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N.Z. GEOMECHANICS NEWS

No. 46

DECEMBER 1993

A NEWSLETTER OF THE N.Z. GEOMECHANICS SOCIETY

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CONTENTS

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NOTES FOR CONTRIBUTORS

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

- Technical contributions can include any of the following:
  - Technical papers which may, but need not necessarily be of a standard which would be required by the international journals and conferences
  - Technical notes
  - Comments on papers published in Geomechanics News
  - Descriptions of geotechnical projects of special interest
- General articles for publication may include:
  - Letters to the NZGS
  - Letters to the Editor
  - Articles and news of personalities

Submission of text material in camera-ready format is not necessary though typed copy is encouraged. Diagrams and tables should be of size and quality for direct reproduction. Photographs should be good contrast black and white gloss prints and of a suitable size for mounting to magazine format. Authors and other contributors must be responsible for the integrity of their material and for permission to publish.

Tim Sinclair
EDITOR

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THIS IS A REGISTERED 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 the newsletter. The basic subscription rate is $36.00 and is supplemented according to which of the international societies, namely Soil Mechanics ($16.00), Rock Mechanics ($16.00) or Engineering Geology ($37.00) the member wishes to be affiliated. Members of the Society are required to affiliate to at least one International Society.

Editor: T.J.E. Sinclair
P.O. Box 5271
AUCKLAND
Phone: (09) 377 1865
Fax: (09) 307 0265

Advertising: S. Palmer
P.O. Box 3942
WELLINGTON
Phone: (04) 4737 551
Fax: ()4) 4737 911

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EDITORIAL

Recent issues of Geomechanics News have given more than a little space to the Building Act. This has now been in force for almost a year and geotechnical engineers are beginning to realise how life has changed because of it. In particular, the provisions of Section 36, subsection 2 appear to offer more security to designers and councils by use of "caveat emptor". This clause, in effect, says that, provided a building does not increase the risk of damage (e.g. landslip, erosion, subsidence), a building consent may be granted even if there is an existing risk to the site, but with the condition that the risks are identified on the certificate of title. For example, building at the base of a cliff or slope judged to be of suspect stability could be permitted under these provisions. Similarly, consent for building on an existing landslip could be granted on the basis that pile or pole foundations would tend to have a favourable rather than an unfavourable influence on the land stability.

This is a radical departure from historical philosophy. For decades, the buyer/builder has been protected from the consequences of his own judgement or the judgement of others but has had to forgo the right to accept certain risks on his own behalf. Now, suddenly ..... "let the buyer beware!".

What does this mean to us? As before, geotechnical engineers will be required to make judgements on the security of a site but now we may also need to advise whether the "building work itself will ..... accelerate, worsen or result in ..... " various possible consequences to the proposed development and particularly to "other property". Are we exposing ourselves in doing this to more, less or the same liabilities? Will engineers and councils (Territorial Authorities) fall back on Section 36(2) as a matter of course? Will lawyers experience a sudden drop in their work load?

Tim Sinclair
EDITOR

(NOTE: Section 36 of the Building Act 1991 is reproduced in full later in this issue of Geomechanics News).

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REPORT FROM THE MANAGEMENT SECRETARY

1. MEMBERSHIP

The following new members are welcomed to the Society:

D. Barrell
T. Coote
A. Cowbourne
P. Denton
G. Gribben
J. McClean
P. Salter
T. Schanz
A. Walker

The Management Committee is currently preparing proposals for a student grade of membership. To encourage students to join the Society, fees for this grade of membership would be substantially less than the ordinary grade.

2. MANAGEMENT COMMITTEE

The Management Committee held its last meeting of the year on 12 October 1993 and will not meet again until May 1994, when the Symposium on "Geotechnical Aspects of Waste Management" will be held in Wellington. The AGM of the Society will also be held at this time. Traditionally, the AGM has been held in February during the IPENZ Conference. However, the Society will not be participating in the 1994 IPENZ Conference in Nelson because of the close proximity to the May Symposium.

Stuart Palmer and Dick Beetham have been leading the effort to organise the Wellington Symposium. Most of the initial organisation is complete and details are included elsewhere in the Newsletter. If takes considerable time and effort to organise Conferences or Symposium (as those who have previously been involved will know) and I would like to acknowledge the effort by Stuart and Dick and their subcommittee in Wellington for their work to date.

3. LIMIT STATE DESIGN

Prof. M. Pender and T. Matuschka have met with SESOC regularly over the last few months to resolve the problems with limit state design of foundations associated with NZS 4203:1992. Prof. Pender has largely been responsible for the preparation of guidelines on the use of limit state design and a paper is included in this Newsletter. It is proposed to conduct a workshop at the main centres early next year to explain the concepts of limit state design to foundation design problems, to provide guidelines on the appropriate approach to take in New Zealand and to provide worked examples.

4. STUDENT AWARDS

The Management Committee has discussed and is currently preparing proposals for an annual Students' Award for presentation of a short paper. Initially, presentations in Auckland and Christchurch in July/August are envisaged. The winners would receive a cash prize and certificate.

5. COMPOSITION AND FUNCTIONS OF MANAGEMENT COMMITTEE

The composition and function of the Management Committee were discussed at the last Management Committee Meeting. A small subcommittee is currently reviewing these aspects and will report back in the form of a discussion paper, which is intended to be published in the next issue of Geomechanics News. It is important that such a review is undertaken to ensure that the Management Committee is best serving the needs of its members.

Trevor Matuschka
<u>MANAGEMENT SECRETARY</u>

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LOCAL GROUP ACTIVITIES

1. AUCKLAND BRANCH

Below is a report of the Auckland Branch meeting held on 17 August 1993:

Pauanui Canal Development:

Aidan Nelson and Philip Kelsey of Earthtech Consulting Ltd provided an informative overview of the geotechnical and environmental aspects of the development by Hopper Brothers. The site is unique on the east coast of the North Island because of the ground elevation (0.5 to 1.0 m above high water level) and the small tidal range (1.2 m). It therefore lent itself to a canal development. The site is underlain by 10 m of sands, a thin brown ash layer (aquiclude) and further sands. Investigations relied principally on the Cone Penetration Testing. These showed potentially liquefiable loose sands from 0 to 3 m depth, underlain by medium dense sands.

Following large scale trial excavations and dewatering trials it was decided to carry out the construction by dewatering and using conventional earthmoving plant rather than using a dredge. The canals were formed by cut and the islands by fill. The fill was used to surcharge the potentially liquefiable sands beneath the islands. Sufficient increase in density was usually achieved in one to two days.

The effect of dewatering on the adjacent Pauanui Spit aquifer (used for water supply) was critical. Environment Waikato placed stringent requirements on drawdown of the shoulder of the aquifer. Therefore water pumped from the dewatering well point and sump system was used to recharge the aquifer along the boundary between the site and the spit. The construction was successfully carried out by careful monitoring and adjustment of recharge. Rain also provided recharge at several critical occasions.

The canals have a beach area below a revetment wall. A hand placed stone wall was used rather than a crib wall because it compared favourably in cost and was superior aesthetically.

Local residents inspired a special consent condition for the development which required the sand excavated from the access channel (40,000 m^3) to be put back into the marine environment. The adjacent estuary beach has therefore been replenished and stockpile created for future replenishment.

Geoff Farquhar
<u>AUCKLAND BRANCH CONVENOR</u>

2. WELLINGTON BRANCH

The Wellington Branch had an extremely active time in the second half of 1993. Speakers and topics were:

July 1993:	Tim Logan, Works Consultancy Services, Groundwater Temperature Profiling in Landslides
August 1993:	Site visit to Museum of New Zealand Site to view dynamic compaction. (In conjunction with the Structural Group)
September 1993:	Report on "Hokkaido-Nansei-Oki" earthquake. (Combined meeting with NZSEE and Structural Group)
	Professor John Hutchinson of Imperial College London, Three Large Peruvian Landslides, (A precis of the talk is presented below).
October 1993:	Harry Poules of Coffey Partners, Sydney, Pile Behaviour - Theory and Application
November 1993:	Dr Fred Kulhaway of Cornell University, on Evaluation of Static Soil Properties. (Combined meeting with Wellington, IPENZ)

Next year promises to get of to a good start with John Berrill, presenting the NZGS lecture in March.

Branch members have also been busy preparing for the seminar on the geotechnical aspects of Waste Management next May. A separate report is presented elsewhere.

Peruvian Landslides
(Wellington Branch Meeting: 15 September 1993)

The speaker was Professor John Hutchinson of Imperial College, London, who spoke on three large landslides in the Peruvian Andes.

