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N.Z. GEOMECHANICS NEWS

No. 25

NOVEMBER 1982

A NEWSLETTER OF THE N.Z. GEOMECHANICS SOCIETY

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THIS IS A RESTRICTED PUBLICATION

"N.Z. Geomechanics News" is a newsletter issued to members of the N.Z. Geomechanics Society. It is designed to keep members in touch with recent developments and events in the field of geomechanics. 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 $15.00 and is supplemented according to which of the International Societies, namely Soil Mechanics ($6.00), Rock Mechanics ($8.50), or Engineering Geology ($4.50) the member wishes to be affiliated. Members of the Society are required to affiliate to at least one International Society.

EDITOR'S NOTES

1. Advertising

With the permission of Engineering Publications given to the N.Z. Geomechanics Society to sell advertising space, advertisements now appear in Geomechanics News for the first time since Number 10 (May 1975).

2. Articles in this Issue

This issue contains two articles, both of which have arisen from topical local group meetings in 1981. In the first Gavin Archer discusses aspects of ground anchor design, drawing on experiences in the Wellington area. In the second Colin Harvey, Peter Riley and Allan Pickens discuss various aspects of problems associated with shrinkage in Auckland clays.

3. Geomechanics Research at Universities

Summaries of geomechanics research over the past four years at the University of Auckland (Dept of Civil Engineering and Dept of Geology) and University of Canterbury (Dept of Geology) appear in this issue - and largely account for the extra bulk of the issue. A summary for the Dept of Civil Engineering at the University of Canterbury appeared in Geomechanics News No.24.

4. Errata in the Article on Analysis of Triaxial Test Results

The above article by P.J. Millar in Geomechanics News No.24 p.28 had several errors in the formulae given. The formulae should read.

1) p. 28, 2nd equation

$$\Sigma\ pi\ qi = d \sum_{i=1}^{n} pi + \text{Tan}\ \beta \sum_{i=1}^{n} pi^{2}$$

2) p. 30 $\bar{\sigma}$ = mean value of $\sigma_i = pi - qi \text{ Sin } \phi$

3) p. 33 $\text{Tan } \phi_{97.5 \text{ min}} =$

5. Membership Application

To assist Society members in recruiting new members, an application form can be found at the back of this issue. Please note that to facilitate the management committee's task of scrutinising the applications, prospective members are required to be nominated by existing financial members of the Society. Prospective members are requested not to send subscription fees with their applications.

6. Change of Address

Members are reminded that changes of address should be notified to the Institution Secretary, using the form provided in the back of this newsletter.

7. Contributions Wanted

Contributions to N.Z. Geomechanics News may be in the form of technical articles, notes of general interest, letters to the Editor, or book reviews, and may cover any subject within the fields of Soil Mechanics, Rock Mechanics and Engineering Geology. Articles on site investigations, construction techniques or design methods which have been successfully used in New Zealand, and which would be of help to other members, would be particularly welcome. All contributions should be sent to:

The Editor,
N.Z. Geomechanics News,
c/- N.Z. Geomechanics Society,
P.O. Box 12-241,
Wellington North.

S.A.L. Read
Editor.

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 $90.00 for the three volume set.
- Proceedings of the Hamilton Symposium "Tunnelling in New Zealand", November 1977. Price $18.00 to members, $20.00 to non-members.
- Proceedings of the Second Australia-New Zealand Conference on Geomechanics, Brisbane, July 1975. Price only $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.
- Copies of back issues of the IAEG Bulletin Nos.15 (June 1977) and 24 (December 1981) are available. Price $3.00 to members.
- Copies of all back-issues of "New Zealand Geomechanics News" are available to members at a nominal price of $2.00 per copy.

(b) From Government Bookshops:

- "Slope Stability in Urban Development" (DSIR Information Series No.122). Price $2.00.

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.

T.J. Kayes
Publications Officer

SOME ASPECTS OF GROUND ANCHOR DESIGN

G.C. Archer

1.0 INTRODUCTION

With an increased amount of building development in Wellington City both temporary and permanent ground anchors are being used on an increasing scale to retain small, moderate and high excavation faces. In a number of cases large buildings already exist above excavated faces which are permanently retained by ground anchors. The foundation integrity of these buildings is in most cases totally dependent on the suitability of design, construction and tensioning of ground anchors.

The design of ground anchors for large excavations does not appear to be generally understood and in some cases this design is being done by engineers and others who may not fully understand the sensitivity of ground anchor design. This article discusses various aspects to be considered during the design of ground anchors.

2.0 ANCHOR LOADS

Ground anchors have a optimum load capacity to ensure with some confidence that they will carry out their design function. For the Wellington greywacke conditions the lock-off tension loads should not exceed 70 tonnes for permanent ground anchors and 80 tonnes for temporary ground anchors. There are a number of cases where permanent ground anchors have been designed and tensioned up to 200 tonnes. It can be argued that by proportioning the bond length to accept the design tension load, the ground anchor has the capacity. Although it may well have the ultimate capacity, it has been determined overseas that highly loaded ground anchors tend to "walk" (i.e. undergo strain creep) after final tensioning and thereby become partially or totally de-tensioned. Secondary grouting is done principally to provide corrosion protection to the ground anchor free length (tendons, strands etc). Provided that this is done immediately on completion of tensioning this will lock the load into the system, however, strain creep will tend to unload the bonded length and load the grouted free length of the ground anchor. This transfer of load will redistribute the original tension force in the ground anchor, and hence, some outward movement could occur at the retaining wall face to provide equilibrium tension in each ground anchor.

3.0 MECHANISM OF LOAD TRANSFER

A number of ground anchor designs which have come to the attention of the author in recent years indicate that the designers in most cases do not fully understand the general mechanics of load transfer from the tendon to the ground. For any ground anchor to function to its design intent, tensioning must create a shortening of the anchor free length relative to the retaining wall face plate. A shortening of the anchor free length indicates that the top of the bond length has moved, albeit slightly, to transfer these tension forces down the bond length. This tension force is resisted by bond shear between the grout and country. Designers should not be trapped into using compression type ground anchors without detailed modifications, as this system will usually develop a reaction to anchor tensioning through the grouted anchor which acts as a strut. Compression type ground anchors are suitable where only the bond length is grouted, without forming a grout column which extends back to the face. This is an unusual anchor type for the Wellington conditions and is difficult to satisfactorily construct. All ground anchor designers should be reminded of the principle of standing in a bucket and then trying to lift it off the ground.

4.0 BOND LENGTHS

Proportioning of ground anchors with large bond lengths is in the opinion of the author a design fallacy. The optimum bond length for moderately loaded ground anchors (up to 80 tonne lock-off load) is about 5 to 6 metres. Despite what the calculations might indicate, the ground anchor bond beyond these lengths will in most cases not be utilised. To transfer the load down the bond length then the ground anchor must undergo elongation, which would be manifested at the retaining wall face as outwards movement. For long bond lengths the amount of ground anchor elongation required to provide equilibrium resistance to the motivated mass being retained will generally exceed the critical strain and thus landslip failure could occur before sufficient resistance force can be developed. The design of ground anchor bond lengths should consider the following aspects.

(a) development of permanent anchor bond and the amount of outwards movement which can be tolerated at the retaining wall face and within the supported mass.

(b) the consequential effect of losing the supporting capacity of one ground anchor on the adjacent ground anchors.

In some cases ground anchors have been proportioned with large bond lengths due to other building constraints. In most cases, however, long bond lengths result from distributing large forces to individual ground anchors because of economic considerations. It can be shown that the economics of using two moderately loaded ground anchors instead of a large over-proportioned ground anchor are similar albeit that the two anchors are slightly more expensive than the single alternative. Despite economic considerations, the technical advantages of optimising bond length design is, in the opinion of the author, the most important consideration.

5.0 PROPORTIONING OF ANCHOR FREE LENGTHS

Anchor free lengths should be based on the geometric pattern of potential shear planes which can develop in the ground which is supported by the ground anchors. All ground anchor free lengths should be a minimum of 3.0 metres and then, if required, lengthened to extend back from the face to at least the potential shear plane on which the maximum horizontal force is developed. Further propotioning of anchor free lengths will be necessary to adequately cope with other potential shear planes, albeit that these planes may generate lesser forces. There has been a tendency by some designers to specify a standard anchor free length irrespective of where the ground anchor is positioned in relation to the potential shear planes. Other designers have proportioned the anchor free length such that the potential failure plane bisects the bond length. This practice should be avoided. Irrespective of what allowable bond stress has been used for design, tensioning of ground anchors will fail the bond between grout and country at the top end of the bond length. This stress will decrease down the bond length and may be fully dissipated within the top half of the bond length, thus not providing any force across the shear plane for which the anchor has been proportioned. As a consequence the development of the shear plane will create tension in the ground anchor at this point and this will be manifested at the retaining wall face by outwards movement.

6.0 FACTOR OF SAFETY

The Code of Practice for General Structural Design and Design Loadings for Buildings NZS 4203:1976 requires that earth retaining structures shall be designed to a minimum load factor of 1.7. There is a variety of opinion as to what factor of safety should be used for large retaining walls; the Ministry of Works and Development suggest a factor of safety of at least 2.0 for normal loading. The choice of an appropriate factor of safety is the designer's responsibility, however this should not be less than 1.7 for permanently ground anchored retaining walls.

As described above, current local practice generally includes for the factor of safety in each ground anchor by proportioning its length accordingly. This procedure leads to over-sized ground anchors. A more appropriate method of providing for the required factor of safety for the retaining wall as a whole is as follows:

(a) proportion each ground anchor to provide a factor of safety of 1.3 against ground anchor failure. Include within the design a loading factor of 2.0 on the sizing of the steel components.

(b) increase the number of ground anchors by 30 percent over that number required to resist the calculated load imposed on the retaining wall by the supported ground mass.

For an overall factor of safety of 1.7 on the retention system this combination will achieve the design intention, (i.e. 1.3 x 1.3 = 1.69). For higher factors of safety on the retention system the author suggests that each ground anchor is proportioned for a factor of safety of 1.3 as outlined in (a) above and the number of ground anchors be increased accordingly. This procedure is now being used overseas and conforms to the general principles of ground anchor proportioning outlined above.

7.0 DESIGN

As outlined, the design of ground anchors for retaining wall support is more complex than just providing restraint to mobilised forces. It is therefore apparent that both structural and geotechnical engineers have distinct involvements in the design of ground anchor retaining walls. Most ground anchor retaining walls form part of large developments, and therefore, they come under the structural engineer's umbrella. The principal designers should be aware of the sensitivity of ground anchor design and involve a geotechnical engineer on the design team.

NEWS FROM THE MANAGEMENT SECRETARY

1. 1983 IPENZ Conference

The conference is to be held at the University of Waikato from 14-18 February, 1983. The technical sessions of the Geomechanics Society will again take place on Wednesday afternoon (16 February).

The scheduled presentations are:

First session - G.A. Salt "Static and Seismic Design Using Residual Strength of Soils".

Second session - Introduction of MWD Civil Division Geotechnical Design Aid "Site Investigations" (Subsurface). CDP 813/B 1982.

Please note that the second session is a late change to the programme and is not included in the IPENZ 83 Conference circular.

2. Annual General Meeting

The Annual General Meeting of the Society will be held during the 1983 IPENZ Conference at 5 pm on Wednesday, 16 February, following the conclusion of the presentation of Society papers.

3. IPENZ Awards

Applications for the following awards are invited by IPENZ.

1. Environmental Award
2. Fulton-Downer Award
3. Furkert Award

Further particulars can be obtained from the Management Secretary.

4. Nominations for 1983 Management Committee

Nominations for election to the Management Committee for 1983 were received on behalf of:

G.G. Grocott
T.J. Kayes
D.N. Jennings
P. Luxford
P.C. McGregor
P.J. Millar
B.R. Paterson
S.A.L. Read

Since the number nominated represents the requisite number to be elected by the Society members, no ballot is necessary. The nominations will be put to the Annual General Meeting of the Society in February 1983 for confirmation of election.

5. Geomorphologists Register

A register of New Zealand geomorphologists is presently being formed. Those interested in being included on such a list should write to:

Dr M.J. Crozier
Department of Geography
Victoria University of Wellington
Private Bag
Wellington.

6. UNESCO Task Force on Hazards

Over the past 18 months UNESCO has co-ordinated a world-wide study on natural hazards. In New Zealand the task force has been convened by Drs M.J. Crozier (Victoria University) and I.G. Speden (N.Z. Geological Survey), and the final volume is in the last stages of preparation. The Geomechanics Society contributed to the document on the following topics - expansive soils, subsidence, liquefaction, and organic-rich materials.

The volume includes contributions reviewing scientific aspects of 24 natural hazards, plus 5 contributions on economic and social topics (insurance (2), medicine, law, land use planning). Any person wishing to obtain further information about the volume (scheduled for publication early in 1983, price to be fixed), may write to the Management Secretary or contact either of the conveners.

7. Membership List

Only 10% of the Society members responded to the request to complete and return the membership list at the back of Geomechanics News No.24 (June 1982). The Society requires an up to date list of addresses and international society affiliations to function efficiently, and avoid members' omission from the International Societies lists of members. The membership form is again included in this issue, and it would be appreciated if members who have not already done so, could fill out the form and return it to the Management Secretary.

G.G. Grocott
Management Secretary.

