.

For the purpose of determining the extent of long term monitoring, each ground anchored retaining wall should be classified into one of two important categories as follows:-

(i) Importance Category A - Highest risk applies where:-

- . overall stability of the retaining wall is only achieved with permanently tensioned ground anchors;
- . failure of only a few ground anchors significantly reduces the margin of safety and increase the overall risk of general failure of the retaining wall system;
- . failure of part or all of the ground anchored retention system imposes high risk to the general public safety, property and utilities.

(ii) Importance Category B - Moderate risk applies where:-

- . ground anchors supplement and increases the overall stability of the slope and or retaining wall;
- . failure of only a few anchors would not lead to a significant risk of overall failure of the retaining wall or slope;
- . there is only minimal risk to public safety and damage to property if several anchors fail.

Most multi anchored retaining walls for inner city commercial developments are classified as Importance Category A.

4.0 METHODS OF MONITORING

The monitoring of a ground anchored retaining wall should include all of the following:-

- (a) A visual inspection of all anchor heads for signs of corrosion, faults, distress, etc.
- (b) Checking the residual loads in a selection of ground anchors by lift off loading technique or the permanent placement of calibrated load cells behind the anchor heads.
- (c) Checking the face deformation and position by normal optical or laser survey techniques preferably measuring direct offsets from various points of position stability.

In some cases where ground anchor retaining walls are constructed in areas containing fluctuating levels of ground water and/or aggressive ground water both the level and acidity should be checked as part of the overall monitoring programme.

The requirements of (a), (b) and (c) above, are the minimum requirements for all monitoring. The design engineer should assess the need for ground water monitoring for the specific project being considered and include this form of monitoring as required.

5.0 FREQUENCY OF MONITORING

The suggested frequency of monitoring of ground anchored retaining walls may be separated into two stages, these being -

1. During Construction - (Construction and Maintenance periods):
Following the suitability and routine testing of ground anchors at the time of construction, a selection of ground anchors (generally those that are to be load monitored), should be visually inspected and have their residual loads checked. This, together with retaining wall deformation surveys should be carried out at one week, one month and then at three monthly intervals after acceptance loading and lock-off. This construction monitoring should apply for all ground anchored retaining walls despite which importance category is appropriate.

2. Post Construction - (Post completion of the Maintenance period):

TABLE 1A : FREQUENCY OF LONG-TERM MONITORING - IMPORTANCE CATEGORY A

Years following Maintenance	Visual Inspection	Residual Load in Ground Anchors	Face Deformation Survey
0 to 2 years	6 monthly	6 monthly	6 monthly
2 to 5 years	6 monthly	6 monthly	Annually
5 to 10 years	Annually	Annually	Biannually
10 to 20 years	Annually	Biannually	Biannually
Thereafter	As required	As required	As required

TABLE 1B : FREQUENCY OF LONG-TERM MONITORING - IMPORTANCE CATEGORY B

Years following Maintenance	Visual Inspection	Residual Load in Ground Anchors	Face Deformation Survey
0 to 2 years	6 monthly	6 monthly	6 monthly
2 to 5 years	Annually	Annually	Annually
5 to 10 years	Annually	Annually	Annually
10 to 20 years	Annually	Biannually	Biannually
Thereafter	As required	As required	As required

6.0 RESIDUAL LOAD MONITORING

The residual load monitoring of selected permanent ground anchors may be carried out by re-establishing a jacking system over each anchor or by fitting load cells behind the bearing plate for each anchor to be monitored. Whilst the latter would be initially expensive to establish it does, in the opinion of the author, provide the best long term form of load monitoring particularly where buildings and developments tend to change ownership throughout their economic lives. A load cell system provides for monitoring of residual loads by the owner's maintenance personnel without involving the original designers, developers etc., in the routine work. Irrespective of which system is used, the number of ground anchors to be monitored should be as follows:-

TABLE 2 : NUMBER OF GROUND ANCHORS FOR LONG-TERM RESIDUAL LOAD MONITORING

Importance Category	Percentage of Ground Anchors for Residual Load Monitoring		
	Number of Ground Anchors in Retaining Wall		
	1 to 50	51 to 100	greater than 100
A	20%	15%	12%
B	15%	10%	7%

7.0 FACE DEFORMATION MONITORING

It is important that the principal designer think out the method of measuring face deformation. It has been common practice in the past to calculate deformation by normal survey techniques using bearings and distances. This practice has been shown to be quite inaccurate and can lead to large errors of inaccuracy. The preferred form of monitoring of face deformation should comprise one of the following:-

either

- (a) offset measurements either from a line of sight or to known fixed and stable points. Survey equipment which is readily available and having a high level of measured accuracy should be used; or
- (b) Extensometers installed into the ground anchored retaining wall at the time of its construction.

