N.Z. GEOMECHANICS NEWS

No. 19

NOVEMBER 1979

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. 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.

Members of the Society are required to affiliate to at least one of the following International societies: Soil Mechanics, Rock Mechanics or Engineering Geology.

EDITOR'S NOTES

1. 3rd A-NZ Geomechanics Conference

By the time this issue is published all the papers should be at hand and the review process well underway. The third bulletin, due before the end of the year, will have details of the conference programme and a list of papers.

An addition to the programme since the note in the previous Geomechanics News is that there will now be two invited lecturers. Professor Brekke from the University of California at Berkeley will be joined by Dr Roy Northey. They will each have a time in the conference programme. Dr Northey has agreed to present his 3rd NZ Geomechanics lecture to the conference, as the organising committee felt that the lecture would be of considerable interest to the visitors from outside New Zealand.

Considerable interest is being expressed in the Conference. If you wish to receive further information, please apply (if you have not already done so) to:

The Organising Secretary
The Third A-NZ Geomechanics Conference
c/o Barr Burgess & Stuart
Box 243, Wellington

2. Soil Description

The last three issues of this newsletter have contained letters on the subject of soil description. These letters have expressed concern at the considerable variation in this country of methods of describing soils for engineering purposes. It seems that we have gone to great lengths to standardise our soil mechanics test methods yet have neglected what must surely be the most basic tool and means of communication for the geomechanics practitioner - a rational and accepted method of soil description. The Society is the vehicle whereby a suggested method for soil description can be initiated.

3. 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.

4. 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.

5. Contributions Wanted

Contributions to NZ 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, NZ Geomechanics News, c/o NZ Geomechanics Society, P.O. Box 12-241, Wellington North.

A.J. Olsen
Editor

PUBLICATIONS OF THE SOCIETY

The following publications of the Society are available:-

a) From the Secretary, N.Z.I.E., P.O. Box 12-241, Wellington North:

- Proceedings of the Hamilton Symposium "Tunnelling in New Zealand" November 1977. Cost $18.00 to members, $20.00 to non-members.

- Proceedings of the Second Australia - New Zealand Conference on Geomechanics, Brisbane, July 1975. Cost $25.00 but as a special offer this is discounted to $15.00

- Proceedings of the Nelson Symposium "Stability of Slopes in Natural Ground", November 1974. Cost $15.00 to members, $18.00 to non-members.

- Proceedings of a Workshop on Lateral Earth Pressures and Retaining Wall Design, February 1974. Cost $1.30

- Proceedings of the Wanganui Symposium "Using Geomechanics in Foundation Engineering", September 1972. Cost $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. A limited reprinting is available at $8.00 to members, $10.00 to non-members.

- Copies of most issues of "N.Z. Geomechanics News" are available to members at a nominal cost of $2.00 per copy.

b) From Government Bookshops:

- "Slope Stability in Urban Development" (D.S.I.R. Information Series No. 122) Cost $2.00

T.J. Kayes
Publications Officer

THE FLOW SLIDE PROFILE

I. J. Smalley

Among the most dangerous and damaging types of landslide are those in which sufficient of the required factors combine to produce an actual flow of material. Waltham (1978) neatly defines a flow slide as a landslide in which the moving debris acts temporarily as a liquid, and he states that there are a number of different ways by which this liquefaction can take place. These types of landslide can develop in various situations; the great lahar mudflow which carried away the Tangiwai railway bridge in 1953, the tip slide at Aberfan in 1966, the flow slide in very sensitive soil at St. Jean Vianney in 1971 - these were all examples when liquefied material caused great tragedies. They all have the flow type failure in common, and this provides a basis for comparing them, and for looking at some rather exotic failures in an attempt to make some useful generalisations about the flow slide mode of failure. This article is essentially an exploration of some flow slide types; the eventual aim is to find ways of comparing these phenomena and perhaps learning something useful about fundamental mechanisms of initiation and progression.

The simple comparative method which has been devised is that of the flow slide profile - a sort of fingerprint for each slide. Each flow slide consists of a mobile system of particulate material dispersed in a supporting medium and five factors can be recognised which make some initial comparisons possible:

(i) mass of particles in the system
(ii) density of the supporting medium
(iii) trigger energy factor
(iv) bond/weight ratio
(v) topography/potential energy factor

The mass variation can be large, from the rock fragments of the Elm rock slide to the clay-sized particles in the sensitive soil at St. Jean Vianney; the supporting medium will be air or water; the trigger energy may be very low - some slight increment of stress which precipitates vast movements, or it may be a very energetic earthquake such as mobilised the Niigata sands; the bond/weight ratio gives an indication of particle interaction in the system - it measures the relative effect on the particles in the system of interparticle bonding and gravitational forces; and the topography factor essentially indicates the site for the start of the flow - in some cases the potential energy is very high, as at Elm, but in some cases it appears very low, as at Niigata. A simple qualitative assessment of the factors involved in the slides under consideration is shown in Table 1.

Although the slides discussed have in common their flow type failures they do not all fit together comfortably in any slope failure classification scheme. In the Northey, Hawley and Barker (1974) scheme they would probably be distributed among sections 5e Mudflows, 5g Flow failures, possibly 6b Clay flows and 8 Sub-aqueous slides. Sub-aqueous flow slides undoubtedly fall within the scope of this paper, but are not discussed except for the possibly marginal case of the response of the alluvial plain at Niigata to the 1964 earthquake. The flow slide profile will be illustrated by consideration of six of the more geotechnical examples from Table 1.

TABLE 1: Flow Slide Factors

Event	M Mass of particles	S Support medium density	E Energy	R Bond/ weight ratio	H Topography
Sensitive soil slide e.g. St Jean Vianney	Very small	water	low	high	low
Dry snow avalanche	small	air	low	medium	high
Nuee Ardente	medium-large	air	high	low	high
Ignimbrite flow	medium	air	high	low	low
Waste tip slide e.g. Aberfan	large-medium	water	low	low	medium
Lahar type mudflow	small	water	low	medium	high
Rock slide eg Elm	large	air?	high	low	high
Niigata earthquake	sand	water	high	low	low
Loess slide e.g. Kansu	small	air	high	high	high

1. St. Jean Vianney, 1971 (See Fig. 1)

The St Lawrence Valley in Eastern Canada is classic ground for flow slides. The 1971 slide at St Jean Vianney was the most serious slide of recent times and gives a profile which is characteristic of the flow slide in a very sensitive soil. The particles are small, about clay size, and the supporting medium is water. No particularly large shock was observed to initiate the St Jean Vianney slide and this is normal with the E. Canadian clays. The bond/weight ratio R is high which accounts for the apparent stability of most of these deposits. The bond/weight ratio concept was devised in an attempt to explain why these sensitive soils could be very cohesive and yet have very low plasticities (Smalley 1971). The flow slide event is facilitated by the fact that the soil consists largely of clay sized primary mineral particles (quartz and feldspar mainly) and the brittle bond between such particles can be strong but is of a strictly short range nature.

Recent mineralogical studies of the sensitive soils in Quebec (Bentley & Smalley 1978) show that there is very little clay mineral material present, which means there is a corresponding lack of long range interparticle bonding forces.