FORTHCOMING CONFERENCES

01-11	February	1983	Solid Earth Sciences. 15th Pacific Sciences Congress. Dunedin, New Zealand.
14-18	February	1983	IPENZ Annual Conference. Hamilton, New Zealand.
	April	1983	7th Asian Pacific Regional Conference on Soil Mechanics and Foundation Engineering. Haifa, Israel.
10-15	April	1983	ISRM 5th International Congress on Rock Mechanics. Melbourne, Australia.
27-29	April	1983	Speciality Conference on Geotechnical Practice in Off Shore Engineering, ASCE. Austin, Texas, USA.
10-12	May	1983	South Pacific Regional Conference on Earthquake Engineering. Wellington, New Zealand.
16-20	May	1983	Underground Works - The Man - The Environment. Warsaw, Poland.
18-20	May	1983	Symposium on Soil and Rock Investigations by In Situ Testing. Paris, France.
12-16	June	1983	Rapid Excavation and Tunnelling Conference. Chicago, USA.
13-17	June	1983	ICASP-4. The International Conference on the Application of Statistics and Probability in Soil and Structural Engineering. Florence, Italy.
19-24	June	1983	Geotechnical Engineering in Resource Development. VII Pan American Conference on Soil Mechanics and Foundation Engineering. Vancouver, Canada.
15-19	August	1983	New Zealand Roading Symposium. Wellington, New Zealand.
5- 8	September	1983	International Symposium on Field Measurements in Geomechanics. Zurich, Switzerland.
12-15	September	1983	International Symposium on Engineering Geology and Underground Construction. Lisbon, Portugal.
4- 7	October	1983	26th Annual General Meeting of Association of Engineering Geologists. San Diego, USA.
5- 9	December	1983	International Conference on Groundwater and Man. Sydney, Australia.
14-18	May	1984	4th Australian and New Zealand Geomechanics Conference. Perth, Australia.

Further information on these conferences may be obtained by writing to the Management Secretary.

G.G. Grocott.

GEOMECHANICS RESEARCH AT THE UNVERSITY OF AUCKLAND

DEPARTMENT OF CIVIL ENGINEERING

P.W.Taylor, M.J.Pender, T.J.Larkin, G.H.Cato and R.G.Compton

1.0 INTRODUCTION

The last report on this topic appeared in the November, 1978 (No.17) edition of Geomechanics News. This article reviews the projects completed since then, and describes work currently in progress.

The undergraduate course in Civil Engineering, modified since 1980, now provides a more logical sequence of subjects related to geomechanics. Introductory Geology for Civil Engineers is now a 1st Professional year subject, followed (in 2nd Pro.) by Geomechanics I. All civil engineering students take Geotechnical Engineering in the first half of the 3rd Professional year, while Geomechanics II is an elective subject in the final half year.

Lecture courses are offered at graduate level also. Researchers studying for PhD or ME by thesis take three such courses, and 'ME by coursework' students, six from the wide range available. The geomechanics-related subjects are Earthquake Engineering, Geomechanics Seminar, Applied Geomechanics and Earth Structures, while Engineering Geology is offered by the Geology Department. Any of these courses can be taken by graduates wishing to further their knowledge in a particular area, as a form of continuing education.

2.0 STAFF CHANGES

Dr J.M.O. Hughes resigned from the Civil Engineering Department at the end of 1979, and is now a principal of Situ Technology Inc (STI) in Vancouver.

Dr T.J. Larkin took up his appointment as Lecturer in Civil Engineering in June 1980. He had completed his PhD at Auckland in 1976, held a post-doctoral fellowship at the Norwegian Geotechnical Institute, and worked with Dames and Moore in San Francisco before returning to join the staff. He was promoted to a Senior Lectureship this year.

Mr H.R. Green, Road Research Fellow, resigned at the end of 1979 and took up a position with Beca, Carter, Hollings & Ferner.

Mr R.G. Compton joined the Civil Engineering staff a Senior Lecturer in 1980, and is now active in research on road pavements, with Mr B.H. Cato.

3.0 EARTHQUAKE ENGINEERING

On the experimental side, aspects under study include the dynamic properties of soils, shear wave velocity measurements in the field, liquefacticn of sands and model tests of spread footings under earthquake-type loading. Theoretical work includes studies of earthquake risk, soil-structure interaction, the effects of earthquakes on earth dams, the modelling of soil response to cyclic loading and seismic hazard.

3.1 Dynamic Properties of Soils

The strain-controlled triaxial apparatus, developed by Baccus, is still in use. The stress-controlled triaxial apparatus has been completely re-built, using an electro-pneumatic system similar to that developed at Berkeley.

35mm diameter samples are mounted in a triaxial cell with electrical readout of load, deformation and pore pressure. Loading is controlled by a signal generator, and combinations of static plus sinusoidal (or saw-tooth) wave-form applied for a given number of cycles. This is being used in the study of increase in pore-water pressures in earthdams under earthquake loading mentioned below.

A third type of dynamic apparatus performs the free vibration torsion test. This laboratory test gives low strain shear moduli which can be compared with those obtained from in situ shear wave velocity measurements. Such a comparison was reported in:

"Comparison of down-hole and laboratory shear wave velocity" by T.J. Larkin and P.W. Taylor, Can Geotech Jour 16 (1) p152-162, 1979.

The free-vibration torsion testing equipment was improved by Terry Mead to give modulus and damping characteristics of soils under shear strain amplitudes from 1×10^{-6} to 0.5×10^{-2} using the PDP 12 computer for data acquisition and analysis. The use of this information to provide theoretical models of soil behaviour is described in:

"Dynamic Soil Models" by T.L. Mead (ME Thesis 1979, supervised by P.W. Taylor). Sch Eng Rep 194, March 1979.*

3.2 Shear Wave Velocities

Further work in the comparison between laboratory measurements of shear modulus dependence on strain amplitude and in situ methods is under way in a PhD study supported by the Structures Committee of the Road Research Unit. Mark Plested has designed an instrument for the in situ generation of horizontal shear waves (with amplitudes up to and beyond 10^{-4}). A number of velocity geophones embedded in the surrounding soil will provide information about the wave velocity and attenuation of strain amplitude with distance from the source. The project title is:

"High Strain In Situ Shear Wave Velocities" (Current PhD study by Mark Plested, supervised by T.J. Larkin).

3.3 Liquefaction of Sands

In an effort to describe liquefaction phenomena in fundamental, rather than empirical, terms, Robert Alexander is completing his studies. A rigid loading frame was constructed to investigate the recoverable deformation characteristics of sand during cyclic compression tests. Low amplitude cyclic shear tests, some under constant vertical load, and others with zero vertical compression were also carried out, using a pneumatically loaded shear apparatus. The thesis which is now nearing completion is titled:

"A fundamental Study of Sand Liquefaction under Cyclic Loading" (Current PhD study of R.C.K. Alexander, supervised by P.W. Taylor).

3.4 Earthquake Effects on Footings

Using a model spread footing (500 mm x 250 mm) founded on clay, Peter Bartlett had carried out tests to determine moment/rotation relationships

* When the thesis or project is also published as a School of Engineering, University of Auckland Report, Sch Eng Rep is used to refer to the report.

and vertical deformations under cyclic rotational displacements ('rocking' tests). The results showed reasonable agreement with the analysis developed. This work showed that, even when rotational yielding occurred, the footing was capable of sustaining vertical load with very little settlement.

Similar work, but for spread footings on sand, was carried out by Paul Wiessing. For surface footings, it was found that settlement, during rocking, tended to be greater than for clays at the same load factor. This is reported in:

> "Foundation Rocking on Sand" by P.R. Wiessing (ME Thesis, 1979, supervised by P.W. Taylor). Sch Eng Rep 203, August, 1979.

A brief summary of the research of both Bartlett and Wiessing was presented at the Tenth International Conference on Soil Mechanics and Foundation Engineering in Stockholm in June 1981:

> "Foundation Rocking under Earthquake Loading" by P.W. Taylor, P.E. Bartlett and P.R. Wiessing, Proc 10th Int Conf ISSMFE, v.3, p313-322.

The concept of foundation yielding, as an alternative to column yielding, in capacity design of earthquake resistant structures was included in the following paper, which received the Freyssinet Award:

> "Foundations for Capacity Designed Structures" by P.W. Taylor and R.L. Williams, presented at the South Pacific Regional Conference on Earthquake Engineering, May 1979. Also Bull NZ Nat Soc Earthquake Eng v.12 (2), p.101-113, June, 1979. Sch Eng Rep 193, January, 1979.

It is intended to extend this work to cover embedded footings on sands, which are expected to have superior characteristics under earthquake loading.

3.5 Seismic Hazard

An assessment of seismic hazard in New Zealand was carried out by Trevor Matuschka in his PhD studies. Two approaches were used. Firstly an empirical statistical approach based solely on earthquake records of the past 136 years. The second is based on seismotectonic data,in which the country is divided into 17 areas; the seismic activity of each being assessed separately. Attenuation characteristics for both peak acceleration and MM intensity are investigated and seismic hazard maps for New Zealand are presented.

> "Assessment of Seismic Hazard in New Zealand" by T. Matuschka (PhD Thesis 1980). Sch Eng Rep 222, January, 1980.

Trevor Matuschka is now with Earth Technology Corp (Ertec) in Long Beach, California.

3.6 Soil-Structure Interaction

One aspect of this topic, with immediate practical implications, is the assessment of earth pressures during earthquakes, on the basement walls of buildings. An ME study by Zacheus Indrawan addresses this problem and provides a simple, practical method of design. The method correlates well with published site measurements made on a building in Japan during an earthquake:

"The Seismic Earth Pressure on Basement Walls exerted by Cohesive Soil" (ME Project Report, 1981, supervised by P.W. Taylor). Sch Eng Rep 265, January, 1981.

The design method proposed in the report was presented, in a briefer form, at a conference in St Louis:

"A Simple Method of Estimating Seismic Pressures from Cohesive Soils against Basement Walls" by P.W. Taylor and Z. Indrawan. Proc Int Conf on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, v.I, p 241-246, St Louis, May, 1981.

3.7 Seismic Stability of Earth Dams

This is, of course, of importance in New Zealand. The current study includes the development of a two-dimensional finite difference program for earthquake response of earth dams. Using the material properties observed in dynamic tests (as described above) estimates of deformations caused by a given earthquake will be made. An equally important aspect of the problem is the build-up of pore pressures during the earthquake, their subsequent redistribution and dissipation, and the effects on post-earthquake stability. Associated with these theoretical developments, testing of materials from a large earth dam now under construction, are being made:

"The Earthquake Stability of Earth Structures" (current PhD study by Marianne O'Halloran, supervised by P.W. Taylor and T.J. Larkin).

4.0 STATIC STRESS-STRAIN BEHAVIOUR AND SOIL-STRUCTURE INTERACTION

4.1 Mathematical Modelling of Soil Stress-Strain Behaviour

One area of intensive development in the international geotechnical engineering scene is the mathematical modelling of soil stress-strain behaviour. The modelling of the behaviour of over-consolidated soil by means of a critical state model using a stress-strain relationship for a work hardening plastic material is a major interest of Dr M.J. Pender. Two research projects have extended this work. Firstly an ME thesis project combined the soil model with the stress path method to give a 'design method' for predicting vertical settlements, both drained and undrained, and horizontal displacements as well as pore pressure changes in a soil profile. An elastic stress distribution used with the soil model provides a means of performing the calculations with much less computer time than a finite element analysis.

Despite the assumptions made it was found, by comparing the calculated results with measured performance, that the method provides a good approximation to foundation deformations. It is superior to conventional settlement procedures and yet the analysis is much less costly than a finite element analysis. The work is reported in:

"Stress Path Settlement Prediction using a Soil Model" by A.R. Hira (ME Thesis 1980, supervised by M.J. Pender). Sch Eng Rep 233, 1980.

In parallel with Hira's work a PhD study resulted in the development of a finite element program which is capable of performing an incremental non-linear analysis of foundation loading problems. This program provides a tool for further investigation of the stress path method as well as a rigorous analysis based on the soil model. This work is reported in:

"Non-linear Soil Modelling" by C.J. Graham (PhD Thesis, 1982, supervised by M.J. Pender). Sch Eng Rep 294, 1982.

As this soil model handles cyclic loading it is also relevant to earthquake response of soils. The following publications cover some aspects of this work:

"Cyclic Mobility - A Critical State Soil Model" by M.J. Pender. Proc Int Symp on Soils under Cyclic and Transient Loading, Swansea, Vol. 1, pp 325-333, 1980.

"The Response of a Soil Oscillator to Dynamic Excitation" by M.J. Pender. Proc Symp on Implementation of Computer Procedures and Stress-Strain Laws in Geotechnical Engineering, Chicago, Vol II, pp 659-702, 1981.

"A Model for the Cyclic Loading of Overconsolidated Soil" by M.J. Pender, in "Soil Mechanics - Transient and Cyclic Loads", Chap. 11, pp 283-311, Wiley, 1982.

4.2 Properties of Natural Soils

One criticism of the critical state framework which the mathematical model discussed above utilises is the fact that much of the evidence in its favour has been obtained from laboratory prepared soils. A current project by Zacheus Indrawan involves an extensive series of triaxial tests on natural soil. It is intended to use this data to investigate the validity of the critical state idealisation for one New Zealand soil. We also have funding from the National Water and Soil Conservation Organisation to build a multi-axial apparatus so that these paths other than those obtainable in the conventional triaxial apparatus can be investigated.