The first landslide described was the catastrophic rock avalanche/debris flow which took place from Mt Huascaran (6656 m high) in 1970. It originated from a near-vertical, 1,000 m high face of granodiorite capped by glacier ice. The failure was triggered by a 7.7 magnitude earthquake with an epicentre offshore, 130 km to the west. The failure volume was 50-100 x 10^6 m^3 and the estimated average velocity 280 km/hour. The debris flowed chiefly down a side valley of the Rio Santa but spilled over a saddle into an adjacent valley, overwhelming the town of Yungay. About 20,000 people were killed.

The failure was anticipated, in principle, by a team of glaciologists from MIT who were climbing on Mt Huascaran after a similar, but smaller, event in 1962. They published an article to this effect in the Lima newspaper of September 1962, predicting that the next slide would overwhelm Yungay. After the 1970 catastrophe, a USGS team mapped a pre-Columbian rock avalanche limit 40 m or more higher than the 1970 limit, again showing that the site of Yungay was exposed to hazards of this type.

The second landslide discussed occurred in 1974 on the Rio Mayunmarca. The landslide travelled over 8 km horizontally and 1.9 km vertically. The initial slide appears to have eroded material downslope and the total volume of material involved in sliding was about 1.6 x 10^9 m^3. A debris dam about 175 m high and extending to 1 to 2 km downstream was formed. Notable features of the landslide were the lack of water and the comparatively fine nature of the debris. When the debris dam was overtopped it eroded extremely rapidly due to its fine grading, with a short-lived peak flow of about 10,000 m^3/sec. The initial wave of water just downstream of the dam was allegedly over 30 m high. Reports of a wave in the river downstream continued to be received for the next 2-3 weeks and the wave was still identifiable when it reached the Amazon River far downstream.

The Mayunmarca landslide was predicted 9 months before its occurrence by a mining geologist working in the area. He wrote a report to his company recommending the evacuation of Mayunmarca but no action was taken.

The third landslide discussed occurred in the right bank of the Tablachaca Reservoir, just upstream of the Tablachaca Dam which had been completed in about 1972. The left abutment of the Dam was founded in a relatively large expanse of in-situ phyllitic rock but the right abutment was banded in only a small outcrop of such material with the landslide boundary less than 10 m upstream of the dam face. The river had an extremely high bedload which would have filled the reservoir in less than a year. Four gates were therefore constructed in the lower part of the dam, one of which was always open to pass gravel and sediment. Operation of these gates, combined with an S-bend in the river upstream of the dam, led to the formation of a large eddy along the toe of the landslide. The velocity of this eddy at the foot of the landslide was about 1 m/sec (upstream) with associated severe toe erosion. Landslide monitoring records showed levels of movement which were tending to increase successively from wet season to dry probably chiefly because of this continuing toe erosion. The decision was therefore made to stabilise the landslide.

Five stabilisation measures were employed. The primary stabilisation measure was the building of a rock buttress to replace the material eroded by the eddy and to further counterweight the toe of the landslide. The post 1972 alluvium which had accumulated in the floor of the reservoir, was found to be liquefaction-prone, and had to be densified by a combination of vibroflotation and stone column construction before placement of the rock buttress could start. Drainage of the slide was effected by driving galleries into the undisturbed phyllite beneath the basal surface and drilling diagonally upwards from these to penetrate and drain the landslide mass. It was not feasible to extend the buttress right to the dam face as this would have meant losing one of the four gates for passing the river bed load. Therefore, about 400 ground anchors had to be installed at 3 levels just upstream of the downface. The final stabilisation measure was to realign the river upstream of the dam to improve the path of water flow into the intake area. As the dam generates about half of Peru's power it was crucial to secure the landslide and thus the dam. The total cost of stabilisation works was about US$35,000,000.

Ian McPherson
<u>WELLINGTON ACTIVITIES CO-ORDINATOR</u>

3. OTAGO BRANCH

As newly appointed Otago Branch co-ordinator (thanks to Dick Beetham and Dave Bell), this is my first report. What happened prior to my appointment is now history, and because I didn't receive notice of any events in the past year, I presume that not a lot happened. So, awake the sleeping dragon!

John Hutchinson (Imperial College) visited Dunedin in Mid-October, and as well as visiting well known landslides in the area, he presented an excellent talk on landslide zonation and landslide remedial measures. The talk was well received by the members and invited guests. Thanks to Dave Bell and Dave Stewart (duly appointed deputy).

We are hoping that an interesting calendar of events will materialise from the Scotch mist in 1994. If anyone has any suggestions .....

Phil Glassey
<u>OTAGO/DUNEDIN BRANCH</u>

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NEW ZEALAND NATIONAL SOCIETY FOR EARTHQUAKE ENGINEERING

Reconnaissance Team Pool Members

We have recently been approached by the New Zealand National Society for Earthquake Engineering (NZNSEE) to nominate people with geotechnical expertise to join their pool of people available to undertake reconnaissance visits following damaging earthquake. NZNSEE operates and maintains a pool of volunteers representing different engineering and scientific interests, willing to undertake investigative reconnaissance visits to sites of damaging earthquakes.

Ideally, the pool should comprise members covering all such interests, with, where possible, a reasonably even geotechnic spread around the country. However, the current list is short on those with geotechnical expertise. NZNSEE has asked the NZGS to consider nominating up to 10 members willing to join the pool list. Consequently, we would like to receive applications from those interested in joining the pool list. From personal experience I know that participation in the reconnaissance teams can be very rewarding.

Please send all applications of interest to the Secretary, NZGS, P.O. Box 12-241, Wellington. Include name, address, occupation, professional qualifications, primary area of expertise and any other skills which may be useful in a reconnaissance. Applications should be received by 28 February 1994.

T. Matuschka
<u>SECRETARY, NZGS</u>

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IAEG LIST OF MEMBERS

The IAEG has finally printed the 1993 edition of the List of Members of the International Association of Engineering Geology.

Taking into consideration the high printing costs and the mailing expenses, the Executive Committee has decided to publish only a total of 2,000 copies and to supply the National Groups with a number of copies smaller than the number of their members.

This same thing was done in 1985, when the last version of the List of Members was published, and the idea is that each National Group should announce to all its members, through a circular letter, that the Secretariat of the National Group has a number of copies of the list which will be sent by the Group only to the members who request it in writing.

Fifty copies of the List of Members have been sent to IPENZ. Anyone wishing to obtain a copy should write to John Eade at IPENZ, P.O. box 12241, Wellington.

EDITOR

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NEW JOURNAL FOR GEOTECHNICS

In January 1994, the Institution of Civil Engineers will launch a quarterly journal to be published as part of the ICE Proceedings, called "Geotechnical Engineering".

The journal will complement "Géotechnique" and "Ground Engineering" magazine. It will have the purpose of bringing modern geotechnical engineering in terms of design and practice to the attention of ICE members and engineers throughout Europe. It will therefore have a very large potential readership.

An editorial panel was formed on 3 March 1993 consisting of national and international geotechnical engineers. The first Honorary Editor of the journal will be Professor Clayton of the University of Surrey.

The editorial panel would like to cover the full scope of geotechnics. Topics for papers might include, for example: technical design, recent practical developments, advances in software, laboratory testing, case records and legislation.

Papers need not be of entirely new material; the panel will be happy to consider reprinting papers which have been published elsewhere if it is felt that there will be a benefit from bringing the information to the attention of a wider or different readership. Indeed, the journal may from time to time reprint classical papers which are perhaps now difficult to obtain.

There is no need to wait! Papers can be submitted NOW to Mr Kevin Aston, Papers and Journals Department, The Institution of Civil Engineers, 1-7 Great George Street, London SW1P 3AA.

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THE FIRST AUSTRALIA-NEW ZEALAND YOUNG GEOTECHNICAL PROFESSIONALS CONFERENCE

(9th to 12th February 1994

We are pleased to announce that the following applicants have been selected to receive the EQC Young Geotechnical Professionals Award to enable them to attend the first Australia-New Zealand Young Geotechnical Professionals Conference to be held in Sydney:

Mr P. Bosselmann	Auckland
Mr P. Brabhaharan	Wellington
Mr M. Fraser	Auckland
Mr S. Terzaghi	Auckland
Mr R. Vreugdenhil	Christchurch

The value of each award is $600 and the NZGS is extremely grateful to EQC for providing this sponsorship. In addition, the successful applicants will have their conference registration fees fully paid for by NZGS.

The successful applicants were of high calibre and we are certain they will represent us well.

It is anticipated that this Conference will be held on a regular basis so that the opportunity exists for other Young Geotechnical Professionals to attend in the future. Such conferences have proved very successful in Europe and we look forward to reporting on the Sydney conference in our next Newsletter together with the papers which the New Zealand attendees will be presenting.