These 'quickclays' are among the most startling of natural flow slide materials. They appear in so many ways like a normal soil that the sudden transition from solid to liquid is even more remarkable. The profile allows comparisons to be made with other flow slide systems but it does not reveal the particular mineralogy which appears to be the critical factor controlling the quickclay flowslide.

2. Aberfan, 1966 (see Fig. 2)

The collapse of No.7 tip at Aberfan in October 1966 illustrated the fact that a man-made system could suffer a devastating flow slide just as readily as a natural system. All the classic conditions were met; the particulate material was available in convenient sizes and without any significant bonding, and the water was there in abundance from buried springs to buoy up the particles, and the relief of the valley gave the system enormous potential energy. Even factors such as the method of tipping (without any subsequent compaction) served to set up an ideal flow slide system. M was fairly large, the material being coal mine waste, and with H being large as well this facilitated slide movement. There was actual injection of water from the springs so the system was adequately supplied with a supporting medium, and the tip design allowed a well placed rotational slip to develop in the tip face which triggered off the flow slide. The actual energy required to initiate the slide was small and the high H value ensured that a long run was obtained.

As an example of negligence the Aberfan affair has seldom been surpassed and there is no doubt that the simplest of precautions could have prevented it. The lessons are clear: site investigation for tips is of vital importance and these should be continuously monitored. Waltham (1978) noted that before 1966 ungraded material (wide spread of size) - Waltham is a geologist - such as that on Tip 7 was thought by many not to be prone to liquefaction, but now of course we know that they are. The boundary conditions for flow slides in waste tips have been considered elsewhere more fully (Smalley, 1972).

3. Tangiwai, 1953 (see Fig. 3)

Just 1900 years ago the town of Herculaneum was buried by a flow of saturated ash from the slopes of Mount Vesuvius. The H factor was critical in allowing the flow slide to develop its catastrophic proportions, as it was on Ruapehu in 1953. The suspended material was relatively fine volcanic ash which was carried very efficiently by large quantities of water. The initial trigger energy need not have been great, a much more significant source of energy was derived from the H factor. The R value would be low; this is a definite contribution to the danger from ash deposits. Since they have been formed usually as airfall deposits they have a very open structure, the co-ordination number of a typical particle is low and the overall interparticle bonding is of a relatively low strength. Had the material been compacted the danger would have been reduced - as was the case at Aberfan. Material dumped from high level buckets must have a relatively open structure, which can easily be modified by running a bulldozer over the tip once or twice.