"The Stress Strain Strength Behaviour of Auckland Soils". (Current PhD Study by Z. Indrawan, supervised by M.J. Pender).

4.3 Soil Structure Interaction

A further aspect of soil structure interaction has been studied in two PhD projects which studied by means of small scale laboratory tests the lateral load behaviour of poles embedded in sand. Both projects were supported by the Structures Committee of the Road Research Unit.

In the first of these projects the displacement of the sand when a pile was driven into a bed of sand was measured. The work is presented in:

"Aspects of Soil-pile Interaction Under Static Loads" by P.R. Goldsmith (PhD Thesis 1979, supervised by J.M.O. Hughes). Sch Eng Rep 43, 1979.

Peter Goldsmith is now with Fraser Thomas Partners in South Auckland.

The second of these projects measured the lateral load behaviour of single driven piles and two pile groups in the sand. It was noted that the zone of sand disturbed during the driving was the zone affected during the lateral loading. This suggests that the behaviour of driven piles is independent of the in situ properties and stresses in the sand before driving. It was also found that for the two pile groups the spacing had to be greater than 14 diameters for there to be no interaction. The work is reported in:

"Laterally Loaded Pile Groups" by H.D.W. Fendall (PhD Thesis 1980, supervised by J.M.O. Hughes). Sch Eng Report 251, 1980.

Hugh Fendall is now working with the Auckland District Office of the MWD.

In addition to the two PhD theses some work on applying the concepts to the behaviour of a real pile load test is reported in:

"Full Scale Laterally Loaded Pile Test of the Westgate Freeway Site, Melbourne, Australia. Load Deflection Predictions and Field Results by J.M.O. Hughes, P.R. Goldsmith and H.D.W. Fendall, Sch Eng Rep 190, 1979.

5.0 FIELD STUDIES

5.1 In Situ Testing

Increased emphasis is currently being placed on in situ tests, considered to be preferable to laboratory tests by some geotechnical engineers. A study was carried out to compare values of undrained shear strength and lateral earth pressure coefficient Ko using the self-boring pressure metre (camkometer) shear vane, and laboratory tests on 'undisturbed' samples. The soils at the site (Hobson Bay) were soft, normally-consolidated clays. The MWD kindly co-operated by providing the camkometer and associated equipment. Values of undrained shear strength deduced from camkometer measurements were 55% greater than by vane.

"A Comparison between the Results of Self-Boring Pressuremeter Tests and Conventional Tests on a Deposit of Soft Clay at Hobson Bay" by Siow-Juay Mok (ME Project Report, supervised by P.W. Taylor and M.J. Pender, 1981). Sch Eng Rep 266, February 1981.

The work was supported by the University of Auckland Research Committee, to enable the camkometer to be hired.

A larger, and more robust, self-boring pressuremeter was constructed as part of an ME project funded by the Structures Committee of the Road Research Unit. Because of the large size of the device the membrane is heavier and more resistant to rupture from shell fragments etc, the drilling module can handle occasional gravel particles up to about 20 mm. It was evaluated in the field by conducting tests near the Tristram Ave interchange on the Auckland Northern Motorway. The work is described in:

"The Development of a Self-boring Pressuremeter" by A.W. Welham (ME Thesis, 1981, supervised by M.J. Pender and T.J. Larkin). Sch Eng Rep 264, August, 1981.

5.2 Earthquake Effects on Water Bores

This study was made at the suggestion of Mr W. Haigh of the Hawkes Bay Catchment Board, who also enlisted the financial support of five local industrial firms. In the Hawkes Bay area, much of the water supply is obtained from underground sources. It is therefore of value to study the effects of a major earthquake on this source of supply.

The study entailed boring and sampling of soils on the site, downhole shear wave velocity measurements, laboratory testing and computer analyses to produce a simulated earthquake record and then to assess ground deformations during the earthquake. The results indicate that curvatures sufficient to produce yield in casing can occur in a major earthquake. This is in accord with evidence that large curvatures and in some cases, fracture, was believed to have occurred during the Napier earthquake, 1931. The work was supported by a Study Award Grant from MWD.

"An Assessment of the Seismic Behaviour of Borehole Casing" by N.G. Watson (ME Project Report, 1980 supervised by P.W. Taylor). Sch Eng Rep 219, March 1980.

5.3 Non-Circular Slip Analysis

Traditionally most analyses of slope stability are based on the assumption of a circular failure surface. In fact, many slope failures have a non-circular shape. This study applied the methods of Janbu (the 'general procedure of slices') to a slope at Te Atatu which had failed. The problems of identifying, sampling and testing the soils along the critical surface are found to be crucial in applying this type of analysis.

"Non-Circular Slope Stability Analysis" by N.J. Augustine (ME Project Report 1981, supervised by P.W. Taylor and T.J. Larkin). Sch Eng Rep 244, January, 1981.

5.4 Railway Stabilisation through Peatlands

This project was to investigate the stability of the Main Trunk railway across the Rukuhia swamp. It was funded by N.Z. Railways who also gave considerable assistance in the field work. The area was investigated using a field vane and electric cone penetrometer and many samples recovered. At Rukuhia station, an automatic recording station was established to monitor deformation of the line (about 12 mm) and subsoil pore pressures during the passing of each train. A comprehensive laboratory testing programme defined the peat properties. The report concludes with recommendations for improving the stability of the track.

"Railway Stabilisation through Peatlands" by D.A. Kettle (ME Thesis Report 1980, supervised by P.W. Taylor). Sch Eng Rep 242, Dec 1980.

David Kettle later joined the firm of Golder, Hoek and Associates in Canada.

5.5 Stability of Railway Embankments

A number of embankments on the railway between Auckland and Whangarei were studied. One objective was to assess the NZR practice of injecting cement grout into unstable embankments. The project included observation of surface movements and pore water pressures at a site near Wellsford. It was found that soils in the unstable sections typically contained montmorillonite, and some were in Onerahi Chaos Breccia. Recommendations for improving stability included improvements to drainage methods and modifications of the grouting procedure.

"Stabilisation of Railway Embankments, North Auckland Line" by E.W. Smith (ME Project Report, 1981, supervised by P.W. Taylor). Sch Eng Rep 252, February, 1981.

6.0 ROCK MECHANICS

The work on rock slope behaviour summarised in the previous report (November 1978) was presented to the Bridge Design Seminar organised by the Structures Committee of the Road Research Unit:

"Rock Slope Design" by M.J. Pender, RRU Bulletin 44, p 73-83, 1979.

A current interest in rock slope stability relates to the behaviour of cut slopes in closely jointed rock and the effect of the non-linear failure envelope expected for such material.

Another aspect of a closely jointed rock mass is the phenomenon of dilatancy. An ME project was devoted to using finite element analysis to investigate the effect of dilatancy prior to failure. An interesting effect on the stress distribution was shown about a tunnel and a pile socket. This is presented in:

"Dilatancy and the Behaviour of Closely and Irregularly Jointed Rock Mass" by W.J. Grey (ME Thesis, 1981, supervised by M.J. Pender). Sch Eng Rep 271, March, 1982.

A new area of rock mechanics research is concerned with the properties of coal. The Energy Research and Development Committee is funding a PhD study. The deformations and stress changes which occur in the Huntly coal when a test excavation is formed will be observed:

"The In Situ Mechanical Properties of Huntly Coal" (current PhD project by K.W. Mills, supervised by M.J. Pender).

7.0 ROAD RESEARCH

The programme of roading research, funded by the National Roads Board, has continued. Since 1978 the following projects have been completed:

"The Shear Moduli of Basecourse Aggregates, and The Effects of Anisotropy" by V.K.T. Lee (ME Thesis 1979, supervised by M.J. Pender). Sch Eng Rep 206, February, 1979.

In this project the earlier works of D.V. Toan and R.J. Maurice were compared, and a modified method allowing for the different boundary conditions in the respective tests was introduced to account for apparent discrepancies.

"Comparative Studies of Observed Performance and Performance Evaluated by Analysis of Five BC/16C Overlay Test Strips on Quarry Road Drury". Report for the Pavements Committee, Road Research Unit by H.R. Green then Road Research Fellow. Sch Eng Rep 234, January, 1981.

A multi-layer computer program (ELSYM 5) was used in conjunction with laboratory tests on pavement materials to predict the deformations and stresses of the Drury test strips. These were compared with field measurements.

"Aggregate Degradation, Particle Size, Grading and Permeability". Report for the Pavements Committee, Road Research Unit by F.G. Bartley and S.J. Woodward, supervised by R.G. Compton. Sch Eng Rep 267, August, 1981.

This laboratory study investigated the effect of degradation during compaction of various gradings of basalt aggregate on porosity and permeability.

Current pavement research projects are as follows:

"Performance of Variously Graded Basecourses Subjected to Known Traffic Loadings" (Project Director - B.H. Cato).

This is a field study of the deflection history of six test strips at Drury Quarry comprising three different gradings of both basalt and greywacke 40 mm basecourse. The test strips were constructed in 1970 as a project to determine the relative compactibility of a variety of basecourse gradings and stone types and recoverable deformation has been recorded at half-yearly intervals since. In addition the permanent deformation changes have been measured using the profile recorder developed as part of this project. Sampling and testing of the aggregate to determine changes in physical properties under traffic are about to commence. An accurate assessment of traffic loading over the service life of the pavement is available from quarry tally records.

"Modulus and Fatigue Strength of Lime-Stabilised Soils" (H.R. Green and R.G. Compton).

Two clays, one of sedimentary and one of volcanic origin have been subjected to static and dynamic tests at various lime contents, densities, moisture contents and curing times to establish flexural- fatigue properties. Laboratory testing has been completed.

"In Situ Measurement of Basecourse Permeability" (F.G. Bartley and R.G. Compton).

This project has consisted of investigations into practical field procedures for measurement of permeability of unbound granular pavement layers.

"Characterisation Tests on Various Basecourse Aggregates" (R.G. Compton and B.H. Cato).

This is essentially a testing programme to determine the resilient modulus and Poisson's ratio under dynamic loading of a variety of pavement materials. A prime objective is to determine whether confidence can be placed in the use of basecourse materials which have been regarded as of only marginal quality. It is also intended to assess the extent to which the behaviour of marginal materials can be enhanced by lime modification.

GEOMECHANICS RESEARCH AT THE UNIVERSITY OF AUCKLAND

DEPARTMENT OF GEOLOGY

W.M. Prebble

1.0 INTRODUCTION

This article reviews the teaching and research activities in engineering geology in progress or completed since June 1977 (see Geomechanics News No.14). Since then a number of research contracts and theses in engineering geology have been completed, a range of field index testing equipment has been acquired and put into use and new courses in both the Geology and Civil Engineering departments have been implemented. The policy continues of complementing the teaching, research and facilities in soil mechanics and rock mechanics in the department of Civil Engineering. Joint research direction and joint teaching of coursework is common practice. Hence the teaching and research in the Geology Department is more qualitative and geologically oriented and the equipment is either the broad-base full analytical range or is portable, field-oriented equipment designed for index testing of in situ materials and field samples.

Although the main dependence on teaching and research direction falls upon the writer, a deliberate involvement of other staff, with special expertise of fundamental importance to engineering geology, has been encouraged and is continuing to expand. This is in addition to the research which the expertise and analytical facilities in petrology and mineralogy have for several years attracted, as N.R.B. funded research and, more lately research into coal and weak rock. Also, applied research in marine geology and surficial deposits has contributed to the Maui Development projects as part of a wider University of Auckland research team.

2.0 UNDERGRADUATE COURSES

An expansion of the Geology Department's teaching in engineering geology to geology students occurred in 1977 at Stage 2 and Stage 3 levels. Some aspects of engineering geology and geologic hazards are introduced at Stage 1 and built upon as a major part of a whole year course at Stage 2. This course (21.201) includes structural geology, mechanical principles and elements common to both structural and engineering geology, geologic mapping and aspects of engineering geology (an introduction to the engineering geologic properties of soils, surficial deposits, weak rocks, strong rock and rock masses). Students doing this course complete an 8 day field exercise in basic geologic mapping and field techniques and shorter duration local field projects in engineering geologic logging, field index testing and simple laboratory index tests.

The Stage 3 coursework is combined with applied geophysics, groundwater, photogrammetry, photogeology, advanced structural geology and advanced field mapping, all in an Applied Geology Course (21.301). The engineering geology content consists of examples of principles and practice complemented with exercises in core logging, mapping, costing and hazard assessment.

3.0 GRADUATE COURSES

A graduate course in Advanced Engineering Geology is taken by B.Sc. (Hons) and M.Sc students who combine field projects with an extensive literature review.

3.1 Engineering School Courses

In the Civil Engineering Department an introductory course in geology for engineers is taught to 1st professional year students and an elective course in Engineering Geology is available to the 3rd professional year. The latter course consists of geological principles applied to engineering problems and is essentially a practical course designed around fieldwork, laboratory exercises and case history studies. Emphasis is placed upon the role of engineering geological expertise, the need to use it, the requirement for adequate planning and funding and the value of an engineering geolcgist in the geomechanics team. It is not intended as a course in geological proficiency for engineers but it does aim to considerably develop their recognition and understanding of geologic features of significance to ground engineering projects.