T. Matuschka
<u>SECRETARY</u>

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PUBLICATIONS OF THE SOCIETY

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

	LIST PRICE		REDUCED PRICE TO MEMBERS
	MEMBERS	NON MEMBERS	
Australia-NZ Conferences on Geomechanics			
Proceedings of the Sixth Australia-NZ Conference on Geomechanics, Christchurch, February 1992	$100.00	$100.00	$50.00
Proceedings of the Third Australia-NZ Conference on Geomechanics, Wellington, May 1980	$ 20.00	$ 30.00	$10.00
Proceedings of the Second Australia-NZ Conference on Geomechanics, Brisbane, July 1975	$ 25.00	$ 25.00	N/A
NZ Geomechanics Society Symposia			
Proceedings of the Auckland Symposium "Groundwater and Seepage", May 1990	$ 25.00	$ 45.00	$10.00
Proceedings of the Hamilton Symposium "Piled Foundations", September 1986	$ 20.00	$ 25.00	$10.00
Proceedings of the Alexandra Symposium "Engineering for Dams and Canals", November 1983 (with joint Symposia with NZSOLD)	$ 40.00	$ 50.00	$10.00
Proceedings of the Palmerston North Symposium "Geomechanics in the Urban Planning" May 1981	$ 20.00	$ 20.00	N/A
Proceedings of the Wanganui Symposium "Using Geomechanics in Foundation Engineering", September 1972 (xerox copy)	$ 8.00	$ 10.00	N/A
Other Publications			
Guidelines for the Field Description of Soils and Rocks in Engineering Use	$ 10.00	$ 13.00	N/A
"Stability of House Sites and Foundations - Advice to Prospective House and Sections Owners"	$ 1.00	$ 1.00	N/A
IEA Guidelines for Provision of Geotechnical Information, etc.	$ 10.00	$ 10.00	N/A
Back dated issues of Geomechanics News	$ 0.50	$ 0.50	N/A

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ADVERTISING IN GEOMECHANICS NEWS

Geomechanics News is continued to be published approximately twice a year and distributed to the Society's 400 members. Charges for advertising are as follows:

ADVERTISEMENT LOCATION	RATE (EXCLUDING GST)	
	SINGLE ISSUE $	TWO ISSUES $
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Inside Cover (Front or Back)	200	320
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The Publications Officer
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P.O. Box 3942
WELLINGTON

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

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PAPERS PUBLISHED BY SOCIETY MEMBERS

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

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

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

GEOMECHANICS SOCIETY LIST OF PAPERS PUBLISHED BY MEMBERS

AUTHOR(S) ______________________________________

TITLE: ______________________________________

____

WHEN PUBLISHED: ______________________________________

WHEN PUBLISHED: ______________________________________

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THE BUILDING ACT 1991: SECTION 36

Limitations and Restrictions on Building Consents

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

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

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

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

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

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

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

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

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

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

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

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

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

(4) *Where -*

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

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

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

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

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

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

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

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

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

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FORTHCOMING CONFERENCES

1994:

January 3:
New Delhi, India. International Symposium on Underground Construction in Soft Ground.

January 5-10:
New Delhi, India. "XIII International Conferences on Soil Mechanics and Foundation Engineering.

January 27-28:
Reno, Nevada, USA. Symposium on Dynamic Geotechnical Testing.

June 1-3:
Austin, Texas. First North American Rock Mechanics Symposium (NARMS).

June 13-15:
Brugge, Belgium. 5th International DFI Conference.

June 16-18:
Texas, A & M University, USA. ASCE Speciality Conference, Settlement '94.

June 10-15:
Edmonton, Canada. First International Congress on Environmental Geotechnics.

August 31-September 2:
Singapore. International Conference - CENTRIFUGE '94.

September 5-9:
Lisbon, Portugal. 7th IAEG Congress.

September 5-9:
Saratov, Russia. 4th International Conference on Problems of Pile Foundation Engineering.

September 5-9:
Stara Lesna (High Tatras), Slovak Republic. 8th European Young Geotechnical Engineers' Conference.

September 12-14:
Sapparo, Japan. International Symposium on Pre-failure Deformation Characteristics of Geomaterials - Measurement and Application (IS-Hokkaido).

September 14-16:
Mamaia Constantza, Romania. 10th Danube-European Conference on Soil Mechanics and Foundation Engineering.

November 5-10:
Fax do Iguacu, Brazil. 10th Brazilian Congress on Soil Mechanics and Foundation Engineering.

1995

February 14-16:
New Orleans, USA. The Geoenvironment 2000.

April 2-7:
St Louis, USA 3rd International Conference on Recent Advances in Geotechnical Engineering and Soil Dynamics.

May 8-11:
Stuttgart, Germany. World-Tunnel Congress and Stuva-Tagung '95.

May 10-12:
Hiroshima, Japan. International Symposium on Compression and Consolidation of Clayey Soils (IS-Hiroshima).

May 28-June 1:
Copenhagen, Denmark. 11th European Conference on Soil Mechanics and Foundation Engineering.

August 29-September 2:
Beijing, China. Tenth Asian Regional Conference on Soil Mechanics and Foundation Engineering.

September 25-29:
Tokyo, Japan. 8th International Congress on Rock Mechanics.

November 14-16:
Tokyo, Japan. First International Conference on Earthquake Geotechnical Engineering (US-Tokyo).

December:
Cairo, Egypt. XI African Regional Conference on Soil Mechanics and Foundation Engineering.

1996:

June 17-21:
Trodheim, Norway. 7th International Symposium on Landslides.

1997:

September 6-12:
Hamburg, Germany. XIV International Conference on Soil Mechanics and Foundation Engineering.

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ARTICLES AND TECHNICAL PAPERS

A LINE OF SPIKES

B.M. Horide, BE, MIPENZ
Waste Management NZ Ltd
Redvale Landfill Engineer

During 1993, Waste Management NZ Ltd (WMNZ) monitored a slowly moving landslip. The instrumentation included a line or row of "spikes". The spikes were 400 mm lengths of 12 mm diameter reinforcing steel bar, sharpened to a blade-shaped point at both ends. The spikes were hammered 300 mm into the ground at intervals on a straight line through the middle of the slip in the direction of the slip movement. The extent and direction of the slip had been determined by mapping of surface cracks and ground surface contours. The horizontal distances between spikes were all approximately 4.5 m which could be measured to within ± 1 mm with a 5 m steel pocket tape.

The distances between the spikes were measured periodically and the data analysed along the following lines:

(a) If the sum of the inter-spike distances changed between successive readings, then it could have been assumed that the row of spikes did not fully encompass the slip zone. If the sum increased (extension), then the slip zone probably would have extended beyond the low end spike. At the WMNZ site, the sum decreased by 25 mm over 150 m (compression). However, an inclinometer at the up-hill limit showed no movement, suggesting that the change was within the accuracy of measurement.

(b) The sum of compressions (negative changes), or the sum of extensions, could have indicated the distance of movement at the centre of the slip mass. However, this was incorrect where some of the compressions were in the tension zone at grabens.

(c) The centre of the slip was determined as the location where the sum of inter-spike changes down-hill from this location was equal to the sum up-hill. This sum was then used as the distance of movement and to calculate the rate of movement.

The advantages of this system were:

- the slip zone was identified to within ± 5 m
- the rate of slip movement was readily calculated
- the spikes provided full coverage (i.e. no gaps in the long section through the slip) whereas extensometer instruments would have been costly for similar coverage
- the system still worked if a spike was knocked over, by simply putting the spike back as near as practical to the same position
- multiple scarps were detected

The disadvantages of this system were:

- the accuracy of measurement would possibly have been inadequate for slower movement rates of less than say 1 mm/week
- two persons were needed for measurements
- frequent measurements were needed for good results

In conclusion, the "line of spikes" method should be considered as a cost-effective method to monitor slowly moving land slips.

****

DOES K_0 EXIST ?

T.J.E. SINCLAIR

1.0 INTRODUCTION

The concept of "Active" or "Passive" pressure is well understood. If a wall moves, a slip surface is assumed to develop and the active or passive force on the wall can be calculated by analysis of static equilibrium. The situation is easy to visualise (see Figure 1).

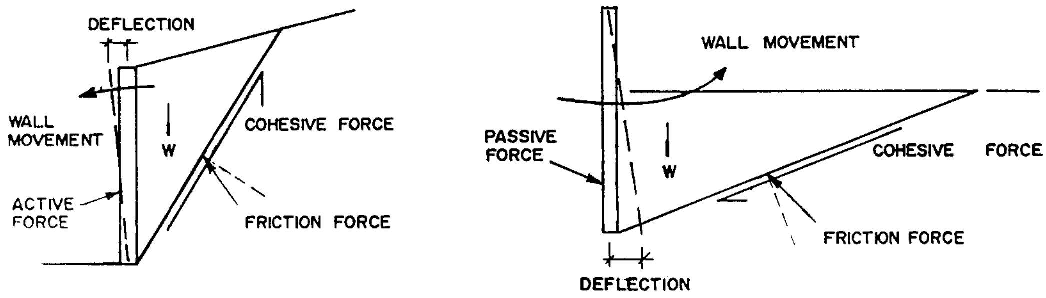


Figure 1: The Usual Physical Meaning of Active and Passive Forces

The Rankine stress state is a little more difficult to visualise. It assumes that a semi-infinite soil (cohesionless) mass with a level surface is in a state of "yield" by tension (active state) or compression (passive state). The stress state is analysed either by considering the static equilibrium of a small prismatic element or, more simply, by using the Mohr circle construction (see Figure 2).