FIG. 1 St Jean Vianney

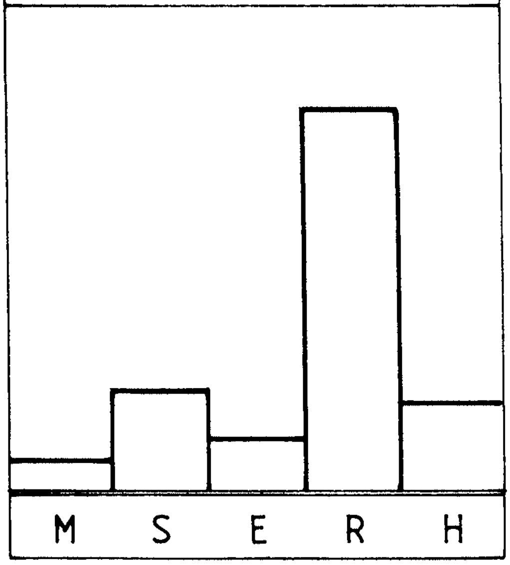


FIG. 2 Aberfan

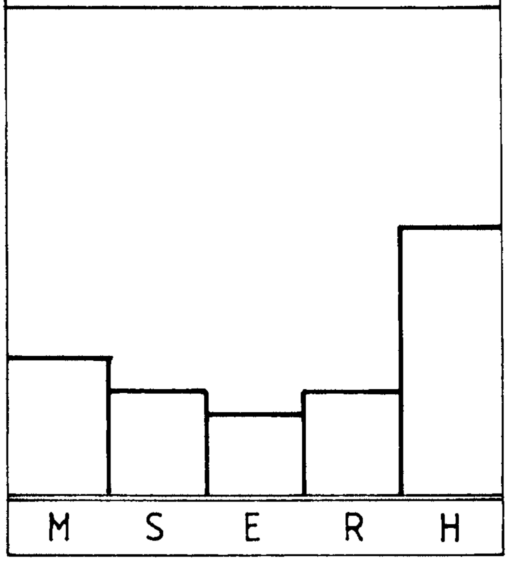


FIG. 3 Tangiwai

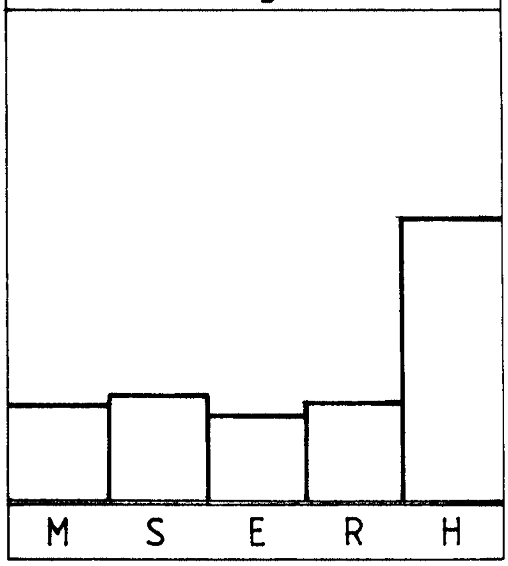


FIG. 4 Elm

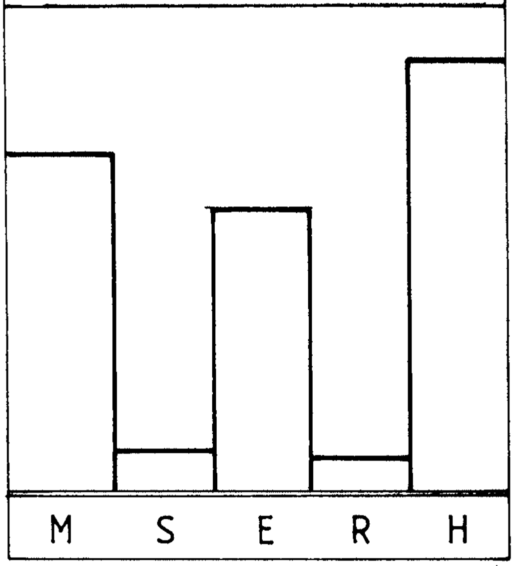


FIG. 5 Niigata

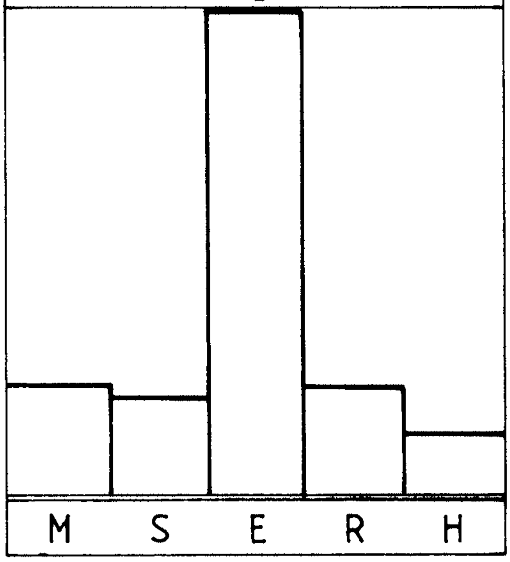
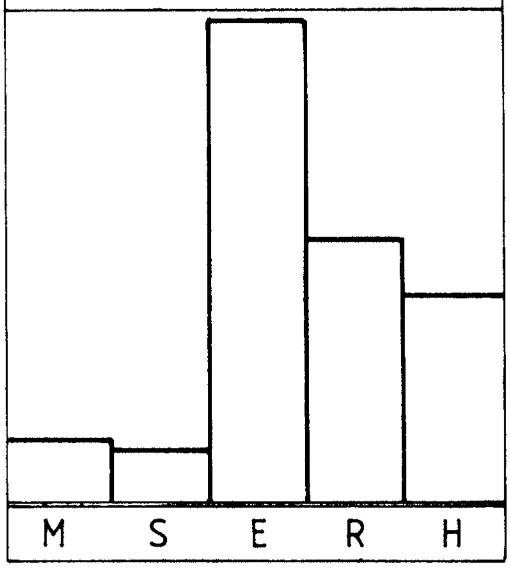


FIG. 6 Kansu



M = MASS OF PARTICLES

S = SUPPORT MEDIUM

E = TRIGGERING ENERGY

R = BOND/WEIGHT RATIO

H = TOPOGRAPHY

4. Elm Rockslide, 1881 (see Fig. 4)

In 1881 a large section of the Plattenberg mountain fell over 1500 ft and landed very close to the village of Elm, in Glarus Canton in eastern Switzerland. The fall was caused by unwise quarrying and over 400 million cubic feet of rubble crashed to the valley floor and in less than a minute travelled nearly a mile before coming to a dead stop (Waltham 1978).

Some detailed studies of the Elm event have been carried out by Hsu (1975, 1978) who has reaffirmed the validity of many of the observations made by Albert Heim soon after the event took place. Heim concluded that rockfalls did not slide, they crashed, and their debris flowed. Hsu suggests that the term 'Sturzstrom' be used to designate the particular kind of exceedingly rapid debris flow generated by a rockfall or rockslide, and he has concluded that the flow movement is an example of Bagnoldian grain flow. This means that we have a subtly different type of flowslide to consider in that in the Bagnold flow mechanism no supporting medium is required and the debris cloud maintains its dispersion by interparticle collisions (Bagnold 1956); so S could be zero. M would be high as quite large rock fragments were involved, and E is high because of the peculiar geometry of the Elm fall: the initial flow energy was gained from the impact of the falling mass, this broke up and formed the debris cloud. So H is also effectively high and R is negligible. In fact it is possible that the Elm event should not be considered with the other slides; there is no doubt that Hsu's perceptive studies should prompt another close look at accepted modes of motion of debris flows.

5. Niigata, 1964 (see Fig. 5)

The Japanese coastal city of Niigata was built on a flat alluvial plain of porous, undercompacted sand, which liquefied extensively during an earthquake in June 1964. The gravitational stress which is usually necessary to cause a flow slide was absent at Niigata in the usual sense - the site was flat; but a gravitational stress was available. This was supplied by the buildings which applied a considerable load to the ground and caused the sand-water system to flow. The high E value was the critical factor in the event, as it was in the Kansu loess slide; sufficient energy was introduced into the system for the widespread liquefaction to occur.

6. Kansu, 1920 (see Fig. 6)

This must have been one of the largest landslides ever; Close and McCormick (1922) estimated that it killed 100,000 people. The material was loess, silt sized and homogeneous - so the particle mass was small. However it was cemented to a certain extent and under normal conditions was strong and stable. The event which caused the landslide was a powerful earthquake which produced stress waves in the thick loess deposits which had the effect of separating the particles and mobilizing them. The E column in the histogram is the critical one - the flowslide was truly the result of an enormous input of energy. Close & McCormick described the consequences: "The most appalling sight of all was the Valley of the Dead, where seven great slides crashed into a gap in the hills three miles long, killing every living thing in the area except three men and two dogs. The survivors were carried across the valley on the crest of the avalanche, caught in the cross current of two other slides, whirled in a gigantic vortex, and catapulted to the slope of another hill. With them went house, orchard, and threshing floor, and the farmer has since placidly begun to till the new location to which he was so unceremoniously transported."

On a much smaller scale it appears that flow failures can occur in wet loess on slopes. Failures of this type have been observed in the Wairarapa and the profile for these would show a larger S value (water rather than air) and a much lower E value; H would probably stay about the same.

In conclusion, it is apparent that if a granular mass can be mobilized and if the particles can be supported and if there is sufficient potential energy available, then a flow slide situation is possible. In some cases such as the very sensitive soils, it appears that the flowing fluid state is nearer to a natural state than the solid form, which only contrives to exist by virtue of the relatively high interparticle forces. Certain either/or situations can be detected whereby if E is great enough the density S can be very small, as in the case of the Kansu loess slide and the Elm rock slide, and there is no doubt that the high E value is a major threat where flow slides are possible. At Anchorage in 1964 it took a powerful earthquake to mobilize the buried sensitive soil, but once the soil was mobilized large amounts of damage were caused. Our areas of ignorance are still considerable; Hsu's (1978) new look at the Elm slide has cast some doubt on the air cushion theory and more investigation would be useful there; there is still no real concensus on what causes the extremely high sensitivities observed in some of the postglacial clay soils in Canada and Scandinavia, and no-one can yet predict where the next major earthquake will strike. Perhaps the best we can do at the moment is to be aware of the great range of possible flowslide situations and endeavour to take all sensible precautions. The horror of Aberfan still remains and it is made all that more poignant by the realization of how absurdly easy it would have been to prevent that particular disaster.

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LETTERS TO THE EDITOR

The following items of correspondence have been received by the Editor:

Sir,

The letter from L.D. Wesley which appeared in the last issue provides an excellent clarification of the terms definitions of fine grained soil fractions (silt, clay, silt fraction, clay fraction) and their behaviour as engineering materials. He is quite correct in stating that their behaviour is influenced by both particle size and particle composition; the principal influences being clay type and the proportion of clay mineral in the clay fraction. However, his view that "particle size measurements are rarely performed on fine grained soils" and that "classification of fine grained soils on the basis of particle size is not a useful exercise and has long been rejected by most engineers" requires comment.

We would suggest that it is standard practice for most civil engineering laboratories to carry out grading determinations on fine grained soils in conjunction with behavioural tests such as Atterberg Limits. In common with most overseas standards, NZS 4402 P (Part 1) includes within its 'Soil Classification Tests' section, the laboratory methods for the determination of both particle size distribution (Tests 9C and 9D) and behavioural characteristics of fine soils.