3.2 Combined Courses

A growing number of science (geology) students, who intend to specialise in engineering geology are advised to, and do, take the Geomechanics courses in Civil Engineering as an integral part of their B.Sc. This enables them to progress to Geomechanics (Graduate level) courses as part of their M.Sc. degree.

For this purpose. Geomechanics I, a second professional year paper,is available to science students as a full science faculty paper and need not form part of their "outside faculty" credit allocation. Some students in geology also take Geotechnical Engineering and Geomechanics II, and a small number also take the graduate course in Applied Geomechanics towards their M.Sc.

No separate degree structure for engineering geology exists or is envisaged. A full undergraduate and graduate training in the subject is available by combining the appropriate courses from Geology and Civil Engineering. In this way, a student completes 3 full one year courses at undergraduate level and 3 full one year courses at graduate level together with a thesis. Those who complete all these courses usually include a large component of theoretical and applied geophysics as well and this interest is often reflected in the thesis topic. The extent of applied geophysics research in the Geology Department is an area of interest to hydrogeologists, soil conservators and catchment authority engineers as well as to ground engineers and engineering geologists. This area of interest could be pursued with the geophysics staff directly concerned, Dr John Cassidy and Professor Manfred Hochstein.

4.0 RESEARCH PROJECTS

Over the last five years, graduate and staff research has concentrated on field oriented studies in predominantly weak rock terrain, volcanic terrain and surficial deposits.

Individual research by staff has also dealt with problems of aggregates, small-scale hydroelectric development, active faulting and zones of weakness adjacent to major construction sites, volcanic hazard assessment, slope stability at viaduct sites in weak mudrocks and volcanic deposits, active landslides and faulting affecting planning decisions, foundation and slope stability in Pleistocene volcanigenic soils and weak rock and the engineering geologic properties of "ignimbrite" units (including weak rock and soil categories). Much of this individual staff research is a result of research contract work, as for example the mineralogical analysis of aggregate degradation conducted by Professors Terry Sameshima and Philippa Black. Some is also an applied geology component of a University of Auckland team research project such as the coastal and marine studies carried out for the

Maui Development Project by Dr Murray Gregory. Many other requests for research are directed through the Applied Research Office of the University and some of these are carried out jointly with staff of the Civil Engineering Department, usually Professor Peter Taylor and Dr Mick Pender.

Theses which have been completed in recent years and are held in the University Library include:

"Geology and Landslides of the Eastern Te Aute District, Southern Hawkes Bay" Dr Jarg Pettinga, PhD. Thesis, 1980.

"Aspects of the Engineering Geology, including mass movement, of the Albany Basin and Paremoremo Area". Mr Dennis McManus, M.Sc. Thesis, 1981.

"Aspects of the Engineering Geology of the Rangitopuni Catchment, Auckland" Mr David Whyte, M.Sc. Thesis, 1982.

Other theses contain a section or chapter discussing features of engineering geological interest, such as Chapter seven on "Erosion and erosion control" in Jill Kenny's M.Sc. Thesis (1980) on the "Geology of the Ihungia Catchment, Raukumara Peninsula".

A number of research contracts from the National Roads Board have been carried out by Terry Sameshima and Philippa Black, and produced as formal University reports. An example is Pavement Research Project B.C.21 "Hydrothermal degradation of basecourse aggregate".

Four major research contracts on geology and mass movement have formed the topics for Ph.D and M.Sc theses. The research contracts, funded by the National Water and Soil Conservation Authority, were:

1. Slope stability in relation to soil types (colluvium) rock type and geologic structure in the Waimarama-Elsthore Valley - Tukituki River area, Southern Hawkes Bay (J.R. Pettinga).

2. Slope stability in relation to soil types (colluvium) rock types, clay mineralogy and geologic structure in the Rangitikei Valley. (R.C. Thompson).

3. Slope stability and hazards in relation to geologic structure, lithology and geotechnical parameters in the Albany Basin - Paremoremo area, Upper Waitemata Harbour Catchment (D.A. McManus).

4. Slope stability and hazards in relation to geologic structure, lithology and geotechnical parameters of the Rangitopuni Stream, Upper Waitemata Harbour Catchment (D.E. Whyte).

Current research topics include:

(a) Investigation of the structural, lithologic and geotechnical parameters in deformed mudrocks of Cretaceous and Tertiary age, in North-east Marlborough.

(b) Investigation of rockslide-debris avalanches in weak rock terrain, specifically at sites in South and West Nelson, northern Hawkes Bay and the Central Volcanic District.

(c) Petrology, mineralogy and fabric of weak mudrocks from a variety of Cenozoic rockmasses in the central and eastern North Island.

(d) Structure, lithology and geotechnical parameters of Miocene mudrocks and Pleistocene soils in Coastal Cliffs, Manukau Harbour, Auckland.

(e) Structure, lithology and geotechnical parameters of deformed Upper Cretaceous and Cenozoic mudrocks and limestones in North Auckland.

The above projects are Ph.D. and M.Sc. thesis topics being carried out by respectively Warwick Prebble, Frank Huppert, Peter Manning and Brian Shakes. All are field oriented, supported by field and laboratory geotechnical indexing and laboratory petrology and mineralogy.

An effort is being made to characterise the materials with index tests and to relate the field data to petrologic features such as gross mineralogy, clay mineralogy, fabric, texture, porosity, pore size, pore size distribution and shape. To this end the index tests include porosity, density, water content, point load index, penetration indices and shear strength indices. The petrologic analytical procedures include X-ray diffraction, X-ray fluorescence, scanning electron microscopy and thin section work. In the near future, analysis of pores, with an intrusion porosimeter, will also be attempted.

Research into slope stability in weak rock terrain has established a number of relationships of landslide occurrence and type with rock mass structure, lithology, tectonics and Quaternary history.

Current research continues in this direction but is expanding into a more rigorous examination of geotechnical properties in conjunction with fundamental lithologic properties such as rock fabric, texture, porosity parameters, skeletal arrangement and mineralogy.

5.0 EQUIPMENT AND FACILITIES

Engineering geology research in the Geology Department is utilising the extensive facilities and expertise for petrology, mineralogy, sedimentology and structure. Some have been referred to in the previous article, and include a sedimentology laboratory, a structural and engineering geology research laboratory, chemical laboratories, a range of microscopes, X-ray diffraction and fluorescence (automated), differential thermal analysis, electron microprobe and, in the Engineering School, a scanning electron microscope. A new item of equipment on order for the Geology Department is a Micromeritics Pore Sizer Porosimeter.

Simple indexing equipment, acquired under a policy of complementing the sophisticated geomechanical testing facilities in the Engineering School with geotechnical field indexing devices, includes the following: portable field shear vanes, penetrometers, rebound hammer, point load tester, cone indentor and a range of portable weighing and drying equipment suitable for field camp use. It is hoped to acquire a slake durability apparatus. A soils-weak rock shear box is used for structural and engineering geologic research and teaching.

GEOMECHANICS RESEARCH AT THE UNIVERSITY OF CANTERBURY

DEPARTMENT OF GEOLOGY

D.H. Bell and J.R. Pettinga

1.0 INTRODUCTION

This article reviews the research activities in Engineering Geology in progress or completed since 1979. Teaching activities remain similar to those outlined in Geomechanics News No.20 (May 1980).

2.0 RESEARCH

Current engineering geology thesis research projects being undertaken in the Department include:

(1) Aspects of Quaternary and Engineering Geology, Central Otago (Ph.D Study by D.H. Bell).

(2) Slope instability problems in relation to bedrock and surficial deposits Strathallan County, South Canterbury (M.Sc. study by M.R. Foley).

(3) Slope instability assessment, Moeraki Township, Waitaki County, North Otago (M.Sc. study by M. Molineaux).

(4) Roading investigations, Western Wanaka, Central Otago (M.Sc. study by P. Ackroyd).

Two further graduates who have recently completed work towards M.Sc. (Engineering Geology) papers are aiming to commence thesis research in early 1983.

Staff research includes continuing work by David Bell on aspects of chemical stabilisation of Port Hills Loess and other dispersive soils. Various field and laboratory projects relating to Tertiary soft rock slope instability are being undertaken by David Bell and Jarg Pettinga. Recent work has also concentrated on study of slaking and slabbing failure of soft rock materials.

Engineering Geology input into subdivision planning has evolved from our involvement through consulting research undertaken particularly in Christchurch and Queenstown.

The following M.Sc. theses in Engineering Geology have been completed since 1979:

Ormiston, A.W. 1981. The effects of water on Tertiary sedimentary soft rocks with special reference to the Patea dam site.

Smith, G. 1981. Regolith instability problems, Hundalee Hills, Southern Marlborough.

Dicker, M.G.I. 1981. The hydrology of the Waimea Plains alluvial basin, Nelson, New Zealand.

Scott, G.L. 1979. Engineering geology and urban planning: a study near Christchurch.

MINISTRY OF WORKS AND DEVELOPMENT GEOTECHNICAL DESIGN AIDS

In recent years the Ministry of Works and Development Civil Engineering Division has produced a number of publications to aid designers. These publications are available from the Government Printer (GP) or Head Office of Ministry of Works and Development (HO/MWD). Publications which may be of interest to Geomechanics Society members include:

CDP Number & Date	Title	Available from (address below)	Price
CDP 702/C July 1973	Retaining Wall Design Notes*	HO/MWD & GP	$5.75
June 1980	Supplement: Reinforced Earth: Notes on Design and Construction	REL	
CDP 706/A Aug 1978	Culvert Manual (two volumes)	HO/MWD & GP	$35.00
Jan 1981	Supplement to Ch 18. Design of Long Span Corrugated Steel Culverts	HO/MWD	$2.00
CDP 708/A Oct 1980	Code of Practice for Falsework Vol 1: Code and Appendices Vol 2: Commentary	HO/MWD & GP	$20.00
CDP 805/A Nov 1973	Subgrade Reaction Method for Pile and Cylinder Design	HO/MWD	$5.00
CDP 812/B 1981	Pile Foundation Design Notes	HO/MWD	$7.50
CDP 813/B 1982	Site Investigation (subsurface)	HO/MWD	$15.00
CDP 817/B 1982	Guide for the Economic Evaluation of Engineering Projects	HO/MWD	$10.50

* It is proposed to revise this document in the near future.

Addresses:

HO/MWD
Plan Records
Civil Division
Ministry of Works and Development
Head Office
P.O. Box 12-041
Wellington North.

REL
Reinforced Earth Ltd.
P.O. Box 12-061
Penrose
Auckland 6
Attention: Mr J.S. Knapp

D.N. Jennings.

NEWS FROM THE INTERNATIONAL TUNNELLING ASSOCIATION

Below is a precis of the General Assembly held during the 8th General Meeting held at Brighton from 6th-9th June 1982.

MEMBER NATIONS REPRESENTED

South Africa, Federal Republic of Germany, Austria, Belgium, Canada, People's Republic of China, Republic of Korea, Spain, United States of America, Finland, France, Iceland, Irak, Italy, Japan, Mexico, Norway, New Zealand, Netherlands, Poland, United Kingdom, Sweden, Switzerland, Czechoslovakia, Venezuela.

AFFILIATES - CHANGE IN THE STATUTES

The statutes of the association have been modified to admit affiliated members under appropriate conditions. The purpose of this change is to provide a broader base for the activities of the association. Information concerning this point and all the activities of the association can be obtained from the Secretariat.

ORGANISATION

The Executive Council of the Association, unchanged since 1980, is the following:

G. GIRNAU	Fed. Rep. Germany	President	until 1983
H.C. FISCHER	Sweden	Past President	until 1986
A.M. MUIR WOOD	United Kingdom	Honorary President	
J.K. LEMLEY	USA	Vice President	until 1983
L. LUPIAC	France	Vice President	until 1983
E. BROCH	Norway		until 1985
Y. ONOUCHI	Japan		until 1983
H.P.S. VAN LOHUIZEN	Netherlands		until 1983
F. DESCOEUDRES	Switzerland		until 1983
V. ROISIN	Belgium	General Secretary	

OPEN SESSION 'THE SUBSURFACE - CONTRIBUTIONS TO ENERGY SAVINGS'

The open session gathered about 200 participants and gave the opportunity to present recent studies on energy savings in subsurface large openings (M. DORUM - Norway), in underground urban transport systems (D. SUTTON - France), by design of mass transit railway (DR T.M. RIDLEY - United Kingdom), by insertion of tunnels in a highway network (F. DESCOEUDRES - Switzerland), by underground heat storage (S. BJURSTROM - Sweden) and to hear I.W. HANNAK (U.K.) about "DINORWIC and energy storage".

WORKING GROUPS

Each of the nine working groups previously created (see Geomechanics News No 22, p16) held working sessions during this conference.

The detailed summaries of the eighth annual meeting, including the main discussions of the General Assembly, the annual reports of the member nations, the reports on the activities of the working groups, the report of the general session "The Subsurface contributions to energy savings" will be published in "Advances in Tunnelling Technology and Subsurface Use" in Vol. 3 number 1 of the ITA Journal, by Pergamon Press Ltd.

G. Ramsay

LOCAL GROUP ACTIVITIES

1.0 AUCKLAND GROUP

1.1 Residential Slope Failures - Who Carries the Can?

A meeting on this provocative subject was held at the Auckland University on 16 September 1982. Legal implications of slope failures were emphasised by the cancellation of a group meeting scheduled for earlier in the year on the Grey Lynn landslip due to pending legal action. General aspects of the landslip, where several houses were destroyed, are given in Geomechanics News No.24.