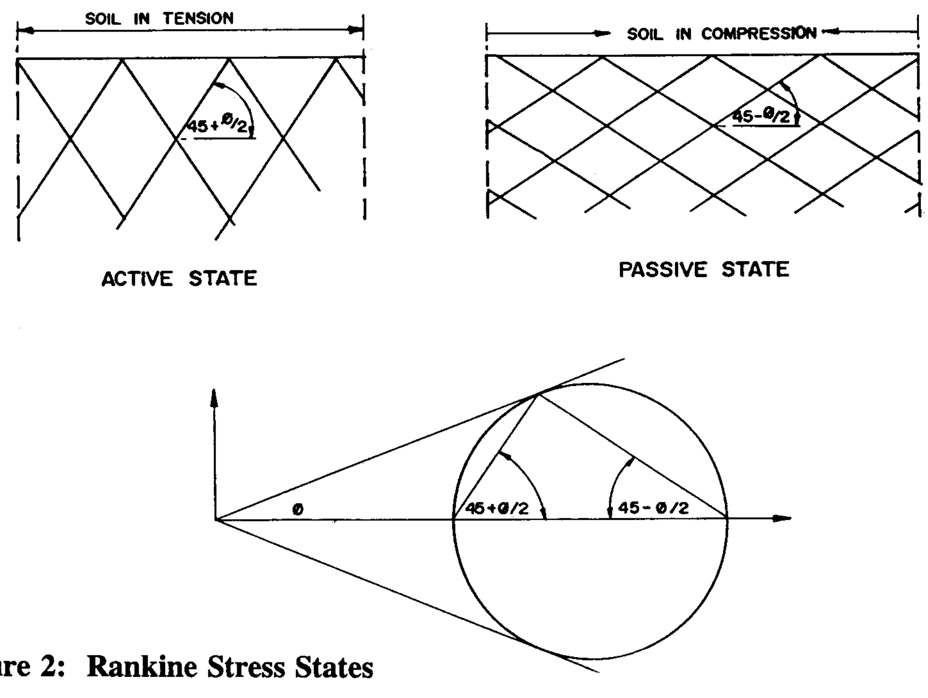


Figure 2: Rankine Stress States

Despite this more abstract concept, the Rankine coefficients tend to be the most commonly used in determining design values for lateral earth pressures:

$$K_a = \frac{1-\sin\phi'}{1+\sin\phi'} \quad \text{and} \quad K_p = \frac{1+\sin\phi'}{1-\sin\phi'}$$

The Rankine values tend to be conservative because they take no account of friction between wall and soil. The Rankine values equate to those derived from a limit-state stability approach (Figure 1) if the wall is considered perfectly smooth. The differences are generally of little consequence for active conditions but Rankine values can be unreasonably conservative for passive conditions. For example, for a rough wall with a level sand or gravel backfill, the Rankine method would overpredict active forces by about 25% but would underpredict the passive capacity by about 250%.

In summary, therefore, it is obviously best to treat the active/passive earth pressure problem in terms of limit state equilibrium (Figure 1). Not only does it lead to more realistic design values, but is also easier to understand the physical model. Most engineers, whether they regard themselves as being geotechnical engineers or not, understand the active/passive concept and are comfortable with designing on the basis of the simple theory summarised above.

However, the concept of "earth pressure at rest" is not so well understood. There are many misconceptions which can lead to costly overdesign, dangerous underdesign and sometimes "right-for-the wrong reason". The paper is intended to highlight some of the misconceptions and to pose the questions:

- Do we ever get "at-rest" pressures ?
- How do we measure or estimate "at-rest" pressure ?

2.0 "AT-REST" SITUATIONS

The text books tend to view the "at-rest" situation in simplistic terms. If the wall is rigid and unyielding then earth pressures on the wall must be those of the "at-rest" case. But this disregards the methods and sequences of construction.
Some common "at-rest" situations and construction methods can be summarised as follows:

End result	Method of Construction
• Stiff wall with props or anchors	• Soldier piles and lagging
• Very stiff cantilever	• Continuous or separated bored piles
	• Batter back and backfill
	• Slurry trench diaphragm wall

All these methods involve some stress relief. The finished product may be unyielding but certainly the soil will have yielded to some extent during excavation and construction.

3.0 FACTORS NEGATING "AT-REST" CONDITIONS

a) As mentioned above, most construction methods require some stress relief. At times, some portion of a cut face may be totally unsupported for a limited duration. Yet once a stiff wall is constructed in front of it and the intervening space backfilled, then it is assumed "at-rest" conditions apply. The question must be asked: Are "at-rest" pressures ever reinstated after excavation stress relief and, if so, by what mechanism?

b) A common design fallacy is to rely on passive pressure to resist toe movement of soldier piles, yet insist that "at-rest" pressures apply within the retained ground. It is well known that large deformations are required to mobilise even a third of the passive resisting force.

c) If there is any tendency for original earth pressures to be reinstated, then pressures on the wall will depend on the relative stiffnesses of original ground and backfill. If the backfill is significantly less stiff than the original ground, then little extra load will be transmitted to the wall.

4.0 WHAT IS K_0 ?

The definition of the coefficient of earth pressure at rest is:

$$K_0 = \frac{\text{Lateral effective stress}}{\text{Vertical effective stress}} = \frac{\sigma_h'}{\sigma_v'}$$

All geotechnical engineers would know this, yet a common error is to use K_0 with total stress and apply water pressures as well - unnecessarily conservative.

The usual assumption for estimating K_0 is to use the relationship proposed by Jaky (1944), namely:

$$K_0 = 1\text{-}\sin\phi'$$

But this applies only to normally consolidated soils, either sands or clays. It makes no allowance for effective cohesion because normally consolidated soils do not have effective cohesion. How often do we have such conditions? Most real soils are at least slightly over consolidated (or equivalent) near ground surface due to desiccation, secondary compression, cementation or conventional unloading.

Various researchers (e.g. Brooker & Ireland, 1965; Mayne & Kulhawy, 1982) have shown that K_0 increases with over consolidation ratio (OCR). This means that K_0 can be as much as 2 or 3 for highly over-consolidated soils (see Figure 3). In deciding whether to use such values in design, we should consider again the original meaning of the term K_0: It represents locked-in stresses due to vertical stresses which once applied. If a cut face is formed, the locked-in stresses obviously reduce to zero at the face. Highly over-consolidated soils would often stand unsupported almost indefinitely with vertical faces. It would clearly be a nonsense to build a wall in front of such a face, designed for a K_0 value of 2 or 3.

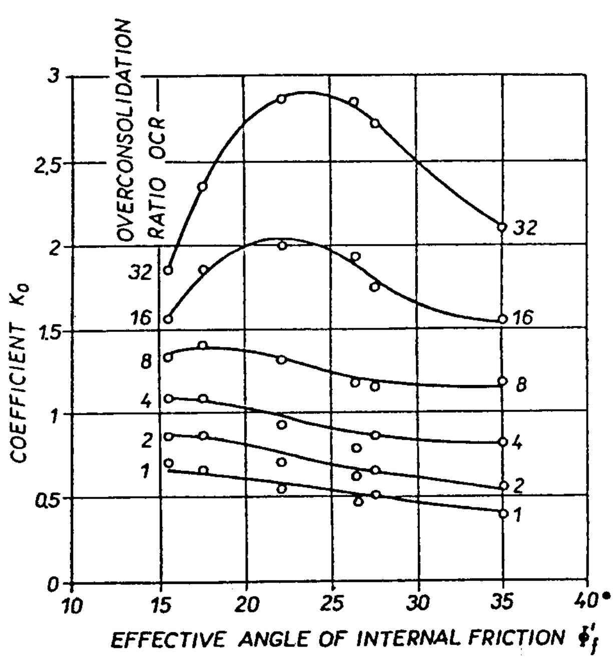


Figure 3: **Relationship between K_0 and OCR (After Brooker & Ireland, 1965)**

5.0 PROPOSED ALTERNATIVE TO K_0

Various researchers have shown how lateral earth pressures reduce to the active case with deflections of the retaining wall (e.g. see Figure 4). The authors proposed alternative approach is to start with the more recognisable and understood "active" case and allow for increases in design earth pressures if ability to deflect is limited. The curves of Figure 4 (Finn 1963) can be "inverted" to give the proposed design curves on Figure 5. A few simple checks will show that the zero-deflection (i.e. at rest) condition equates approximately to the Jaky value (K_0 = 1-sinϕ'). So, what's new? Very little: Just a more logical approach!