The USBR Earth Manual (2nd Edition, 1974, p.3) states that "size distinction is not made between silt and clay" but the Unified Soil Classification chart (p.12) implies that behavioural tests (dilatency, dry strength, toughness) performed on a remoulded sample are field identification procedures only and states that it is necessary to "use (the) grain size curve in identifying the (soil) fractions as given under field identification". The behavioural characteristics of the remoulded soil are determined in the laboratory from the testing for the Atterberg Limits and confirm the unified soil symbol (eg CL, CH) as assigned in the field.

The example given on the chart (CLAYEY SILT) thus has a name implicitly derived from its grading while its unified soil symbol (ML) is determined from the behavioural characteristics of the remoulded sample. Recognition of a soil such as a CLAYEY SILT solely from field identification criteria would appear difficult, and it is therefore necessary to clarify whether field or laboratory procedures (including grading) are to be used separately or in conjunction, when naming/describing fine grained soils.

We would also point out that the soil description method used by the Engineering Geology Section of NZ Geological Survey and introduced by G.W. Borrie (NZ Geomechanics News 17, November 1978) is based on the Unified Soil Classification with modifications to allow for the description of the in situ characteristics of the soil. This method is aimed primarily at improving logging methods by standardising description format, and, although it may at first appear cumbersome, it very rarely results in a complex form of description. The basis of the system is summarised on a single sheet of paper for inclusion in reports and soil descriptions within the report text are tailored to meet the needs of that report.

The suggestion that soil (and rock) description methods should be standardized within New Zealand via the Geomechanics Society is again supported and the soil description method used by NZ Geological Survey is available to initiate this standardization.
Yours faithfully, S.A.L. Read, D.F. Macfarlane.

Sir,

NZS 4402, TELARC, and GEOMECHANICS

It is clear from the report in NZ Geomechanics News No.18 of the symposium in Auckland last November that all is not sweetness and light between NZS 4402 Part 1, TELARC and geomechanics. From my own experience similar troubles have arisen in other areas and I know of yet others at second hand. In all such cases I suspect the trouble arises because the technologist, who should know better, forgets that testing standards and TELARC are only tools whose value in practice is never more than the skill with which they are used. The sharper the tool the more damage it can do in unskilled hands!

Now, I believe that both NZS 4402 and TELARC are good serviceable tools and a credit to their originators and developers. But what purposes do these tools serve and how can they be put to good and effective use in geomechanics? And, further, are there any ways in which their usefulness can be improved?

The purpose of NZS 4402, as I understand it, is to set out the agreed test procedures which are appropriate, in general, to the engineering testing of New Zealand soils. The standard does not, and indeed cannot, set out the manner in which these test procedures should be selected and their results applied in any particular case. That is a matter for the skilled geomechanics technologist to decide. The same can be said of virtually every testing standard.

While this fact is virtually self evident in the case of part 1 of NZS 4402 (only the ignorant or foolish would predict a soil's precise properties from its PI; or its colour; or the phase of the moon at the time of sampling!) it applies with equal force to the whole of soil testing. For example the strength and consolidation tests of part 2 of NZS 4402 are only more or less crude attempts to model the real life situation. The aptness of the model and the wisdom with which the results are interpreted and applied are always vastly more important in the end than whether the test was done in accordance with a particular standard. Wise interpretation of results always includes evaluation of the appropriateness, or lack of it, of the test procedure gone through, and of the quality of the testing work.

The purpose of TELARC is to provide a warranty that a particular testing organisation has demonstrated its competence to carry out those tests for which it is registered. It gives no more than an inferred warranty that a particular test has, in fact, been carried out correctly. It does not and can not give any assurance that the testing done was necessary or relevant to the problem nor that the testing organisation has any skill whatsoever other than that of carrying out tests in accordance with the letter of the specification.

Thus NZS 4402 whether or not associated with TELARC, gives no more than an appearance of competence. I think it might be a wise move to point out the difference between this appearance and the substance of the matter by attaching a disclaimer on the following lines to every set of test results:

"On their own these test results are meaningless. They only have meaning when evaluated in the context to which they may be applied. The amount of meaning they then develop depends primarily on the knowledge, experience and wisdom of the evaluators, not on the precision of the testing."

and where TELARC is involved the following extra disclaimer:

"TELARC registration only relates to competence in testing. It gives no assurance whatsoever that the testing organisation is competent to evaluate the test results or to judge their relevance".

Such disclaimers would, I hope, reduce the risk of good tools being misused.

While the obvious way to increase the effectiveness with which NZS 4402 and TELARC are used is to upgrade the competence of those who claim to be geomechanics experts, there are, I believe a couple of ways in which their general usefulness as tools could be improved. Firstly NZS 4402 (or for that matter any testing standard) cannot be more than a codification of what is essentially routine nor cover more than a fraction of the possibilities. Thus the first task of the drafting committee should be to set down clearly the exact area they intend to cover. It would be very helpful to have this statement published as an integral part of the standard so that technologists contemplating using the standard could judge its relevance to their problem without having to acquire a detailed knowledge of its provisions (and of the provisions of other possibly suitable standards).

Secondly, while I believe TELARC must at all costs avoid claiming special competence in any technology which testing laboratories are set up to serve, there is one very common aspect of applied testing to which they could profitably extend their influence. This is the sphere of quality assurance. I have more than once been appalled by the naivety of otherwise competent technologists in this area. They assume that a quality assurance problem can be solved merely by requiring TELARC testing! The community will only reap the return on their investment in TELARC when the improved testing that TELARC has brought about is properly employed in industry and commerce. A well conducted, continuing education programme in the concepts of quality assurance is essential to this end and TELARC seems to me to be the most appropriate educator.

Thirdly, naivety about the place of testing in geomechanics is widespread. Many clients, or worse, their consultants (who should know better), specify irrelevant tests and standards of precision. It is quite common to find clauses in earthworks specifications such as "All tests shall be in accordance with NZS 4402 and shall be carried out by a TELARC registered laboratory". What should the competent geomechanics practitioner do in such a case? What I suggest is that he adds a note to his bill as follows:

"The following tests required by the specification were irrelevant to the problem .... They were carried out as specified by a TELARC registered laboratory at a cost of $........

Other, relevant, tests required by the specification were also carried out to the required standards and additional relevant testing done to appropriate standards. This relevant testing was instrumental to a satisfactory outcome and cost $....."

Even the most prodigal of clients is going to start asking questions when he sees his money squandered on irrelevant tests. In time enough questions will be asked to make the "blanket" specification writer think again, and again!

Finally, I repeat that both testing standards and TELARC (and for that matter testing laboratories) are very useful and necessary tools in the hands of competent technologists. But in the hands of the unskilled they can be dangerous. They become even more dangerous if they cease to be passive tools and develop wills of their own. Were that to happen the whole of geomechanics would degenerate into a code of black magic practice. To avoid this, three things are, I believe, necessary.

a) The knowledge, experience and wisdom of practitioners of geomechanics must be continuously developed and improved,

b) The appropriateness of testing standards must be kept under review and the standards brought up to date as necessary. Competent practitioners should always be willing to make their skills available for this process, and

c) Blind use of standards and TELARC must be resisted and the cost of irrelevant testing, test procedures and precision made plain to those paying for them.