The first speaker was John Clapperton of Jackson Clapperton Limited. John related his experiences in a court case in 1981 relating to a cliff slope failure in 1979 where 14 feet of lawn dropped off a property without damaging the buildings. The site had been investigated for the owner in 1975 on a limited budget. The case took five days to hear, with various consultants reporting on aspects of the original investigation and the failure. The decision of Justice Holland was based on the findings of a second consultant who claimed that the failure was of a circular type in the surface clays and that the original investigation should have drilled down to, and established the depth to unweathered material (which he found at 6 m).

This was disputed by Mr Clapperton who claimed that vertical limonite bands were evident in the cliff after failure and that failure was initiated by slabbing of the mudstone in the cliff face. This view was supported by the plaintiff's statement that during the rain storm in question a loud crash was heard, as if something heavy had fallen from the cliff. He looked out the window and there was no evidence of movement. Partial costs were awarded against Mr Clapperton.

John concluded with the comment that the house was still there and still occupied, but settlement was made because of loss of value of the property. Appeal against this decision was considered, but legal costs made further court action unjustifiable.

On the legal situation John made the following five points:

(1) The engineer should ensure that he understands the client's reasons for the report.

(2) If the report is for the Council, this should be stated.

(3) The financial limitations under which the report was done should be stated clearly.

(4) It should be remembered that an adversary situation always applies.

(5) The report must be phrased in layman's language with diagrams to support the reasoning.

Don Leadbeater of the Auckland City Council spoke next describing the Council's point of view and how they seek to control site stability. He opened his discussion with a remark that a property owner can make claims for up to six years after damage is first apparent, however, the initial construction/development, and possible source of fault can go back an unlimited number of years.

Council is first involved in assessing stability in approving land for subdivision which sets out to create a building lot. Council is duty bound by the Local Government Act to refuse to approve the scheme plan if the land is subject to erosion, slippage, or inundation arising from such erosion, subsidence or slippage. Much of the land left in inner city areas has problems of this nature. Many of these are existing sections and in an effort to allow the land to be used the owners frequently request a covenant on the title disclaiming the Council or the Earthquake and War Damage Commission from responsibility. Councils cannot sidestep their obligations in this regard. The Auckland City Council do however use a Memorandum of Encumbrance, which provides a warning on the title of any future problems. Legally this is a form of mortgage for a nominal charge which registers it in writing on the title. Such an encumbrance may limit the filling, excavation etc. on the title and enables a permanent recording of problems encountered.

Section 641 of the Local Government Act also requires that a Council shall refuse to issue a building permit if the land is subject to erosion, subsidence, slippage etc. The 1981 amendment Section 641A relaxed this to allow Council to issue a building permit for the erection of a building that is designed to be relocatable, if it is satisfied that the building can be relocated from that site. In some cases this will not be of much use, for example, in the Grey Lynn slip total failure occurred in twenty minutes from when residents first noticed that failure was occurring.

Mr Richard Worth, litigation partner of Butler White and Hanna, was the third speaker, then spoke from a legal viewpoint. Section 641 of the Local Government Act is also supplemented by Part 20 of the Local Government Act which deals with the powers of the Council in the context of subdivisional approvals. Regarding responsibility, the Limitation Act of 1950 gives a cut off point for responsibility of six years from the date of breach of contract; in the case of claims in tort however, the six year period runs from the date of damage. Damage can occur many years after the original negligent act. Causes of action can be in contract, for example, between an engineer and client, or in tort where duties can be owed to a wider group of people, for example, a Council to a subsequent building owner. Professional indemnity insurance can be taken out to give cover against such claims and where cases of negligence arise, judgement is given in terms of the state of the art applying at the time that the advice was given.

Where responsibilities for an error are shared, the solvency of co-defendants may mean that financial responsibility transfers to the stronger party, albeit that the negligence of that party is minor only, which in many cases will be a Council. The practice of including disclaimers of liability between a client and an engineer will not necessarily operate in the hands of a third party.

When cases come to court a recent trend is to exchange expert evidence before the case to assist determination of the issues.

Mr Don Taylor of Tonkin and Taylor in summing up said that it is very difficult to give a completely positive answer when the material being dealt with is as variable as natural soils. A soils engineer gives professional advice not a guarantee. He should not assume risks for the client, but evaluate these for the client to make his decision. Another question which arises is how much should be spent on a site investigation. In many cases an increase of one hundred times in site investigation expenditure would give only a 10% better answer.

P.B. Riley.

1.2 The Ruahihi Canal Repairs

The final meeting of the year was held on 3rd November 1982. It was a combined meeting with the Auckland Branch of IPENZ and was attended by approximately 140 people.

Professor Taylor who chaired the meeting, introduced the first of the three speakers John Galloway, with the comment that the committee report was highly recommended as it was even acceptable to the residents.

John Galloway, a member of MWD Committee convened by the Minister of Works and Development to report on the Ruahihi Canal failure, started his discussion with a description of the overall project, and how it basically changed from deep cuts at the canal intake to sidling cuts and buttress fills towards the forebay adjacent to the top of the penstock slope. He outlined the difficulties encountered with construction in the very recent volcanic ignimbrite material. The material was not considered as being untypical of New Zealand volcanic material but on the "bad" side of average. The wet weather encountered also hindered construction.

He then discussed the events leading up to the final failure such as sinkhole formation, fluctuating groundwater flows, sand boils, etc. and the puzzling settlement behaviour of the forebay. The failure mechanism for Fill A, as deduced by the Committee, was outlined in detail. The material of Fill A was moved bodily in large blocks with minimal toppling. The failure was sudden, energetic and orderly indicating artesian pressure build up overcoming the resistance of the soil and "floating" the fill away.

As a general comment he added that areas of recent volcanic material (volcanic rubbish tips) should command the greatest respect, that water retaining structures are the most difficult civil engineering structures to design and build and failures unleash the tremendous power of the retained water.

Bruce Houghton of the N.Z. Geological Survey at Rotorua then gave an erudite and enertaining talk on the geological setting of the general Tauranga and Bay of Plenty area. He drew parallels between the Omokoroa slides and the failure at Ruahihi commenting that the Bay of Plenty was "the slipperiest area in New Zealand". His explanation of the mechanism of ignimbrite flows and the effects on the engineering behaviour of the materials was very informative.

Peter Hay of Beca Carter Hollings and Ferner, the final speaker, acknowledged the permission of the Tauranga Joint Generation Committee, and then outlined the various options identified to reinstate the scheme and gave a detailed description of the recommended option. A description of the justification for and type of the canal lining system was given.

He then described the way in which the works had been subdivided into separate contracts and that work had been completed to date. Expenditures are currently 1.0 million dollars per month peaking to 2.4 million dollars per month next year.

Following a brief question and discussion time Mr P. Riley thanked the speakers and about 40 participants attended an enjoyable dinner at the Professional Club.

P.C. McGregor.

2.0 WELLINGTON GROUP

2.1 Te Marua Storage Ponds

The meeting held on 21 July 1982 was the second of a series of two meetings on the Wellington water supply arranged in conjunction with the local IPENZ Branch. In June at the first of the two meetings (a combined Water Group/ IPENZ meeting) the water supply system for the Wellington region was outlined together with the important role that the new storage ponds at Te Marua would play in the system.

Terry Kayes (Tonkin and Taylor) in his address to the second meeting discussed aspects of design and construction of the Te Marua storage ponds.

Terry described the geology of the area and explained how the two lakes are located on two terraces, one about 12 metres lower than the other. Aspects of the site investigations and testing to evaluate materials for design were discussed. The embankments were designed using the terrace gravels and seismic loading was carefully considered particularly as the Wellington fault trace passes the site. It is expected that large earthquake events would result in some permanent displacement of the embankments. An important feature of the design was the use of the limited deposits of loess on the site for the impermeable linings of the storage ponds. Performance of the loess lining was evaluated with a test reservoir constructed on the site prior to final design.

Protection of the storage ponds from contamination from hill water run off and groundwater movement has been provided through diversion channels and an underdrainage system respectively. Instrumentation to monitor pore pressures and the performance of the underdrainage system was described.

Design aspects of ancillary reinforced concrete structures were discussed. After a period of interesting questions the meeting closed with an expression of appreciation for Terry.

D.N. Jennings

2.2 ICOLD Meeting, Rio de Janiero

A meeting was arranged in conjunction with N.Z. Society on Large Dams on 16 September and the guest speaker was Henry Kennedy, MWD. Henry discussed aspects of the 14th International Congress on Large Dams (ICOLD) held in Rio de Janiero in May which he attended as New Zealand's ICOLD representative.

The first formality was the 50th ICOLD executive meeting. Henry notes that there were 74 member countries of which 45 were represented. They have 14 active select committees. A new one to investigate dam safety was established, with a 6 year brief, which created considerable interest. The Congress was structured around particular questions which included:

* Dam Safety and Operation. Henry noted that not including China there are some 14,000 large dams (height>15 metres) in the world.

* In situ testing.

* Reservoir Sedimentation and Stability. Although we can identify potential slides it is difficult to predict how they will behave.

* Materials. Rockfills are popular in which K generally $>10^{-5}$ m/sec and water content at compaction is not significant. The approach now is to leave in fines for greater density and strength and lower contact stresses. Geotextiles are still not favoured for main filters.

Henry referred those interested in further details to the five volumes of proceedings from the Congress.

The highlight of the Brazilian visit was the post Conference study tour on which Henry visited the powerful Tucurui scheme on the Rio Tocantins (Mean flow 11,000 m^3/s, the world's largest spillway capacity 104,000 m^3/s, dam height 86 m and installed capacity 8000 MW). He also visited the massive Itaipu scheme on the Parana River (mean flow 8,500 m^3/s, dam height 185 metres and installed capacity 12,600 MW). Henry's many slides provided an interesting view of these enormous civil engineering projects.

The large group enjoyed Henry's presentation and a lively discussion developed. Terry Kayes proposed a vote of thanks which was supported with acclamation.

D.N. Jennings

3.0 CHRISTCHURCH GROUP

3.1 Geotechnical Engineering in Hong Kong

On Thursday 4 November, John Rutledge presented a talk on "Geotechnical Engineering in Hong Kong - with an emphasis on slope instability" to an audience of 22. The presentation was similar to that delivered to the Wellington Group on 20 May which was fully reported in N.Z. Geomechanics News No. 24 p20-22.

John gave a well-balanced description of the local geology; the occurrence of slope failures in relation to the terrain and rainfall; terrain evaluation techniques and the evaluation of different methods of stability analysis. From discussion following the talk it was obvious that the range of topics dealt with was very much appreciated by an audience of varied interests and backgrounds.

B.R. Paterson

4.0 OTAGO-SOUTHLAND GROUP

4.1 Land Stability in the Kilmog Seacliff Area

Fifty-five people were present at a meeting held on 21 June in association with the IPENZ Otago branch. The discussion involved three speakers, Mr I.C. McKellar (retired from NZGS), Mr M.P. Stock (NZR) and Mr P.N. Jacobson (MWD).

Mr McKellar described the geology of the Kilmog and Seacliff area using a simplified geological map after Benson. Mr McKellar's lecture was illustrated with actual samples of the various rock types concerned.

Mr Stock discussed land stability as it related to the Railway Corporation's activities. The cost to the Corporation was estimated to be $750,000 per year maximum including overheads. Mr Stock mentioned that 13 sites in the Seacliff area were causing instability problems and of the 12 derailments over the last year, 5 were attributable to ground instability. Mr Stock outlined several methods that the Corporation had used to stabilise land movements.

Mr Jacobson discussed the problem in relation to the State Highway through the Kilmog area. The annual cost to the NRB was estimated to be $250,000 maximum including overheads. The main emphasis of Mr Jacobson's lecture was on Hammond's slip. Mr Jacobson used a simple sliding block model to describe this slip and proposed that, following a recommendation of Veder, a 5-10% improvement in the factor of safety was required to adequately stabilise a slip.

R.N. Croad

4.2 Tunnelling in Schist

Forty-three people were present at the meeting held on 1 November. Four speakers, namely Mr I. Pairman (Duffill, Watts and King), Mr R. Hayes (Dunedin City Council), Mr B.R. Paterson (NZGS) and Mr R. Ashcroft (MWD) gave addresses to the meeting.

Mr Pairman gave a discussion of the Waipori power scheme. This scheme, built just after the turn of the century and added to since, involves several tunnels passing through schist terrain.

Mr Pairman read extracts from Mr Dummond's 1939 paper on the Waipori system published in 'New Zealand Engineering'. He referred to two particular points:

(a) that blasting should be to the final payline since trimming proved to be uneconomic; and

(b) an excellent account of the bad ground conditions experienced in sections of the pressure tunnel.

Overbreak figures ranged from 150-200 mm for good schist rock to over 300 mm for poorer rock which required support. Hydraulic testing of the No. 1 and Branch supply tunnels indicated Mannings 'n' values of 0.04 for unlined tunnel and 0.015 for concrete lined tunnel based on the payline profile.

Mr Hayes discussed the Deep Stream tunnel, currently being constructed, which forms part of an extension to the water supply scheme for Dunedin City. The tunnel is in quartzo-feldspathic schists with foliation dipping at 10 -20^{o} along the tunnel line and which have a strong blocky character.