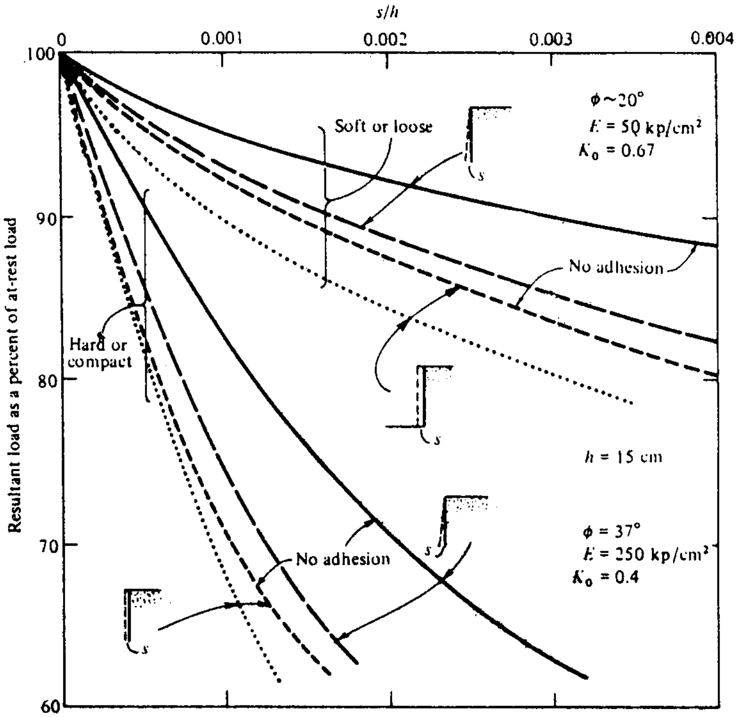


Figure 4: **Resultant lateral load vs. wall movement (after Finn, 1963)**

Also, it should be noted that Figure 5 is presented in terms of "pressures" and not in terms of the coefficient of active earth pressure (K_a). This is suggested to allow for the reduction in lateral pressures due to cohesion:

$$q_a = K_a \sigma_v' - K_{ac}c'$$

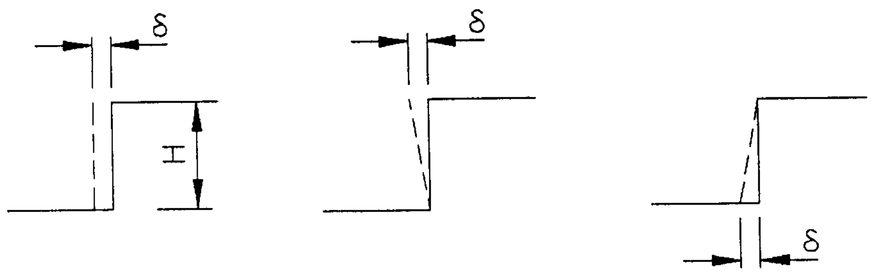
If there is true, sustainable cohesion, the active pressure may be zero and so the proposed design method would also allow zero pressures for the "at-rest" case. This gets away from the anomaly of very high theoretical K_0 values for highly over-consolidated soils and allows realistic design of temporary works on a total stress basis.

This alternative approach is suggested to make allowance for the "real world". It is not precise, nor conservative but factors of safety would cover the inaccuracies as they do in other "fuzzy" aspects of soil mechanics. There are few recorded cases of damage to rigid walls due to underestimated earth pressures. Notwithstanding this, the design should also take account of the actual construction situation. For example, earth pressures may be governed by the nature and compaction of backfill or true swelling of expansive soils or rocks.

6.0 CONSTRUCTION SITUATIONS

In deciding on the design earth pressures, either for temporary works or permanent structures, the mechanics of stress transfer should be thought through. Some examples are discussed below.

$$\alpha = \frac{\text{Lateral earth pressures/forces with limited wall deflection}}{\text{Lateral earth pressures/forces under active conditions}}$$



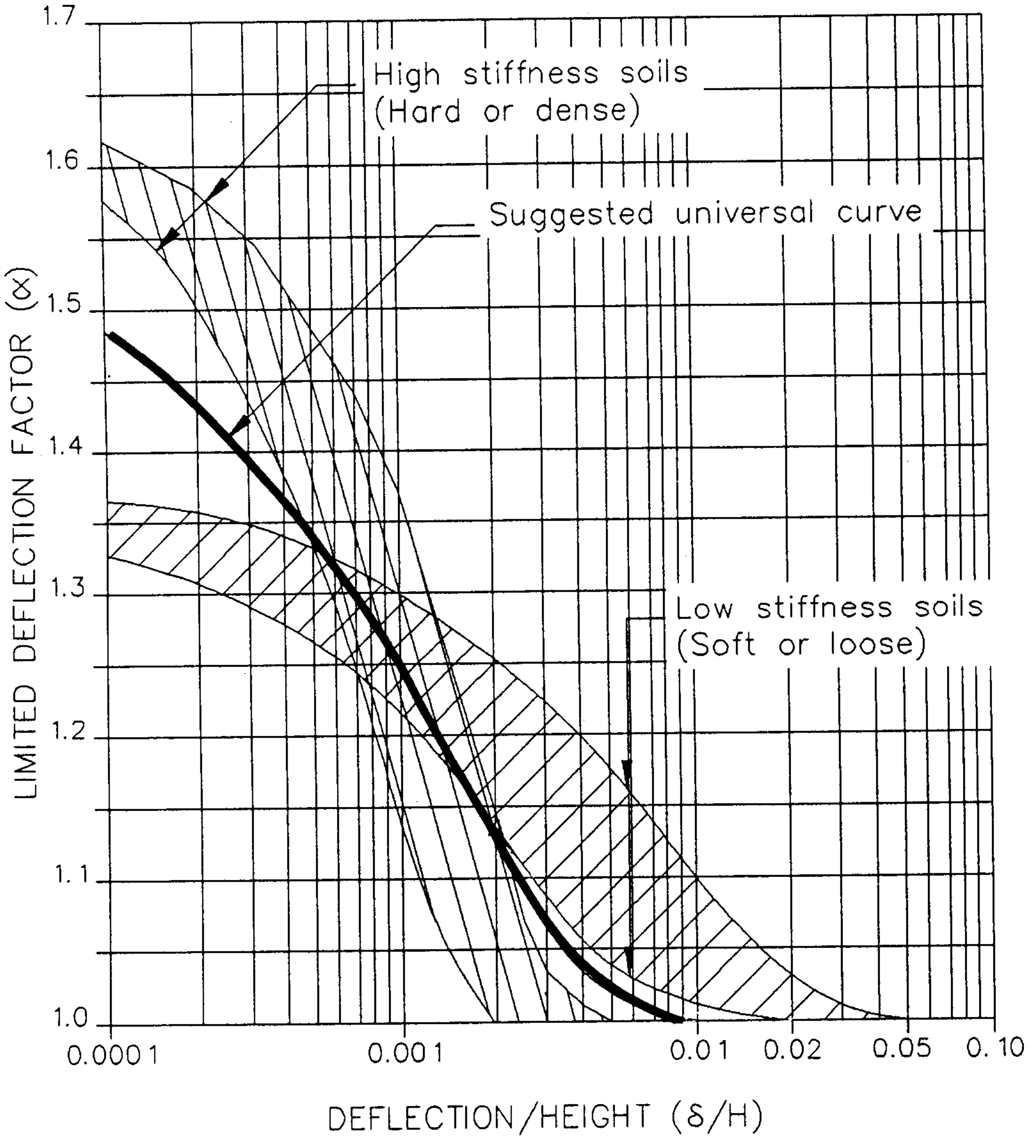


Figure 5: **Earth pressures on wall with limited deflection**

6.1 Hit and Miss

Hit-and-miss construction is usually carried out with some form of soldier pile and lagging (or permanent panel) and is probably the nearest situation to "at-rest" conditions. It usually involves vertical cut faces over short lengths, relying on arching between the ends. The soldier piles therefore carry extra load as if the whole face is being supported by a wall (see Figure 6). If the soldiers are stiff, propped or anchored, then they should be designed for the limited deflection pressures suggested above. The lagging or panels between the soldiers can be designed for active pressures or whatever pressure the backfill or grout may impose. This is <u>not</u> an additional load on the piles. Any backfill pressure will unload the arching effect by the same amount as the lagging/panels are loaded, thus keeping the loads on the soldiers constant.