Yours faithfully,
J.H.H. Galloway.

Sir,

I should like to add my thoughts to those of Dr Burland on the subject of research (Geomechanics News 18, pp 9-11). He points out that 'research' is an activity which, quite rightly, engages the attention of academics (I hasten to add, as one who works in a university tower, that it is anything but ivory!). I agree with Dr Burland that research is an attitude of mind and one that is central to successful endeavour in geotechnical engineering. I want to ask: Is it an attitude that is well developed in this country? It seems that I am not alone in asking this question. The leading article in NZ Engineering for September 1979 raises, on a broader front, some of the points I make below.

Dr Burland's point that each new job involves us in the difficult task of discovering the essential features of a site and then making a suitable idealisation of this for design purposes is generally accepted. However, how many of us can honestly say that we follow the second research phase of the work through? As Burland says, the only way we can know whether our initial idea about the ground at a particular site was reasonable is to observe carefully what happens during and after construction. Thirdly, these findings need to be shared with the rest of the profession so that collectively we have a greater pool of experience to draw upon. The most convenient way of doing this is by publication. Too often such monitoring is regarded as an unnecessary luxury. Without it we condemn ourselves to a mindless repetition of textbook assumptions. In New Zealand we pride ourselves on our unique geology and unusual materials. Thus no one else can do this monitoring work for us. The work published in overseas literature is interesting and may be a source of many useful ideas, but is it relevant here?

Part of the difficulty is perhaps related to the New Zealand temperament. We are, as a nation, more interested in doing than reflecting. This carries over into our approach to engineering. The exacting task of taking careful observations, reflecting on them and reporting them seems unattractive in comparison with the excitement of getting the next project underway. It is of interest to remark that the best monitoring work seems to come from agencies sponsored, at least in part, from central government. I have in mind the Geotechnics Division of the Building Research Station in London, of which Dr Burland is the leader, the Norwegian Geotechnical Institute in Oslo and, closer to home, the Applied Geomechanics Division of the CSIRO in Melbourne. The quality of the work done by these groups points out the advantage of having a small group of people dedicated to taking these measurements.

As organiser for the technical programme for next year's Australia - New Zealand Geomechanics Conference I have been disappointed that there are no New Zealand papers on underground works. We are, and have been, heavily involved in this aspect of geotechnical engineering. The Terrace and Kaimai Tunnels have recently been completed. Both have been given symposium treatment in NZ Engineering and Transactions. Although useful, these papers are of a general "Here's how we built it" type. What is immensely more useful is the "Here's what happened" paper supported by properly documented measurements. In the case of the Terrace Tunnel many very good measurements were made of the ground deformations during the tunnelling. A published record of these seems necessary in terms of Burland's definition of research. More so because the type of ground the tunnel was driven through, the transition from rock to residual soil with the added complication of faulting, is just the type of non-classical situation that Dr Burland mentions specifically.

Rather than criticising the MWD I am suggesting that they have not been making enough of the major achievements that these tunnels are. My point is that the information is available and that the civil engineering profession is better off if it is published.

What, then, do we see from the private sector? I find, since moving to Auckland two years ago, that life here carries with it a continuous stream of propaganda about the merits of private enterprise. When one asks about monitoring work in the private sector the usual defence is that the client is not prepared to pay for such frills.

In summary, my point is this: The geotechnical engineering profession could probably survive without much of the purely academic research that is published. Without the type of research discussed in this letter and Dr Burland's article, all we can look forward to is stagnation. Because of the special conditions we have in this country we must engage in this work ourselves. We cannot expect to rely on results gained by others with more resources for research.

Yours faithfully,
M.J. Pender.

LOCAL GROUP ACTIVITIES

1. WELLINGTON GROUP

1.1 Excavation and Ground Anchoring at the Wellington Chief Post Office Site:

The meeting, held on Tuesday 12 June 1979, was addressed by four speakers - Mr D. Robertson (Fletcher Construction Company Limited), Mr P.J. Lemmon (Lemmon Piling and Drilling Limited) Mr M.C. Jones (Stresscrete Wellington Limited) and Mr E. Blaikie (MWD, Wellington District Office). 25 people attended.

Mr D. Robertson outlined the contract for the recently completed 9 m deep, 90 m x 50 m excavation in a sub-horizontal sequence of 3-4 m of Fill ("greywacke" debris), 1 m of Beach Gravels (sand, shells, gravels with some clay) and underlying Alluvium (moderately to highly weathered "greywacke" sandy gravel, $\phi = 32^o - 38^o$, C = 0-100 kPa). In situ moderately-highly weathered "greywacke" is encountered 0-13 m below invert level, which is 7 m below MSL. The excavation was vertically supported by 2 - 254 channels placed back to back to act as soldier piles at 1.8 m centres. The piles were taken to 5 m below invert and set in 30 MN concrete. A 150 mm thick lining slab with a drainage sump was placed at invert at completion of the excavation.

Mr P. Lemmon first outlined the installation of the soldier piles with Watson Caisson equipment, followed by the drilling of the holes for the ground anchors and their placement. The three rows of 150 mm diameter anchor holes were drilled from each of the soldier piles at depths of 2 m, 5 m and 7.5 m with an inclination of 15^o to the horizontal. The 17 m long upper row was drilled using a Hangmaster 36-C rig with water flushing through Fill and Beach Gravel and into Alluvium, cased for the full length and pressure grouted using 0.36 water cement prior to withdrawal of the casing. The 15 m middle row was drilled with a full flight auger through Alluvium, and the 15 m lower row with air or water, flushing through Alluvium occasionally reaching "greywacke".

Mr M. Jones outlined the supply and stressing of the 384 ground anchors. Each anchor consisted of 4x12.5 mm diam. super strands capable of being stressed to a total design load of 590 kN (80% ultimate tensile strength of anchor). Stressing was done using a Macolloy 100 ton centre pull jack (acting on all strands simultaneously) with the upper row receiving a load of 400 kN and the lower two rows 500 kN at increments of 0.15, 0.4, 0.8, 1.0, 1.3 and 1.3 the anchoring off load. One anchor in ten was subjected to further acceptance stressing, and all anchors were checked for loss of load after 7 days before cropping off and protection of the strands. The design loads and bond lengths of the anchors were verified prior to full scale installation using 12 strand test anchors at each level.

Mr E. Blaikie described the stability analysis of the excavation and the monitoring of the deflections of the anchors, and excavation during and after their installation. Stress distributions were considered for varying soil springs, and distributions of bond "shear strengths" of the grout column. The load fall off tested after one week was much higher in the upper row anchors (up to 12%) and deflections in the tops of the soldier piles were up to 30 mm away from the excavation after stressing. Concern for possible corrosion of the anchors was expressed due to the emergence of water with a pH of 12 from the ends of the anchors.

The overall operation was successful with no problems due to anchor failure. There was a high degree of co-operation between the interested parties, e.g. the driller placing the anchors. The meeting also provoked further contact between parties interested in stressed ground anchors in soils, a field in which there is little literature.