The relatively high component cost and expertise required to measure extensometers generally precludes their general use for normal commercial development projects. The use of survey techniques as described in (a) above, generally provides an acceptable accuracy in assessing trends rather than absolute measurements.

8.0 INTERPRETATION OF MONITORING

Equally as important as carrying out the monitoring processes the principal designer should establish the basis for interpretation of monitoring such that there can be no misinterpretation in the years after construction. The effort and cost of putting in place monitoring systems are completely wasted where insufficient clear detail for interpretation of monitoring is not provided. The principal designer should establish the interpretation criteria specific to the project being considered taking account of the levels of risk attached to failure of the system. A suggested generalised interpretation of the monitoring results is given below.

8.1 Residual Load in Ground Anchors

Permanent ground anchors will seldom retain their original lock-off loads with time. It has been the author's experience that ground anchors tend to de-stress with time and this can be caused due to a number of isolated and complimentary factors. In order to permit anchor de-stressing within normal and acceptable limits it is now practice to lock-off ground anchors at 110 percent of their required permanent tension forces. The monitoring of ground anchors on a number of projects over a period of time has shown that where there are no other contributing factors such as face deformations, anchor failures, etc., ground anchors tend to de-stress generally in accordance with the relationship as follows:-

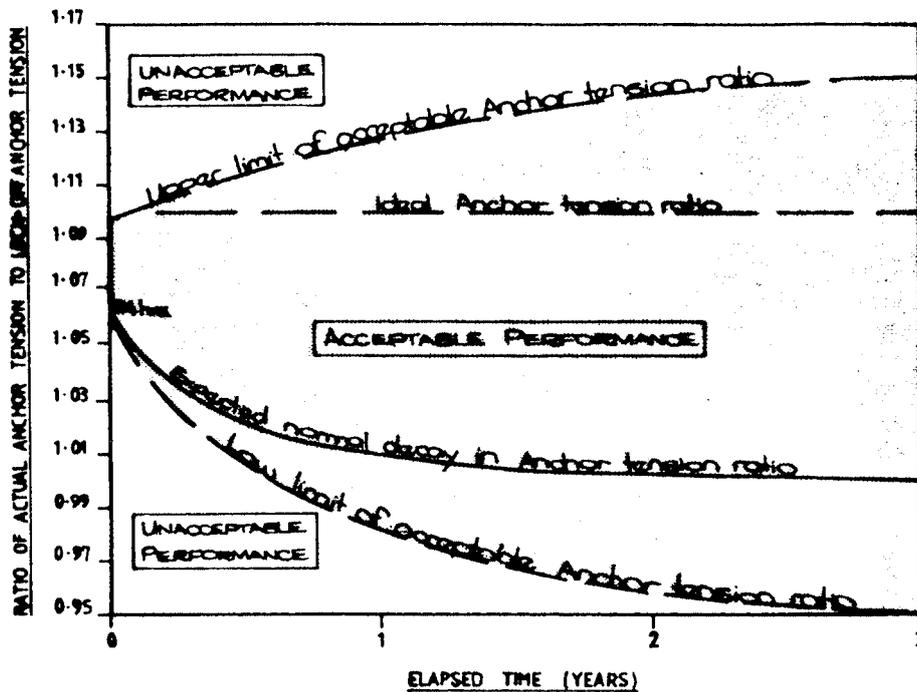


FIG. 1 ACCEPTABLE LIMITS OF ANCHOR RESIDUAL TENSION RATIO

Interpretation of ground anchor residual load monitoring should firstly consider the normally expected decay in anchor tension load as given above. Any departure from the limits placed on this tension load decay should be considered abnormal and further more detailed monitoring carried out. Similarly anchors may gain in tension load although this is generally less common than the time dependent loss of tension force. Where the anchor tension ratios are outside the acceptable performance range given above, additional intense monitoring should be immediately implemented to isolate:-

- (a) whether the results are one-off measurements which tend to show a monitoring inaccuracy, failure of equipment, reference points, etc.;
- (b) whether the results are sustained as a trend.

The residual ground anchor tension load monitoring results may be interpreted according to Table 3 below.