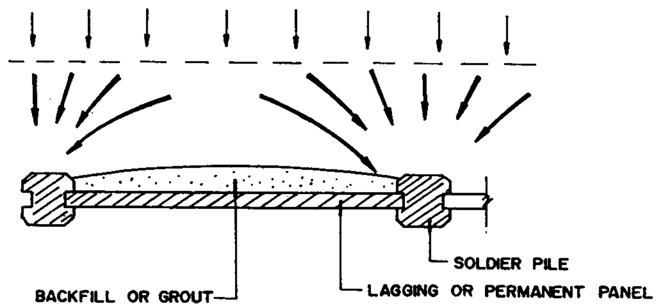


Figure 6: Arching between soldier piles

6.2 Braced Excavations

The earth pressure distributions on braced or propped retention systems are well known (e.g. Peck, 1969, see Figure 7). It should be noted that these all relate to the coefficient of active earth pressure (K_a) and not K_0.

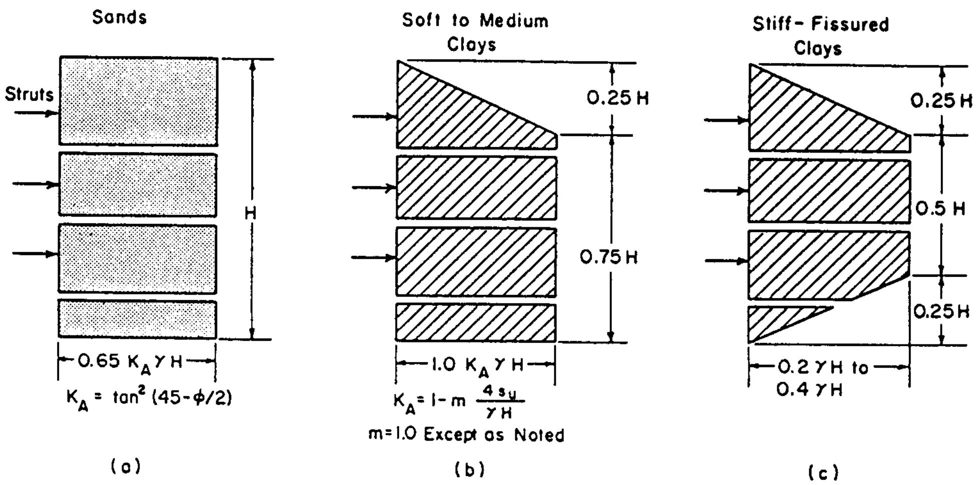


Figure 7: Earth pressures on braced excavations

6.3 Diaphragm Wall

The earth pressures on a diaphragm wall constructed by slurry trenching will be governed by the fluid pressures of slurry or wet concrete. The horizontal effective stresses could be as much as 1.5 to 2.5 times the vertical effective stresses.

6.4 Retaining Wall Near Rock Face

The lateral earth pressure acting on a wall built close to a stable rock face can be estimated using silo pressure theory which gives (Ref. Frydman & Keissar, 1987):

$$\sigma_x = \frac{\gamma b}{2\tan\delta}\left[1-\exp\left(\frac{-2Kz.\tan\delta}{b}\right)\right]$$

where	b	=	distance between wall and rock
	z	=	depth from top of wall
	K	=	a coefficient of lateral pressure
	γ	=	unit weight of granular backfill
	δ	=	angle of friction between fill and wall/rock

Frydman & Keissar (1987) suggest that the coefficient K may be taken as K_0 (= 1-sinϕ') for immovable walls and K_a for deflecting walls.

The theory applies only to lightly compacted, dry granular backfill. Heavy compaction and water pressures would obviously increase the lateral stresses.

6.5 Compaction effects

Various researchers (e.g. Broms, 1971: Ingold, 1979; Symons & Clayton, 1992) have shown how earth pressures from compacted granular fill can be estimated. Symons and Clayton (1992) point out that there is no established method of predicting horizontal stresses caused by compaction of cohesive soils. Pressure changes which occur subsequent to construction will depend on the placement water content in relation to depth.

7.0 CONCLUSIONS

The title of the paper is somewhat facetious. The concept of "K_0" is both necessary and useful, provided it is used with understanding of the soil mechanics. Certainly, in discussing and modelling stress history, we can't do without it.

However, this paper proposes that, for analysis of real engineering problems, K_0 is abandoned and earth pressures are determined with the starting point of active conditions, modified according to method of construction and ability for the structure to deflect.

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DEEP PIT DRAINAGE IN ONERAHI CHAOS FORMATIONS IN NORTHLAND

Bhajan Singh

INTRODUCTION

This paper describes a creep type instability which occurs widely in Northland's allocthonous geological formations more commonly known as Onerahi Chaos-Breccia. The broad implications of this type instability on roading in the region is discussed and specific surface and subsurface features are described. Methods of identification and analysis, as well as remedial measures are examined. A successful method of stabilisation using horizontal drains bored from a deep pit is described.

SLIP FEATURES AND GENERAL TOPOGRAPHY

A large number of instabilities in Northland occur as slow intermittent creep movements which move at varying frequencies and intensities. Intensity of movement may increase in the wet months but movements are not always attributable to particular hydrological events. Their presence is usually betrayed by depressions in the road surface and high road roughness.

The topography in the area is usually gently sloping and very hummocky, with instabilities occurring at slope angles as low as 6° but more often in the range of 10 to 15°.

GEOLOGY

A large part of Northland consists of allocthonous units which have been structurally displaced to their present locations and in the process have become highly sheared and fractured. The allocthon emplacement is believed to have occurred during the Oligocene-Miocene period and the allocthonous rocks consist mainly of older Cretaceous sandstones and mudstones unconformably overlying younger rocks (Hayward, Brook and Issac, 1989).

The term Onerahi Chaos-Breccia is currently used quite commonly to describe in general the Mangakahia and Motautu complex sandstones, mudstones and limestones forming the Northland Allocthon. The term was initially coined and associated with chaotic sedimentary soils occurring in the Onerahi Peninsula in Onerahi, Whangarei (Kear and Waterhouse, 1967, 1977) and are distinct from the predominantly igneous allocthonous Tangihua Complex rocks.

Figure 1 shows the distribution of allocthonous rocks in Northland (after Hayward, Brook and Issac, 1989). The unshaded areas on the west coast mainly represent upper tertiary and quartenary sediments whereas the unshaded areas on the east coast indicate where the allocthon meets the Permian-Jurassic Metagreywacke basement. The Northland State Highway network is superimposed on this map and areas of high instability, compiled from an inventory on instabilities on the State Highway network, have been highlighted.

ECONOMIC IMPLICATIONS

Catastrophic failures seldom occur in these undisturbed formations, although in some areas where embankments are constructed on these instabilities or cuts made through them, some more spectacular failures may occur. Depressions on the road surface need to be filled up and graded frequently, increasing maintenance costs, or if left untended other road user costs are accrued from roughness, travel time increase and accident hazard. At many sites in Northland with low traffic volumes, it has been necessary from economic considerations to allow affected stretches of road revert to metal at the inconvenience of the road-user, rather than pay the high cost of maintaining the seal. As an example, maintenance costs for a two kilometre

IDENTIFICATION OF INSTABILITIES

The very gentle and sometimes thickly vegetated slopes on which some of these instabilities occur, their intermittent and somewhat erratic behaviour and the absence of readily discernable head scarps or toes occasionally lead to difficulties in appreciating the scale of the problem. Subsurface investigations are usually necessary but careful observation of surface features can often offer some useful insight into the instability. Ariel strip maps and oblique photographs are useful in revealing the structure of the soil beneath the mantle. In open pasture seeps and springs, clusters of reeds or rushes, leaning fencelines and irregular hummocks are useful indicators of instable areas. In bushed country leaning trees and scouring streams with collapsing sides are a good indicator.

SUBSURFACE INVESTIGATIONS

Various tests may be carried out to determine subsurface conditions at the instability. One successful approach that has been used in the past is to use a good spread of penetrometer soundings which will give an indication of the extent of the instability and to supplement this with visual inspections of the substrata using bore holes or test pits. Apart from allowing visual appreciation of the strata where the movements are occurring, the test pit also reveals special features such as zones of permeability, fissures, slickensides and permit in-situ testing or retrieval of samples for testing in the laboratory.

Subsurface conditions vary greatly from site to site. There is usually a layer of completely weathered and very soft residual clays/silts ranging from 5 to 8 m in thickness overlying a highly to moderately weathered soft parent rock. The upper layer has a Dutch Cone Penetrometer resistance of 0 to 1 MPa, gradually increasing to 5 MPa before suddenly becoming hard. Test pits and bore holes reveal a gradual transition between the soft upper clays and the underlying less weathered material. The material at this transition may show some of the structure of its parent rock but is generally very disturbed. There are numerous slick surfaces occurring within this material, some appearing on a continuous plane for some length and others on a localised scale. These slickensides are often carry free water.