S.A.L. Read.

1.2 The Itaipu Hydro-Electric Scheme:

On Tuesday, 14 August 1979, Mr S.A.L. Read of the DSIR Geological Survey, gave a most interesting illustrated talk on the Itaipu Hydro-Electric Scheme, where he worked as a site engineering geologist in 1977.

Itaipu, the world's largest hydro-electric scheme, is a bi-national scheme shared by Brazil and Paraguay, and is situated on the Parana River where it forms the border between the two countries. The scheme, which is another stage in the quest for hydro-electric energy in Brazil, has a present generating capacity of 18,500 MW, to be increased by a further 10,500 MW by 1980. The largest portion of this power is developed in the Parana River and its tributaries. The river, which is one of the seven largest rivers in the world, is 4,000 km in length and has a drainage area of 3 million square kms. At the Itaipu site the river has an average flow of 8,500 m^3/sec.

Construction activities commenced in October 1975, and the first power is scheduled to be generated in early 1983. The dam has a maximum structural height of 176 m and crest length of 8.5 km. Generating capacity will be 12,600 MW from 18 turbines to produce 70 billion kWh/year. The reservoir will be 200 km long, flooding 600 km of Paraguay and 800 km of Brazil. The total price is presently put at US $6,000 million at time of completion. Brazil, which is financing the scheme, will use nearly all the power generated. Paraguay will receive an expected revenue approximately tripling its earnings from traditional exports in 1972, once their part of the construction costs has been taken in generated power by Brazil.

The Itaipu site geology is a sequence of basalt lava flows and inflow breccias dipping at 2^o upstream. Four flows and three interflow breccias form the foundations of the important structures, and additional flows exist to well below the influence zone of the structures. The flows, which vary from 10 m to 60 m in thickness, are dominated by high strength dense basalt. Jointing is moderately to very widely spaced and dominated by subvertical cooling joints. The tops of the flows are formed by 0 - 10 m thick zones of moderate strength vesicular and amygdaloidal, which contain few joints. The interflow breccias which vary from 0 - 20 m, are free from joints. They are variable in geometry and geomechanical properties, ranging from high strength baked sandstone to low strength cavernous breccia. Overburden, consisting of red sublateritic soil and weathered basalt or breccia, varies from 5 - 20 m in thickness. The contact with the bedrock is usually reached abruptly. No major faulting has been recognised in the site area.

Testing of the foundations has concentrated on obtaining information of bulk deformation of intact rock properties using large flatjacks, plate bearing, borehole deformation and direct shear methods. In situ stresses were measured by the overcoring method using the Strain Tensor Gauge. Water pressure testing used 0.1 of the lugeon unit, thus reducing the pressure used in testing. A large diameter shaft has been dug with accompanying drivers to examine possible problem areas - contacts between flows and breccias, structural discontinuities within flows, water loss zones in flows and breccias.

Diversion Canal

In the diversion canal excavation, the slopes in rock are 70 m high consisting of 1H:4V with 4 m benches each 20 m. In the vicinity of the diversion structure, the slopes steepen to 1H:20V with a height of 75 m on one side of the excavation and 50 m on the other. Untensioned rock dowels of reinforcing steel together with associated drainage holes, provide the slope reinforcement of those steeper slopes. Over half the excavation was completed before reinforcement commenced, which allowed destressing of subvertical columnar rock prisms to occur. These caused problems on final slopes which were accentuated when blasting techniques used excessive particle velocities, causing further damage.

Discontinuities in the dense basalt in one of the flows originate as a result of mineral differentiation within the flow during cooling. Glass and other minerals rich in expanding clay (nontronite and other montmorillonites with subsequent weathering) are concentrated in a "flow shear zone" ± 3 m below the top of the dense basalt of the flow. The feature, which is a continuous weathered and montmorillonitic clay filled zone, was mapped and geometry studied prior to driving some short tunnels for in situ shear testing.

Rockfill Dam

The rockfill dam is the principal wingdam and varies in height from 55 m - 75 m. The core was borrowed from completely weathered breccia where this formed the soil overburden. The weathered breccia was preferred to weathered basalt for its slight sand content, which reduced the risk of dispersive action in soils that contain some montmorillonite, and also assisted in filter design. The rockfill was obtained (together with concrete aggregate) from the diversion canal excavation. Under the core there was no cut off trench but a shallow grout curtain. The foundation conditions were good, and the only real concern was loss of material by internal erosion through "exfoliation" type joints. Concrete patches were placed on the core foundations where necessary to eliminate sharp angles and prevent erosion. The whole core was also slush grouted immediately prior to core placement. The only interesting area of the foundation geologically, was a zone of breccia running across the dam. It was formed between two lava tongues which almost coelesced, and at one stage was thought to be a fault zone.

Because it is so large and shared between two countries of different cultures, languages and attitudes to progress, the Itaipu Project has a complex, multi-facited administration and organisation. Brazil is a very large country, bent on progress and with a good deal of hydro-electric construction experience, while Paraguay is small (3 million population) and without similar drive and experience. The labour force which is large even by Brazilian standards (with a peak of 15,000),and machinery, etc., is shared by consortiums of Brazilian and Paraguayan contractors. The construction materials (e.g. for the principal dam; 1500 tons of cement/day at peak) are obtained from supply points further than 300 km from the site. Large additional costs will also result from Brazil having 110 volt/60 cycle current and Paraguay 220 volt/50 cycle current, thus probably necessitating that the turbines be dual frequency.

T.J. Kayes

2. AUCKLAND GROUP

The major activity for this year was the presentation of the 3rd New Zealand Geomechanics Lecture. The lecture was given by Dr R.D. Northey of the D.S.I.R. Soil Bureau, and was titled: The Acceptability of Geotechnical Risk (see last edition of Geomechanics News).

At a mid year meeting in June, the subject was New Zealand Involvement in Geotechnical Work Overseas. Mr J.P. Blakely of Beca, Carter, Hollings and Ferner and Dr L.D. Wesley of Tonkin and Taylor gave accounts of some of the experiences of their firms when engaged in work in Southeast Asia and the Middle East. Mr Blakely described work on two large projects in Indonesia, one a natural gas development in North Sumatra and the other a nickel mining operation in Sulawesi (Celebes). Dr Wesley described site investigation work for a cool store in Bahrain, a geothermal project in Indonesia and an earth dam in Malaysia.

The third meeting planned for the year is a joint one with the Auckland Structural Group on the Design of Slabs on Grade. This is a topic which receives little attention in the literature, and which is possibly treated too lightly by many engineers.

L.D.Wesley

NEWS FROM THE MANAGEMENT SECRETARY

1. 1980 NZIE Conference

The programme for the 1980 NZIE Conference to be held in Dunedin from 11-14 February has now been finalised. The Geomechanics Society has taken one full day with the following papers scheduled:

Wednesday 13 February

9.00 - 10.15 a.m.

Workshop session "Recent Developments in Pavement Evaluation for the Practitioner"

N.G. Major

10.45 - 12.00 noon

"Control of Pile Driving Operation using the Pile Driving Analyser"
W.A. Trow

2.00 - 3.15 p.m.