The dominant constraints which decided the tunnel line were outlined. In discussing the tunnel profile, Mr Hayes said that a 3 m high x 2 m wide profile had been chosen to minimise the amount of lining required, but that a concrete base was provided throughout for abrasion resistance. The various empirical and theoretical approaches for determining lining thickness were given, all of which generally lead to a lining thickness of 200 mm.

Excavation of the tunnel used conventional drill and blast methods. Generally 35-40 blast holes per round with 1.5 m pulls were being achieved. Overbreak largely controlled the transverse jointing of the schist.

Water problems were being experienced in the tunnel and in discussing the effect of the inflows on the excavation, the need to pipe the water to the portal to prevent sedimentation problems around the rail track and along invert drainage ditches was stressed.

Difficulties with site access associated with snow and heavy rainfall were being encountered. The low tunnel temperatures, typically 4 -4.5^{o}C, made underground conditions uncomfortable and caused problems in the initiation of water-gel explosives.

Mr Paterson opened his address on the influence of geological conditions on tunnelling in schist by mentioning the sparcity of empirical data on tunnelling in this rock type, particularly for lined tunnels which have been excavated to a design shape.

The differences between quartzo-feldspathic schist and pelitic schist were outlined. In summary:

(a) schist is a very heterogenous and anisotropic rock from both a lithological and textural point-of-view;

(b) schist contains a complex network of rock defects including penetrative schistosity, joints and a wide spectrum of faults;

(c) the behaviour of schist in construction is generally determined by the occurrence and distributions of the schist lithologies as well as the density and mutual geometrical relationships of the rock defects.

Initial geological investigation requirements for tunnels in schist would involve firstly standard surface geological mapping along the tunnel line extending far enough in all directions to obtain a satisfactory model of the geology and secondly a thorough aerial photo interpretation for a further extended area in search of superficial evidence of faults, joints, slope failures, springs, outcrop patterns and other features. The quality of the data was dependent on the occurrence and distribution of outcrops. An area barren of outcrops may well consist of relatively weak rock or harbour a major fault zone.

The next step is to explore the cylinder of rock to be excavated for the tunnel. This, however, is where investigations are often curtailed.

The need for subsurface investigations was emphasised. There was a bias in the surface data to recording of the resistant parts of the rock mass. In addition, subsurface investigations reveal data on the depth of weathering; persistence and openness of rock defects; lithological variations with depth and ground water conditions to mention a few factors. Also stressed was the need to establish a cut-off in subsurface investigations beyond which there may be minimal return for extra expenditure.

The detrimental effects that groundwater can have on tunnelling were discussed. A weak zone in a tunnel may in itself present only minor problems until it is subjected to concentrated groundwater inflows, when it may start to ravel and ultimately lead to progressive caving.

The combined effect of lithology and rock defects are important to tunnelling in schist. The difference in hardness between quartzo-feldspathic and pelitic schist affects drilling and blasting. Also pelitic schist breaks down more readily to form a silt which presents sedimentation problems with drainage and increases rail maintenance.

Schistosity plays an important role in the breakout of rock. With pelitic schist, the platey schistosity absorbs blasting energy and reduces its efficiency in cutting out a profile. The excavated shape is affected by the attitude of schistosity.

Mr Ashcroft's presentation discussed the Paerau Diversion tunnel currently under construction. The Paerau Diversion works constitute the headworks for the Maniototo Combined Power/Irrigation scheme. Storage water for the scheme is held in the Great Moss Swamp by the Loganburn dam, a 17.5 m high rock fill concrete membrane dam.

The main geological features of schist as they affect tunnelling, were discussed including rock types, weathering, schistosity, joints, shear, crush and gouge zones and water.

Mr Ashcroft followed with a discussion of the method of advance in the tunnel stating that conventional drill and blast methods were being used utilising face crews of 4-5 men on 3 shifts per 5 day week. He stated that generally 42 to 46 holes per 1.5 m round were used with 37 kg of ANFO explosive. The use of water-gel explosives by themselves, has not been successful due to the proximity of charged cut holes causing these explosives to desensitise.

The support systems in the tunnel included sets 100 x 100 x 19.4 kg/m steel sections at 1-1.4 m spacings. Rock bolts were the 1.8 m long x 22 mm diameter resin anchored type. Mechanically anchored bolts were considered unsuitable because of the rapid deterioration of the rock on exposure.

Problems were being experienced with squeezing ground in pelitic schist. Methods being used to control this were outlined.

Finally, Mr Ashcorft discussed the basis of payment used in the tunnel contract. He stated that payment for excavation was based on the geological classification of the ground encountered. Four ground classes were defined, three by specific reference to the type and level of geological defects present and for which payment was at scheduled rates. The fourth class covered more severe, but unspecified, conditions in which event payment for approved excavation methods became essentially reimbursement of cost.

R.N. Croad

FROM THE INTERNATIONAL VICE-CHAIRMEN

1.0 ROCK MECHANICS

1.1 5th ISRM Congress, Melbourne 10-15 April 1983

Two papers from New Zealand have been accepted for the above Congress.

(1) Pender M.J., Graham C.J., and Gray W.J. Prefailure Dilatancy and the Stress Distribution in a Closely Jointed Rock Mass.

(2) Hegan B.D., Read S.A.L., and Millar P.J. Geomechanics Investigations at the Proposed Raupunga Damsite, Mohaka River, New Zealand.

In total over 200 papers are to be presented and have been divided into the following themes.

A. SITE EXPLORATION AND EVALUATION

1 - Geophysical testing and exploration
2 - In situ and laboratory testing
3 - Classification, prediction, observation and monitoring
4 - Hydro-geology

B. SURFACE AND NEAR-SURFACE EXCAVATIONS

1 - Stability of rock slopes
2 - Foundations on and in rock, including dam foundations
3 - Near-surface construction especially in cities

C. DEEP UNDERGROUND EXCAVATIONS

1 - Mining excavations and mining methods including caving
2 - Permanent underground excavations including tunnels, power stations and storage caverns
3 - Coal mining including ground control and gas outbursts
4 - Prediction, control and measurement of subsidence
5 - Nuclear waste disposal and thermal behaviour of rocks

D. ROCK DYNAMICS

1 - Drilling and blasting
2 - Crushing and grinding
3 - Petroleum reservoir behaviour and in situ fracture methods for resource development

E. SPECIAL TOPICS IN ROCK MECHANICS

1 - Fracture and flow of the earth's crust, including tectonic stresses
2 - Numerical modelling of rock behaviour
3 - Future developments and directions in rock mechanics

The second bulletin for registration for the Congress is due for distribution and additional copies together with a list of the accepted papers will be available from the Vice-Chairman.

A post conference tour of New Zealand is planned and the possibility of inviting a keynote speaker to New Zealand is being investigated.

1.2 ISRM Commission on Geomechanics Computer Programs

The Commission is compiling a "List of Existing Computer Programs in Rock Mechanics" which it is intended should ultimately be published in Intl Jl of Rock Mech and Mining Sciences. Anyone who has developed a computer program for use in rock mechanics (excepting programs for hand calculators) is invited to submit an abstract for consideration for inclusion in the List. Copies of an abstract questionnaire form and instructions are available from the Vice-Chairman.

P.J. Millar

2.0 ENGINEERING GEOLOGY

2.1 Australasian IAEG Vice-President

The nomination of Mr D.H. Bell, Senior Lecturer in Engineering Geology at the University of Canterbury, Christchurch (also previously N.Z. Engineering Geology Vice-Chairman) for Australasian IAEG Vice-President has been fully supported by the Australian Geomechanics Society and will be submitted to the IAEG Executive Council at New Delhi in December 1982.

2.2 IAEG Bulletins

IAEG Bulletin No. 24 was received and distributed to members. If you have ordered but not received your copy please write to the Vice-Chairman.

This issue contains final reports on site investigations, recommended symbols for engineering geological mapping, and rock and soil description and classification for engineering geological mapping - prepared by IAEG Commissions. Members will find these useful for reference.

2.3 IAEG List of Members

A survey of members affiliated to IAEG was carried out recently, and a list was sent to the Secretary General IAEG for inclusion in an international list of members - the last list was prepared in 1978.

There are now 97 members of the N.Z. Geomechanics Society affiliated to IAEG of which 60 subscribe to the Bulletin. This is an increase of 17 above the 1980 membership.

2.4 Report on National Activities

A report on the N.Z. IAEG National Group Activities for 1981-82 was prepared for the IAEG Secretary General for inclusion in the next IAEG newsletter. Copies of the report may be obtained from the Vice-Chairman.

2.5 IAEG Council Meeting, Istanbul 1981

Minutes of the Istanbul IAEG Council Meeting were received, from which the following items may be of interest.

(a) papers are invited for Bulletins 26 and 27

(b) an IAEG newsletter should be published in 1982

(c) 1983 Annual IAEG Council Session will take place in Lisbon at the "International Symposium on Engineering Geology and Underground Construction".

3.0 SOIL MECHANICS AND FOUNDATION ENGINEERING

3.1 Technical Subcommittees

The ISSMFE has 18 technical subcommittees as listed below:

1. Information Advisory Committee
2. Site Investigation (pending more recent information from 1977-81 Subcommittee, extended)
3. Penetration Testing
4. Research Cooperation
5. Geomechanical Computer Programs
6. Sampling and Laboratory Testing:
   - 6.1 Residual Soils
   - 6.2 Soft Rocks
   - 6.3 Sand Gravels
   - 6.4 Soils of Volcanic Origin
7. Policy on Standards, Manuals, Specifications and Codes
8. Symbols, Units, Definitions and Correlations
9. Landslides
10. Centrifuges
11. Allowable Deformations of Buildings, and Damages
12. Tropical Soils, Laterites and Tropical Saprolites
13. Filters and Filter Criteria
14. Hydraulic Fill Dams, Tailings
15. Penetrability, Driveability of Piles
16. Preservation of Old Monuments and Cities
17. Constitutive Laws and Equations
18. Professional Practice- Responsibilities, Ethics, etc

Nominations for membership of the subcommittees are being sought. Would Society members who are interested in becoming a member of any of the subcommittees please contact the Vice-President.

G.J. Schafer

PROBLEMS ASSOCIATED WITH THE SHRINKAGE OF AUCKLAND CLAYS

C.C. Harvey, P.B. Riley, G.A. Pickens

1.0 INTRODUCTION

In April 1981, at the end of a rather dry summer, shrinkage problems were manifested over many areas of Auckland. The Geomechanics interest in the problem resulted in two local branch meetings in Auckland. At the first Mike Wesseldine presented his paper on "House foundation failures due to clay shrinkage caused by gum trees". His paper has been published in Trans IPENZ Vol. 9 1/CE March 1982. At the second, three speakers:

Colin Harvey from Kingston, Reynolds, Thom and Allardice, (formerly from Ceramco Ltd).

Peter Riley from Beca Carter Hollings and Ferner

Alan Pickens from Tonkin and Taylor

discussed different aspects of problems associated with shrinkage in Auckland clays - see Geomechanics News No. 22, p.18 for a general review. In this article the texts of the addresses of the three speakers are given in Chapters 2, 3 and 4 respectively. Aspects of the mineralogy, weathering, and slope and foundation stability of clays derived from the Waitemata Series rocks, which underlie Auckland, are given.

2.0 THE WAITEMATA SERIES SEDIMENTS FROM A CLAY MINERALOGY VIEW POINT

2.1 Introduction

The Waitemata Group sediments are used as a source of basic clay for the manufacture of bricks. This use of the clay for ceramics means that both the particle size and mineral composition are of interest. In this section the clay mineralogy approach rather than the clay size approach is used.

The geological history of the Waitematas is briefly reviewed followed by an outline of the typical weathering profile developed in the Waitematas, and its clay mineral composition. A technique for estimating both type and amount of clay mineral present is given together with examples from the Auckland area.

2.2 Geological History

The Waitemata Group sediments were deposited in what was believed to be a shallow Marine environment in mid Tertiary times - possibly 30 million years ago. The sediments were derived from an eastern greywacke land mass with some contribution from andesitic volcanism to the west from which were formed the Waitakere ranges.

These sources of supply contributed the products of subaerial weathering - kaolinites and Illites from the greywacke and kaolin and Montmorillonites from the andesites. Unaltered minerals from these sources were also washed into the basin, including rock fragments, quartz, mica, feldspars and heavy minerals.

The changes that occur during deposition in the marine environment are not known and depend on the physico-chemical conditions prevailing. Overseas studies in recent sedimentary basins (such as the Gulf of Mexico) have shown that kaolins, montmorillonites or illites washed into the basin from a land mass may be stable in the marine environment but transformations from one clay mineral to another can occur.

No such studies have been made for the Waitemata Series sediments although it may be stated that (i) under marine conditions and burial, alkaline and reducing conditions would prevail, high in Ca^{++}, K^{+} and Mg^{++}, and (ii) montmorillonite, illite and kaolinite might be stable while iron sulphide (pyrites) also would be formed.

Following their deposition and consolidation, the Waitemata Group sediments (referred to now as the Waitematas) were uplifted and eroded to form the present day exposures around the Auckland Harbour. The actual history of uplift and fluctuations in sea level can be related to the terrace levels and erosion surfaces around Auckland.