Groundwater levels within the slip area are generally very high and often at ground level. Small seasonal fluctuations in level may occur. Permeability tests carried out in the upper clays give results in the order of 10^{-8} to 10^{-10} cm/sec whereas the underlying fractured bedrock have more permeable values in the range of 10^{-5} to 10^{-10} cm/sec.

Residual shear strength angles measured from direct shear strength tests have been found to be in the range of 19 to 23°. Tests carried out by Wesley (1992) on "Onerahi Chaos" soils from Whangarei using ring shear apparatus give a lower range of 13 to 16°.

STABILITY ANALYSIS

Due to the difficulty in identifying the exact boundaries of the movements and the long length of the instabilities, the most convenient approach has been to model the instabilities as infinite slopes. Back analysis of some of these slopes have revealed residual friction angles in the order of 20°, in agreement with test results but not explaining why the instabilities occur at slope angles as low as 6 to 15° unless unexpectedly high piezometric levels are postulated.

This leads to the general theory that the principal mechanism promoting instability in this type of formations is high ground water pressures. In the gully head locations where many of these instabilities occur, ground water flow through the more permeable, sheared and fractured "bedrock" emerge through the less permeable upper soils as springs and seepages.

Under high flow conditions these seepages and springs are unable to cope with increased flow, creating artesian conditions and lowering the stabilising forces below the driving forces and hence driving the instability in an intermittent or jerky movement. This causes a change in equilibrium and permeability regime of the material, allowing the instability to remain stable until the next build up of pressure. This in part helps to explain why no definite correlation exists between hydrological events and mobilisation.

REMEDIAL STABILISATION WORK

The very gentle slope angles at which many of these instabilities immediately dismisses remedial measures such as altering the geometry and retention as ineffective. Surface drainage measures do not bring about much benefit as the problem is more to do with water pressures within the more fractured medium at depth. Consequently deep drainage is seen as the most effective stabilisation measure. Sub-horizontal bores from the surface would need to very long in order to be sufficiently deep to be of use due to the low slope angles. This led to the development of a deep pit drainage system in Northland in 1987 by the former Ministry of Works and Development which consisted of a drilling chamber sunk to some depth from which horizontal drains could be drilled into the permeable bedrock to alleviate water pressures believed to be the principal cause of the instability.

The system currently in use consists of E-shaped precast concrete units which are assembled into a circular chamber on site and sunk to the required depth a short distance downhill of the unstable road formation. The diameter of the chamber is sufficient to accommodate a small drilling rig and drilling can proceed safely from within the chamber. Photo 1 shows such an assembled chamber and figure 2 shows the general steps involved in the deep pit drainage operation.

The major advantage of this system is that a good appreciation of the subsurface conditions are obtained from observation of the exposed sides of the chamber during sinking. In addition, the units are economical and light in weight and can be easily transported to and assembled on site. Permanent access to the chamber allows monitoring of the performance of the drains and maintenance if necessary. Additional drains can be bored at a later stage if existing drains become ineffective or if monitoring of the situation dictates the need. The drainage chamber also performs as a well and permanently lowers the water table in the area and aids in stabilising a block within the unstable formation.

Fifteen deep pit units have been installed in Northland to date and all have displayed positive signs of stabilising what used to be very active instabilities affecting stretches of highway. General subsidence and consolidation have been observed at several of the sites drained as would be expected with the lowering of water table and reduction of pore pressures.

Problems were experienced at two locations where pumped systems were used instead of gravity drainage but these were due to failure of the pumps rather than the overall concept.

A local precaster holds steel moulds for the concrete "E" units and several local Contractors have had experience in this type work causing recent tenders to be very competitive.

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stretch of State Highway in the Umawera Range south of Mangamuka totalled to $35,000 in 1990, compared to less than $10,000 that would be required for a comparable stretch with no instabilities.

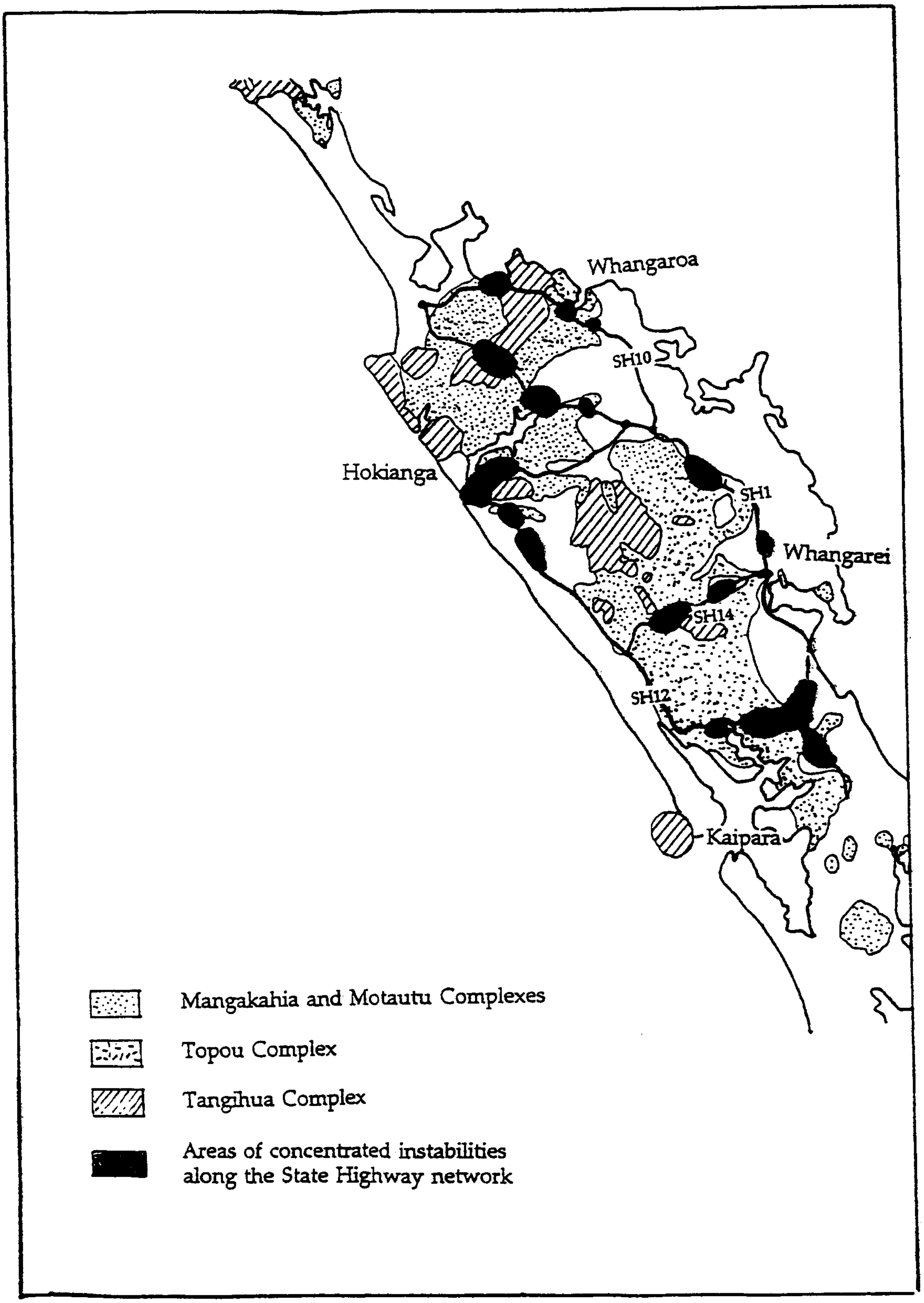

Figure 1 Outcrop Distribution of allocthonous rocks in Northland (after Hayward, Brook and Issac, 1989) and concentration of slope instabilities along the State highway network.