"Urban Slope Instability" J.G. Hawley

3.45 - 5.00 p.m.

"Huntly Power Station Cooling Water Intake Structure"
D.K. Taylor, G.A. Pickens, L.D. Wesley

5.00 - 5.30 p.m.

NZ Geomechanics Society Annual General Meeting

2. New Members

The following applications for membership have been received since the last issue of Geomechanics News:

G.H. Rowe	Wellington
H.R. Turnbull	Lower Hutt
R.W.J. Fookes	Tawa
P. Buol	Tauranga
Taupo County Council	Taupo
W.D. Castle	Wellington

3. Forthcoming Conferences

7-11 April 1980

"International Symposium on Landslides: New Delhi, India

14-18 April 1980

"Symposium on Hydrological Forecasting: Oxford, U.K.

14-17 April 1980

"Ground Movements and Structures" Cardiff

22-24 April 1980

"International Conference on Compaction " Paris

7-9 May 1980

"International Conference on Structural Foundations on Rock" Sydney

12-16 May 1980

"3rd A-NZ Geomechanics Conference" Wellington, N.Z.

19-23 May 1980

"The Safety of Underground Works" Belgium

23-27 June 1980

"Rockstone 80: International Symposium on Subsurface Space" Stockholm

7-17 July 1980

"26th International Geological Congress" Paris

15-19 June 1981

"10th ISSMFE Conference" Stockholm

Summaries of papers for this conference should be submitted to the Management Secretary, NZ Geomechanics Society by 1 April 1980. Papers in final form are due by 1 August 1980.

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

I.M. Parton
Management Secretary

ENGINEERING GEOLOGY

D.H. Bell

Abridged notes for a seminar entitled "Civil Engineering - A Survey of New Developments" held at the University of Canterbury from 11 to 13 October 1978.

1. Introduction

Engineering geology is the application of the geological sciences to the location, design, construction, operation and maintenance of engineering works. Its prime concerns are with *site foundations* (the "geological model" is quantified by soil or rock mechanics techniques) and with *active processes* (earthquakes; volcanic eruptions; landslides; etc.) which may represent hazards to the structure during its design life. The geological *approach* to a particular problem tends to be intuitive, is often qualitative, and stresses the geological complexities of the real site: in contrast, the engineer relies on numerical data which must often be applied to gross oversimplifications of the actual site conditions. The successful marrying of these two approaches to *foundation engineering*, and the increasing recognition of the need for a rational geological basis to engineering design, constitute the major advances in *Geomechanics* in recent years. The dependence of rock and soil mechanics on realistic engineering geology is now well established, but there remain far too many practical engineering situations where there has either been inadequate geological input during design and construction, or the relevance of engineering geology has not even been recognised.

2. Rock Materials

2.1 Some Definitions

ROCK: In engineering usage, rocks are "hard and rigid", require blasting or some equivalent method during excavation, and are not significantly affected by immersion in water. On the other hand, geological usage regards all materials below the depth of modern weathering (see below) as rock irrespective of their degree of consolidation or cementation.

SOIL: In engineering usage, soils are naturally occurring loose or soft deposits which can be excavated by "normal" earthmoving equipment, and which either disintegrate or become remouldable on immersion in water. In geological usage, soils have formed at the surface of the earth in response to physical, chemical and biological processes, and represent a weathering or alteration feature superimposed on engineering soils and/or rocks.

The first problem to which *engineering geology* addresses itself is thus one of communication between engineer and geologist. There is an important distinction between rock material and rock mass:-

ROCK MATERIAL: Intact material composed of mineral grains and pore space (air and/or water-filled) and comprising the "solid" portion of the in situ rock mass.

ROCK MASS: Includes both the rock material and the defects (joints, faults, etc.) which separate the "rock blocks".

As will be shown later, it is the defects which largely control the mechanical response of the rock mass to engineering loads, and hence laboratory testing of rock material samples "abstracted" from the engineering site may be of very limited practical value for design purposes. It is the recognition of this fact which constitutes the single most important advance in engineering geology, as it marks the "birth" of rock mechanics as a separate discipline.

2.2 Physical Properties

In 1972 the "Commission on Standardization of Laboratory and Field Tests" of the *International Society for Rock Mechanics (ISRM)* published Document No. 2, which suggested methods for the determination of water content, porosity, density, swelling, and slake-durability index properties of *rock materials*. Basic tests satisfactorily characterise rock material properties, as follows:-

1. Saturation moisture content (i_s), which is a measure of porosity and hence of void space within the material.
2. Dry apparent specific gravity (G_d), which is a measure of mineral grain density or specific gravity.
3. Uniaxial swelling strain (ε_S), which quantifies the bond strength between the mineral grains constituting the rock material.

The above laboratory tests are relatively easy to perform (Duncan,1968) except in the case where the rock material disintegrates in water (i.e., the mineral grains are only weakly bonded, and what appears to be a "hard" rock when dry in fact behaves essentially as a soil). The so-called "soft rocks" (sands and muds) widely developed throughout New Zealand come within this category, and are very clearly problem "rocks" for the practising engineer.

2.3 Mechanical Properties

ISRM Document No. 1 was also published in 1972, and describes methods for determining uniaxial compressive strength and the point load strength index of *rock materials*. The uniaxial compressive strength test, in addition to giving a basic measure of rock strength, can also provide elastic moduli such as Young's Modulus (E) and Poisson's Ratio (υ) if the necessary strain measurements are made during the test. Research into the stress-strain behaviour of rock materials has shown, however, that there exists a large class of rock materials which display semi-elastic or even non-elastic response to stress. Studies of the creep behaviour of such materials thus become necessary for design purposes, and much research has been generated in this field, especially that of salt mining (Baar, 1977).

The point load strength index test is designed as a field method for the rapid estimation of rock strength, using either drill core specimens or irregular lumps of rock material (Broch and Franklin, 1972). A concentrated load is applied using conical platens which are operated by an hydraulic ram, and tensile failure is induced in the specimen. Although an empirical relationship has been found between the point load strength index ($I_s(50)$) and uniaxial compressive strength (σ_{co}), the theoretical basis for this relationship is suspect, and the point load strength is best regarded as another measure of rock strength. The point load strength test can also be considered as a modification of the Brazilian (or line-load) test, which has been in use for many years (Jaeger, 1972).

Conventional triaxial testing of cylindrical drill cores under differing confining stresses, and the subsequent interpretation of the Mohr circle plots in terms of the parameters C (cohesion) and Ø (angle of frictional resistance) is now widely applied in roch mechanics (as it has been for many years in soil mechanics). Likewise, rock behaviour in terms of effective stress parameters has been investigated as well as for total stress, and the effects of water on rock strength and elasticity are well documented (Attewell and Farmer, 1975). However, as noted earlier, it is the presence of defects within the *rock mass* which frequently determine stress response, and it is thus the mechanical properties of the defects themselves which assume dominance.