TABLE 3 : SUMMARY OF INTERPRETATION OF GROUND ANCHOR RESIDUAL LOAD MONITORING

Load Condition	Contributing Factors to changed state of Residual load	Identification Procedures for isolating contributing factors
Increase in residual load above the lock-off load achieved at the time of ground anchor construction	.A rise in the level of the static groundwater table above the level taken for retaining wall design	Check the level and location of the groundwater table behind the ground anchored retaining wall
	.Thermal change of the ground anchored mass behind the retaining wall	Check the residual ground anchor loads and compare the results between summer and winter monitoring
	.Face deformation causing an elongation of ground anchor and increase in tension load.	Check face deformation by precise survey and other acceptable and accurate techniques.
Decrease in the residual load below the lock-off load achieved at the time of construction	.A drop in the level of the static ground water table	Check the level of the ground water table
	.Thermal change of the ground anchored mass	Check the residual load in ground anchors and compare results between summer and winter
	.Anchor creep	Reload the ground anchor and carry out suitability testing in accordance with F.I.P. 2/7, 1982.
	.Anchor Corrosion	Failure likely.

Where an unacceptable trend in the residual ground anchor load is detected then the intensity of monitoring must be increased and this should be done in conjunction with the advice of a geotechnical engineer experienced in ground anchor design and monitoring. The frequency of residual load monitoring should be increased according to Table 4.

TABLE 4 : NON CONFORMING PERFORMANCE - INCREASED MONITORING

Case	Frequency of Residual Ground Anchor Load Monitoring	Other
a) Change in the level of the static ground water table	1 month intervals	Check water levels weekly
b) Thermal change of ground anchored mass	3 month intervals	-
c) Face deformation	Weekly intervals increasing to 1 month intervals	Deformation survey daily increasing to monthly
d) Anchor creep	Return to construction monitoring	-
e) Anchor corrosion	Twice Weekly followed by construction monitoring	Implement remedial construction immediately

9.0 FACE DEFORMATION

Outwards movement of the retained face may be indicated by increased levels of tension load in ground anchors. Situations can develop however, where face deformation has failed anchors or where movement occurs on planes which extend behind the anchored zone. For these cases there may not be any measurable increase in ground anchor tension loads. Face deformation could have major implications on both the development for which the retaining wall was built and any other development reliant on the retaining wall for support. As a consequence any change in the face position detected by survey monitoring should be treated with concern and urgency. In reviewing the survey measurement results however, trends should be established rather than interpretation of absolute measurements, as despite the best effort there remains a degree of inaccuracy in measuring face deformation by survey techniques. Face deformation equivalent to $\pm 0.001 H$ where H is the height of the retaining wall is suggested as acceptable before more frequent face position monitoring is required.

In order to obtain a generous data base on which to establish the trend of face movement, the frequency of survey monitoring should initially be increased to daily and then be carried out as required by the appropriate engineering advisers. If permanent outwards deformation is established, then investigative and remedial works should be implemented immediately.

The frequency of deformation monitoring may be relaxed to that given in Table 1 if the trend is not sustained or the deformations are considered within acceptable tolerances.

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APPLICATION FOR MEMBERSHIP

of

New Zealand Geomechanics Society

A TECHNICAL GROUP OF THE INSTITUTION OF PROFESSIONAL ENGINEERS NEW ZEALAND

The Secretary
The Institution of Professional Engineers New Zealand
PO Box 12-241
WELLINGTON

I believe myself to be a proper person to be a member of the N.Z. Geomechanics Society and do hereby promise that, in the event of my admission, I will be governed by the Rules of the Society for the time being in force or as they may hereafter be amended and that I will promote the objects of the Society as far as may be in my power.

I hereby apply for membership of the N.Z. Geomechanics Society and supply the following details:

NAME _____
(to be set out in full in block letters, surname last)

PERMANENT ADDRESS _____

QUALIFICATIONS AND EXPERIENCE _____

NAME OF PRESENT EMPLOYER _____

NATURE OF DUTIES _____

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

I wish to affiliate to:

<u>International society for Soil Mechanics and Foundation Engineering</u>	(ISSMFE)	Yes/No	(\$11.00)
<u>International Society for Rock Mechanics</u>	(ISRM)	Yes/No	(\$15.00)
<u>International Association of Engineering Geology</u>	(IAEG)	Yes/No	(\$ 7.00)
			(with Bulletin \$19.00)

Signature of Applicant _____

Date _____ 19 ____

NB: Affiliation fees are in addition to the basic Geomechanics Society membership fee of \$20.00 which is reduced to \$16.00 if member of IPENZ.

PLEASE DO NOT SEND FEES WITH THIS APPLICATION. AN ACCOUNT WILL BE SENT ON YOUR ACCEPTANCE INTO THE SOCIETY

Nomination:

I _____ being a financial member of the N.Z. Geomechanics Society hereby nominate _____ for membership of the above Society.

Signed _____ Date _____ 19 ____