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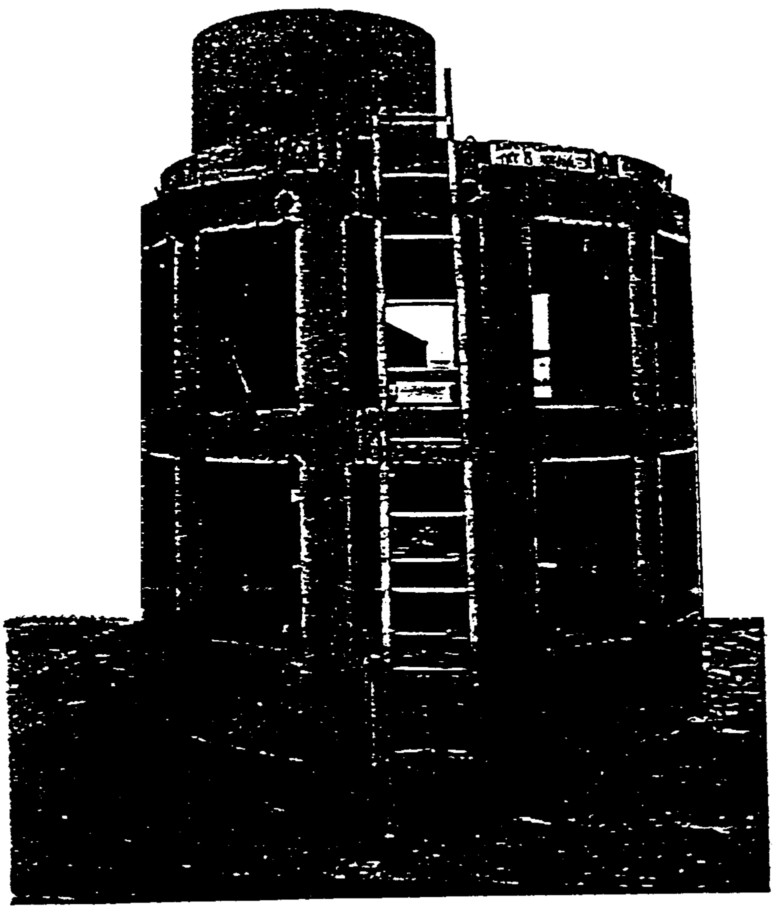

Photo 1 An assembled precast concrete drilling chamber. After being sunk into the ground this structure will be provided with lights and ventilation, forming a secure working space from where drilling will be conducted. The circular piece above the chamber is the first of several manhole rings which provide access to the ground surface.

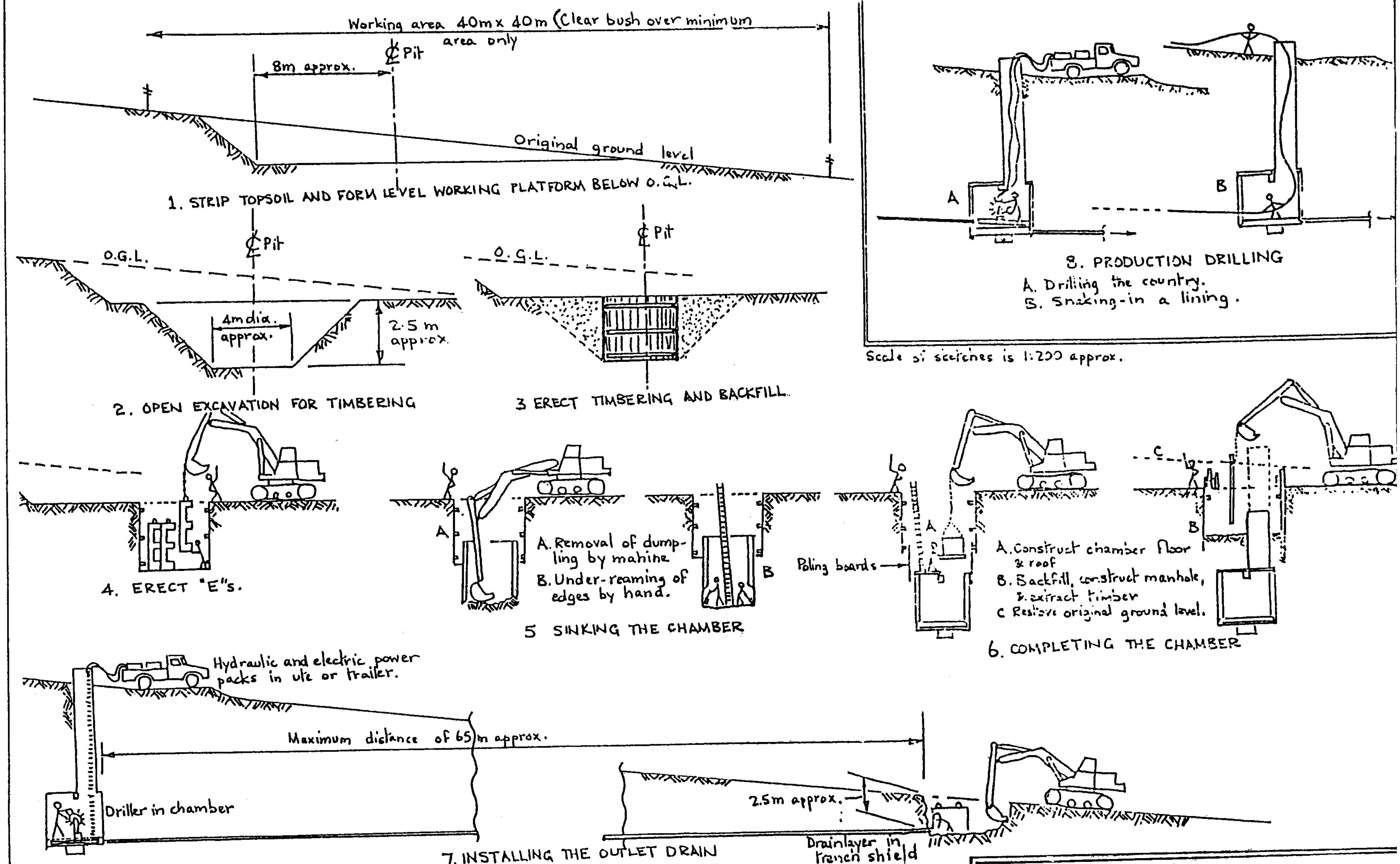

FIGURE 2 - STEPS INVOLVED IN DEEP PIT DRAINAGE OPERATION.

GEOTECHNICAL ASPECTS OF WASTE MANAGEMENT
WELLINGTON SYMPOSIUM
MAY 1994

The New Zealand Geomechanics Society is convening a two day symposium on the topic of "Geotechnical Aspects of Waste Management" to be held in Wellington on 13 and 14 May 1994.

PROGRAMME

The symposium's organising committee has had a good response from invited speakers allowing an extensive programme to be assembled.

During the two days of the Symposium, a full programme of over 35 selected speakers will be making presentations on a wide variety of topics, collected together in five sessions, all falling within the general area of **Geotechnical Aspects of Waste Management**. The following sessions, including time slots allocated for open discussion, have been arranged.

FRIDAY 13 MAY 1994

Session 1 (9.00 a.m. - 10.00 a.m.)

Welcome and Keynote Address

By Professor Laurie Richards of Lincoln University.

Session 2 (10.30 a.m. - 1.00 p.m.)

Planning and Legislative Requirements

Lawyers, planners and engineers working in this field will discuss the scope and implementation of the Resource Management Act and other legislation.

Session 3 (2.00 p.m. - 5.00 p.m.)

Waste Containment

Investigation design and construction of waste containment systems will be discussed.

Symposium Dinner (7.00 p.m. -)

SATURDAY 14 MAY 1994

Session 4 (8.30 a.m. - 12 noon)

Rehabilitation of Contaminated Sites

Investigation and treatment of contaminated ground.

Session 5 (1.00 p.m. - 4.30 p.m.)

Mining

Presentations will include those relating to talings dams, pit slope stability and overburden dumps.

4.30 p.m.

Symposium Close and Farewell

REGISTRATION

The registration fee will be $250 inclusive of the conference dinner teas and lunches. Day and part day registrations will also be available.

Bed and breakfast accommodation at a university hall of residence and a programme for accompanying persons will be offered.

If you are interested in attending the symposium please complete and return the following form.

Post To: The Organising Committee
NZ Geomechanics Symposium 94
PO Box 41054
Eastbourne

or fax to 04 562 8731

Yes! I am interested in the 1994 Wellington Symposium. Please send a registration brochure to:

Name: ______________________

Address: ______________________

____

APPLICATION FOR MEMBERSHIP
of
New Zealand Geomechanics Society

A TECHNICAL GROUP OF THE INSTITUTION OF PROFESSIONAL ENGINEERS OF NEW ZEALAND

The Secretary
The Institution of Professional Engineers of New Zealand
P O 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: ______________________ in full in block letters, surname last)

PERMANENT ADDRESS: ______________________

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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:

International Society for Soil Mechanics for Foundation Engineering	(ISSMFE)	Yes/No	($16.00)
International Society for Rock Mechanics	(ISRM)	Yes/No	($16.00)
International Association of Engineering Geology	(IAEG)	Yes/No	($10.00) (with bulletin) ($37.00)

SIGNATURE OF APPLICANT: ______________________

DATE: ______________

NB: Affiliation Fees are in addition to the basic Geomechanics Society membership fee of $36.00 which is reduced to $30.00 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.