2.3 Weathering

Based on studies at three localities in western Auckland, the weathering profile developed during subaerial weathering of the Waitematas can be recognised. The various zones in which definite mineral transformations are taking place are summarised in Figure 1.

A compilation to the overall trends may be alternating layers of sandstones and siltstones, which are a feature of the Waitematas. Possibly high clay content layers may occur at depth because they were deposited as such in the marine environment. However, in the 3 profiles studied, the total clay content decreased with depth.

2.4 Measurement of the Clay Content and Type of Clay Mineral

Rapid techniques which give a good indication of the type and amount of clay minerals present have been developed and are based on two measurable properties of clays - ignition loss and moisture absorption.

ZONE		MINERALOGY CHANGES	OVERALL TRENDS WITH DEPTH
ATMOSPHERIC ZONE	LEACHING BY CARBONIC ACID IN GROUNDWATER	Feldspar and ash → Kaolin Montmorillonite → Kaolin Pyrites Unstable Clay mineral aggregates broken down → Clay sized particles released Removal of cations as sodium bicarbonates	CLAY CONTENT DECREASES KAOLIN/MONTMORILLONITE RATIO DECREASES PERMEABILITY DECREASES ALKALI CONTENT INCREASES
WATER TABLE ZONE Oxidising Reducing		Montmorillonite & Pyrites Stable	
ZONE OF LOW PENETRATION		UNWEATHERED WAITEMATAS (may contain significant clay mineral content in aggregates)	

Figure 1. Weathering Profile in the Waitematas.

Moisture absorption is determined by equilibriating undried clay at 75% relative humidity by placing it in a dessicator containing saturated NaCl and maintaining a vacuum for at least 48 hours. The moisture absorption, expressed as a percentage of the dry weight, is obtained by weighing the clay (a) after it has been equilibriated and then (b) after drying overnight at 100-110°C.

Ignition loss is determined by drying a pressed plaque of the clay at 100-110°C overnight and then heating it to 1000°C in a laboratory kiln. The ignition loss, expressed as a percentage of the dry (100°C) weight, is the weight loss obtained between 100 and 1000°C.

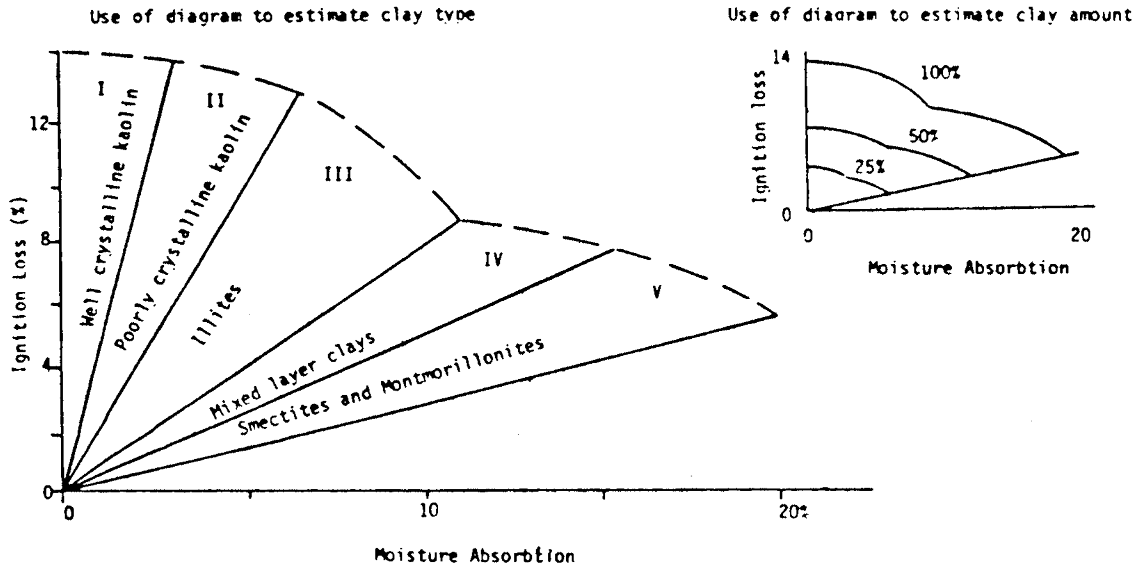
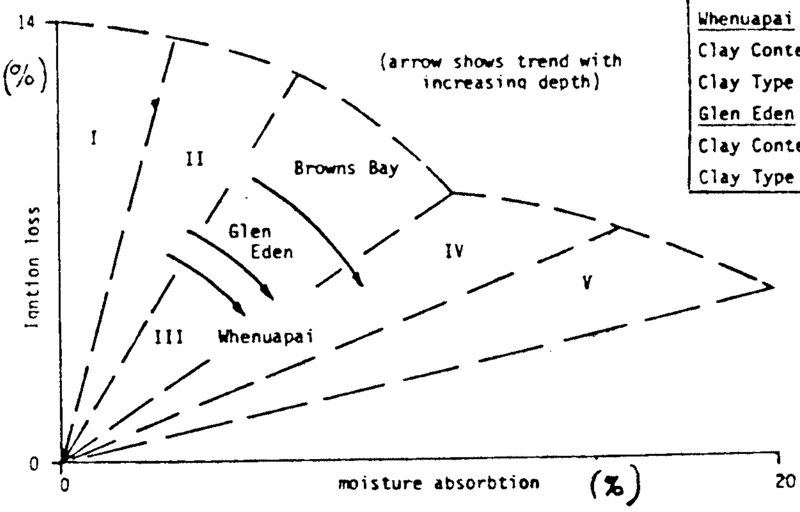


Figure 2. Determination of clay mineral type and amount using graph of ignition loss v moisture absorbtion.



	SURFACE		10m DEPTH
Browns Bay			
Clay Content	70%	progressive decrease	60%
Clay Type	III		IV
Whenuapai			
Clay Content	50%		45%
Clay Type	II		III
Glen Eden			
Clay Content	55%		50%
Clay Type	II		III

Figure 3. Trends with depth of clay mineral type and amount at three Auckland locations.

From these two properties a graph was constructed and modified after the initial work of Keeling of the British Ceramics Research Association to gain an indication of clay mineral type and amount (Figure 2).

LIMITATIONS - These methods are not effective in materials having either a high organic content or high iron content.

2.5 Studies in Auckland Area

Tests carried out on three profiles in Auckland at Browns Bay, Whenuapai, and Glen Eden showed the trends given on Figure 3 for both the amount of clay mineral and the type.

Chemical analyses using X-ray diffraction, differential thermal analysis and other techniques confirmed that the Waitematas are mixtures of kaolin, illites, mixed layer clays and montmorillonites in the general proportions which are indicated by the graph on Figure 3. Therefore, the typing of these clays into one of the five categories assists in estimating the kaolin to montmorillonite content and the relative amounts of clay mineral present.

2.6 Clay Mineral versus Clay Size

In the introduction the importance of clay mineral as against clay size was stressed. These data are a measure of clay mineral composition and, by investigation of the mineral distribution with particle size we have found that much of the clay mineral present is cemented as coarse aggregates which are compacted together as a consolidated mass. For example, the blue so-called Waitemata bedrock was shown to contain about 40% clay mineral as compacted aggregates.

However, if you subject these aggregates to acid leaching, the cement holding these agglomerates together may be dissolved away (possibly calcium carbonate cement?) and the clay minerals will be released as clay sized particles. It should be noted that the highest clay mineral content values we have found are close to 80% in brown surface clay at Browns Bay while the lowest, in well indurated blue 'bedrock' in the Western Suburbs of Auckland contained over 40% clay mineral high in montmorillonites. In the indurated agglomerate form, such clay-rich materials may present no problems, but if the clays are released by dissolving the cement, the engineering properties will be drastically changed.

For example, if during subdivision preparation or roading earthworks fractures or joints are opened up to allow the groundwater to attack 'consolidated clay-rich horizons' at depth. Then if cementing materials are dissolved, the clay-sized particles may be released.

2.7 Further Work

This work was carried out on samples from only 3 localities and from a restricted sequence of depths. No attempt has been made to apply these conclusions elsewhere in the Waitemata Group. However, a systematic study of the Waitemata Group and other cemented clay-rich sediments which outcrop so frequently in New Zealand may well be justified.

3.0 ENGINEERING CONSEQUENCES OF CHANGES IN CLAY MINERALOGY IN THE WEATHERING PROFILE

3.1 Introduction

There are many intriguing consequences of the clay composition of the Waitematas and these are introduced as ideas so that some of them may be related to any particular problem related to shrinkage and to slope stability.

3.2 Weathering of the Waitematas

The Waitematas are composed largely of clays with the percentage up to 80% in fine grained siltstones and 50% in sandstones. It is important to note that this is clay as a mineral not clay as a particle. The clay mineral in the Waitemata sandstone occurs as cemented agglomerates of clay minerals with these agglomerates in turn being cemented together to form the rock mass of the Waitemata Group. As weathering occurs the bonds between the larger particles break down, the strength of the Waitematas decreases, the soil structure becomes sandy. This slowly releases the primary clay minerals in the surface soil layers and initially the proportion released would be expected to be similar to those in the parent rock, with this being rich in montmorillonites in the reducing environment below the water table in the surface weathered layers. Initially the particles are sand and silt sized and behave accordingly, that is, they have moderate frictional properties and are only moderately plastic.

As further breakdown occurs the agglomerates of clay particles in turn break down resulting in clay sized minerals being present with a corresponding increase in the surface area of clay particles and an increase in plasticity and a reduction in strength. This material will readily absorb water and will also readily dry out.

Closer to the surface, above the water table, a mineralogical change occurs with the montmorillonites converting to kaolinites, which are less troublesome in engineering terms, with a lower plasticity and with high strength. These clays form the dessicated layers common around Auckland in the near surface soils. The montmorillonites are more severe in their shrinkage properties than the kaolinites, but both can give problems.

Figure 4 illustrates the changes in properties through the soil weathering profile. It is therefore not surprising that we have shrinkage problems in Auckland, with clay-rich rocks weathering to surface soils even richer in clay. The shrinkage problems are accentuated by our dry summers and wet winters and by our efforts in removing water from the soil with trees and from changes in the drainage pattern created by excavations and trenches.

3.2 Engineering Properties of Clays and Consequences

The Waitemata clays appear to reach a shrinkage limit at about a 20% moisture content, when removal of further moisture from the soil skeleton does not result in any further shrinkage. However, when water is added to the soil again the clay then swells and may either remedy the problem or create further problems.

Figure 5 shows a cut slope and illustrates the zones where troubles can be expected. In particular these are from reductions in moisture content in the normally fairly saturated soil layers in the middle of the weathering profile and from changes in the moisture content due to drainage above cut areas.

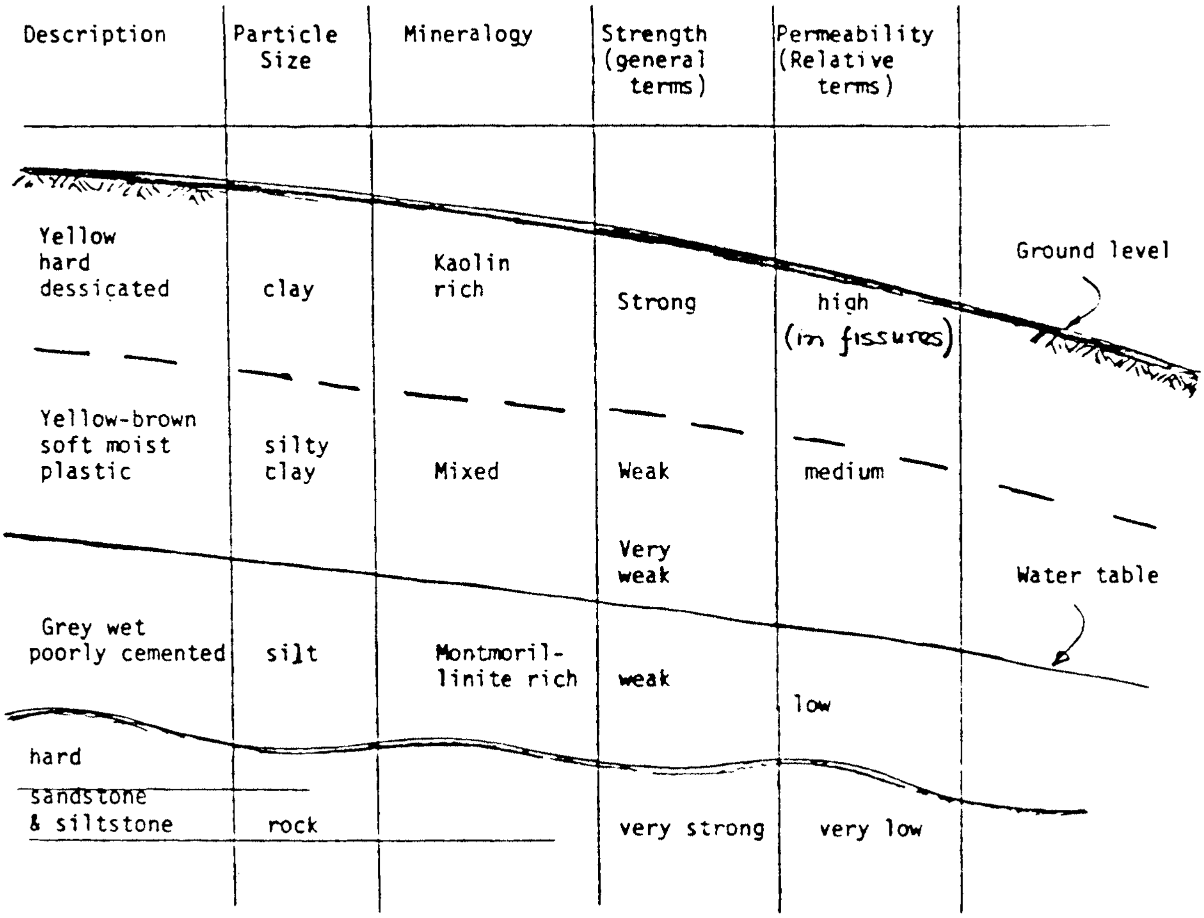


Figure 4. Changes in Soil Properties through the Weathering Profile.

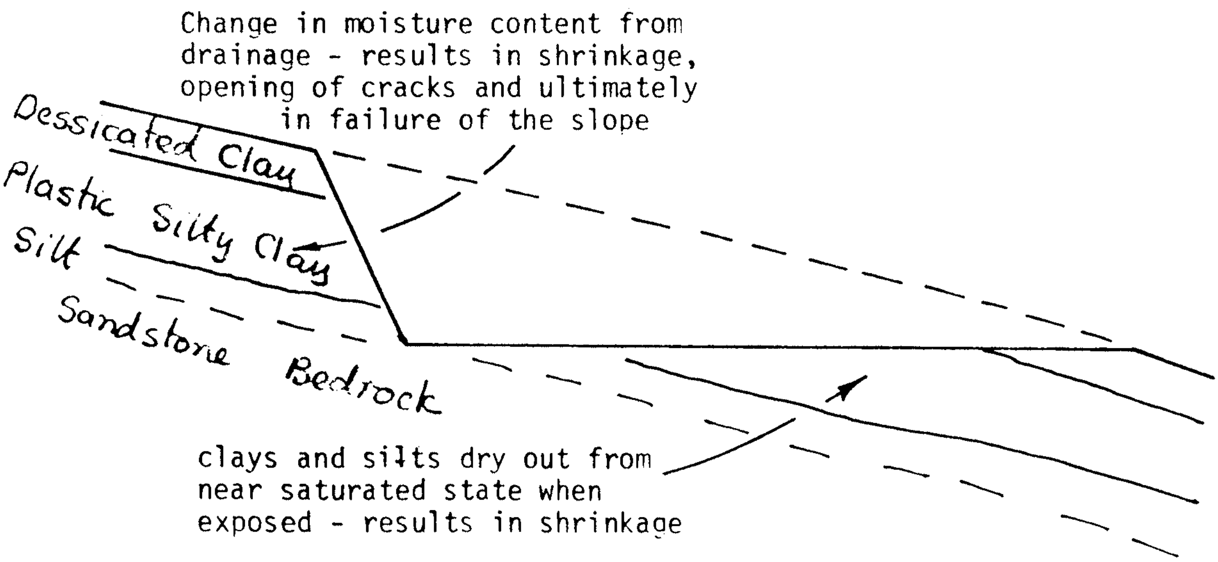


Figure 5. Shrinkage effects associated with the cutting of a Slope.

One further point for consideration is the implications the clay mineralogy has on slope stability. Montmorillonite is notorious in engineering circles as being responsible for some very spectacular slope failures. In the Waitematas, it has been shown that the montmorillonite rich layer is just above the bedrock and this in fact is where more slope stability problems in the Waitematas originate. There are other reasons for this other than just the presence of montmorillonite. Water percolating into a slope will flow downwards until it reaches an impermeable layer, and this is usually the very low permeability Waitemata sandstones and siltstones. The water will then flow down the slope, and if some discontinuity is present, will flow along this causing leaching with the acid bicarbonate water removing some of the alkalis such as sodium and potassium, breaking down the bond between the soil particles, and in fact producing clays along the discontinuity. This results in low strengths, and ultimately in slope movements along that plane when conditions are provided which lead to the factor of safety dropping below 1, such as in a heavy rain storm or when the slope is undercut by excavation.

3.3 Conclusion

The change in clay minerals through the very characteristic weathering profile of the Waitemata Group thus offers explanations for the behaviour of these weathered sediments. This behaviour diverges from classical soil mechanics predictions (developed on uniform northern hemisphere clays) because of the weathering pattern and mineralogical differences. Auckland City is built largely on these weathered sediments, and all of us working in geomechanics must deal with the problems of shrinkage and slope failure. Solutions will be more successful when the mechanisms causing failure are understood.

4.0 MEASURES FOR REMEDYING PROBLEMS CAUSED BY AUCKLAND CLAYS

4.1 Introduction

The problems arising where concrete floor slabs are cast over swelling clay are outlined, followed by examples of problems in shrinkage clays together with some example remedies.

4.2 Concrete Slabs

Problems with concrete floor slabs founded on clays arise when the water content equilibrium of the underlying clay is changed. Construction over these materials requires a certain amount of management to avoid changes in water content. For example, a plastic clay site should not be cut to grade and then left unprotected for some time before further development occurs. In the summer, exposed soil will dry and shrink, and the ground can swell when the ground surface is eventually sealed, causing distortion and cracking of the floor slab. The risk of a wet clay drying out and shrinking after construction is generally not very great because the equilibrium water content under the slab tends to be high.

There are severe practical problems associated with developing large industrial sites on plastic clay in winter conditions. One method of protecting the subgrade is to grade it to parallel subsoil drains before overlaying a running course of hardfill. Another method that has been used successfully is lime stabilisation of large surfaces, permitting winter trafficking without the use of large amounts of hardfill.

Finally, engineers should be reminded that floor slab movements caused by volume changes of clay subgrades are less likely to cause cracking of the floor slabs at their edges if the slab is not structurally tied into the perimeter foundations.

4.3 Private Housing

Now the problems just described are known by the engineering profession, but, although similar situations arise in housing construction, they are not generally understood by the private homeowner. There are countless examples of damage to houses in areas of Auckland which have plastic clay soils at the surface, for example: East Coast Bays, Remuera, Pakuranga, Half Moon Bay - in fact nearly every part of Auckland, outside the areas covered by volcanic material. Much of the damage is attributable to soil shrinkage accentuated by large trees close to the house. The worst offenders seem to be Silver Dollar trees and Acmena Hedges, whose roots extend out further than the height of the tree (see Mike Wesseldine's paper).

The remedies to these difficulties are many and varied and also depend on the attitude of the homeowner. Part of the answer may be to cut down trees close to the house but to some people, privacy is more important than the appearance of their house or difficulties with opening and closing doors. The simplest means of alleviating the effects of soil shrinkage is simply to keep the ground moist with frequent watering. This technique has stopped further deterioration in many cases and often leads to a significant degree to recovery. Another approach I recommend was to intercept the lateral roots of a neighbour's large acmena hedge which were increasing shrinkage locally and causing severe cracking of brickwork. A trench was dug below root level and filled with concrete and the local area wetted up. This relatively cheap do-it-yourself method worked.

More elaborate techniques to prevent further damage and achieve more positive restoration involves some type of underpinning. Damage is often concentrated at the corner of the house. Bored piles can be installed just outside the foundation line at the corner with a beam cast between the pile tops. The house can then be jacked and the foundation attached to the beam as sketched on Figure 6 below.

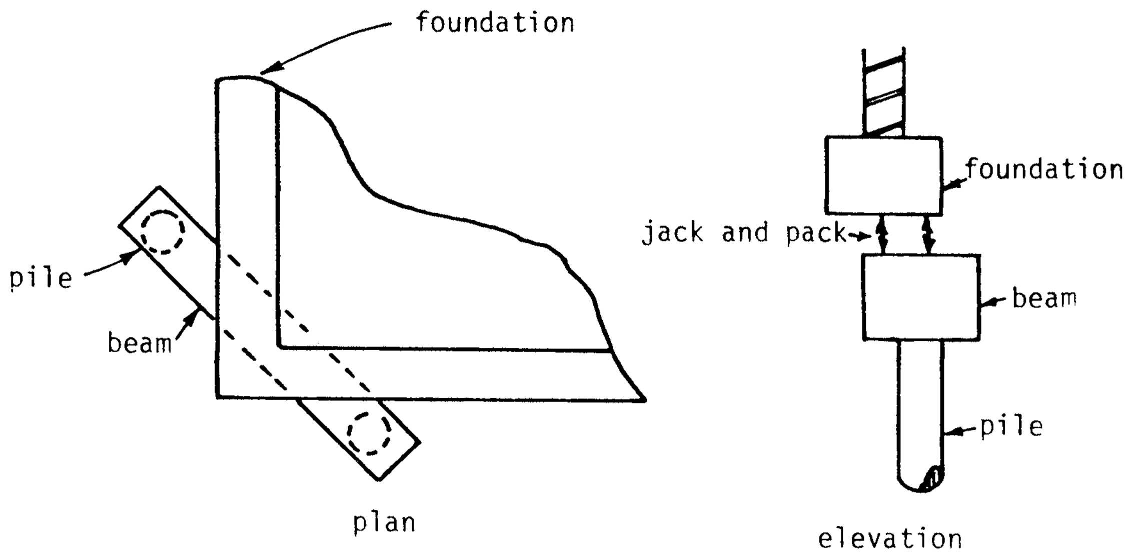


Figure 6. Corner Underpinning.

This type of work is labour intensive, often beyond the houseowner's ability to do it himself, and hence fairly expensive. In 1980 terms, such a corner repair would cost \$2000-\$3000. A large part of the work can be done by the home handyman and in this case the cost would be reduced substantially.

We do not have accurate information on the depth affected by shrinking and swelling. A figure of 1 metre is quoted fairly frequently. The usual practice is to install the piles to a depth of approximately 2 metres.

A slightly different approach is used to underpin the side of a house. A vertical pile can be bored close to the foundation and a corbel can then be cast on top of the pile to support the house foundation. In some cases where extensive underpinning is necessary and foundation conditions may not be suitable for drilling, inclined steel H piles can be driven. Both arrangements are shown in Figure 7.

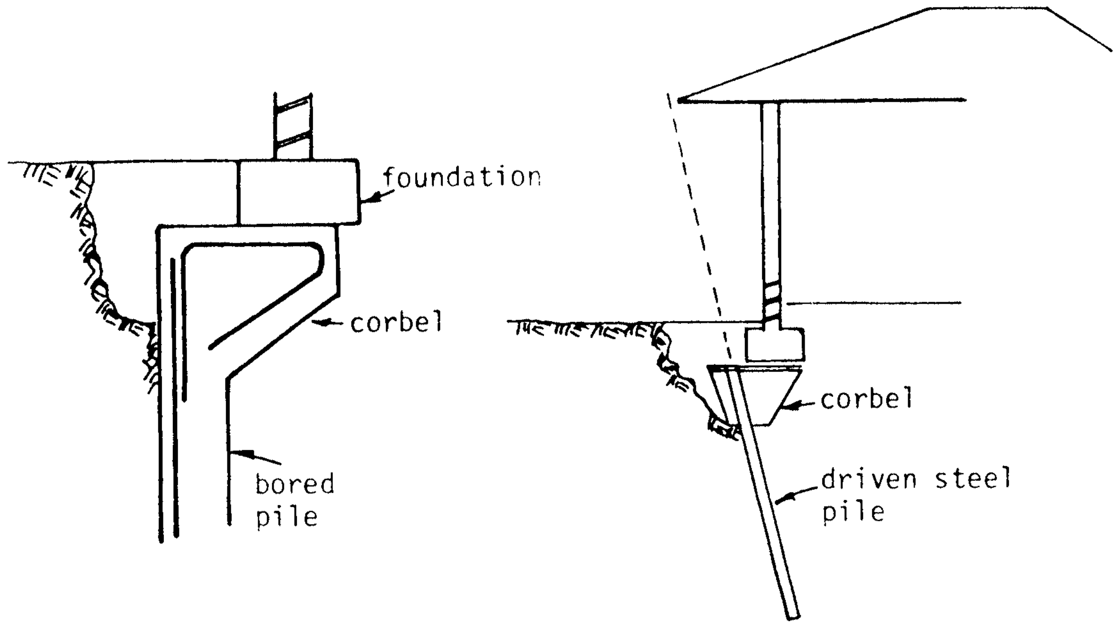


Figure 7. Underpinning of the side of a house.

Once again this type of work is fairly expensive. Typical costs (kindly supplied by Gilberd Hadfield Limited) range from $5000 to $6500 (1980 terms) for one side of a house to $20,000 for a whole house.

Clearly the best solution is the proper design and construction of foundations to eliminate the problem right at the outset.

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AUCKLAND

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
P.O. Box 12241
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)

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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.) Affiliation fees are in addition to the Geomechanics Society membership fee of $15.00.

I wish to affiliate to:

International Society for Soil Mechanics and Foundation Engineering	(ISSMFE)	Yes/No	($6.00
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PLEASE DO NOT SEND FEES WITH THIS APPLICATION. AN ACCOUNT WILL BE RENDERED ON YOUR ACCEPTANCE INTO THE SOCIETY.

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P.O. Box 12-241
WELLINGTON

Dear Sir,

CHANGE OF ADDRESS

Could you please record my address for all New Zealand Geomechanics Society correspondence as follows:

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