3. Rock Masses

3.1 Rock Defect Studies

3.1.1 Geological and Geomechanical Classifications

In general the shear strength which can be mobilised by fresh, intact rock material is considerably greater than that available from the rock defect surfaces themselves. In turn, greater shear strength can be mobilised on rough defect surfaces than on smooth ("pre-sheared") surfaces, due to the interlocking of irregularities and their dilation against the applied normal (effective) stress. An analogy can thus be drawn with peak and residual shear strength concepts for clay soils, and from the geomechanical viewpoint it is clearly critical to establish whether $Ø_r$ or some greater value can be mobilised on a particular defect surface. In terms of geomechanics, defects can be classified therefore as either (a) contacting, in which case the roughness of the defect and the compressive strength of the walls themselves determines the available shear strength; or (b) non-contacting, in which case the defect is "filled" (often by clay minerals) and the physical, mechanical and mineralogical properties of the materials separating the defect walls control ultimate stability.

Defects may also be classified in terms of geological origin, as follows:-

A. Primary (examples include sedimentary bedding planes and metamorphic foliation, either cleavage or schistosity);

B. Secondary i. joints a. tensile) weathered/unweathered
b. unloading)
c. cooling)
d. shear) infilled/open
ii. faults and shear zones (usually filled with clay seams)

In terms of defect origin, it may also be important to distinguish between clay seams which have developed by *in situ* alteration of crushed material, and those resulting from infilling by percolating groundwaters. Likewise, defects provide the principal subsurface "channels" for water migration through (and alteration of) rock masses: secondary (fracture) permeability is commonly orders of magnitude greater than primary (rock material) permeability, and weathering proceeds from the defects into intact material. This secondary weathering of rock defects thus lowers still further the frictional resistance which can be mobilised, and influences *rock mass strength* accordingly.

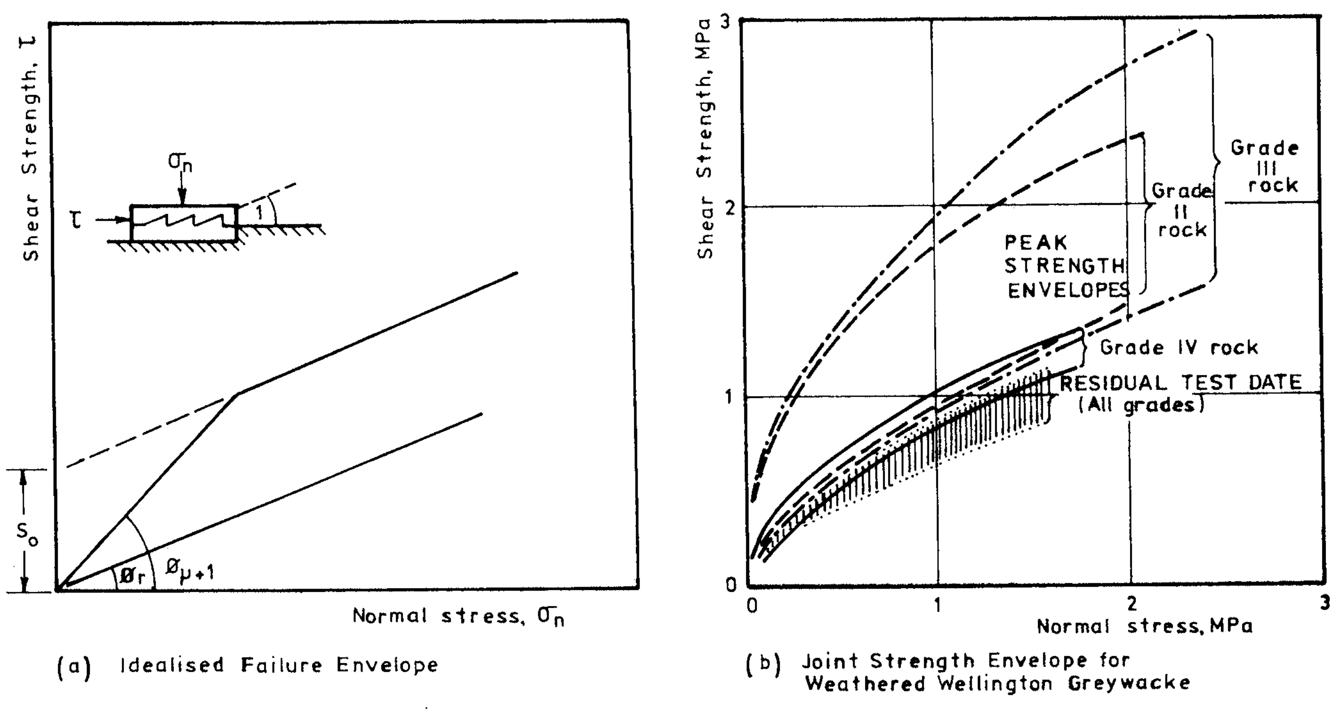
3.1.2 Strength Parameters

ISRM Committee on Field Tests Document No. 1 (1974) outlines methods for both the in situ and laboratory determination of the direct shear strength of rock defects. Laboratory studies generally use test specimens of maximum dimension 150 mm, set in quicksetting plaster and loaded by hydraulic rams mounted vertically (to provide the normal stress) and horizontally (to shear the specimen). Both peak and residual shear strengths are determined by the method, and theoretical and actual envelopes are shown in Fig.1. Conventional interpretation of shear strength data in terms of Fig.1 is as follows:-

1. At normal loads the defect irregularities remain intact and the shear strength may be written as

$$\tau = \sigma_n \tan(\phi_u + i)$$

where ϕ_u is the frictional angle on the planar surface of the "teeth" and i is angle of inclination of the "teeth" or irregularities.



(a) Idealised Failure Envelope

(b) Joint Strength Envelope for Weathered Wellington Greywacke

Fig. 1: JOINT SHEAR STRENGTHS (after Martin & Millar, 1974)

2. At high normal loads the irregularities shear off and the shear strength can be written in terms of the conventional Coulomb shear parameters as

$$\tau = S_o + \sigma_n \tan\phi_o$$

An empirical shear strength criterion has been developed in recent years (Barton, 1973; Barton and Choubey, 1977) in an attempt to more realistically model rock defect behaviour. Barton's shear strength criterion is written as

$$\tau = \sigma_n' \tan \left[JRC \log_{10} \left(\frac{JCS}{\sigma_n'} \right) + \phi_b \right]$$

where τ = peak shear strength

σ_n' = effective normal stress

JRC = joint roughness coefficient

JCS = joint wall compressive strength

ϕ_b = basic friction angle (from residual shear tests)

It thus relates the peak shear strength which can be mobilised on unfilled defects to the roughness of the defect surface (which controls the actual contact area of intact rock material across the defect) and to the degree of weathering in the "skin" adjacent to the defect surface. As such, this represents a significant new advance in the field of rock mechanics, although the practical relevance of the method remains to be demonstrated.

For rock defects on which only residual shear strength can be mobilised, the conventional Coulomb parameters are usually adopted, with C = 0 and ϕ_r a function of mineralogy and moisture content. For remoulded clay "gouge", friction angles commonly vary between 10 and 20^o: a marked lowering of the coefficient of static (sliding) friction occurs from the air-dried to the saturated state (Horn and Deere, 1962), at least for the sheet silicate minerals. Likewise, effective stress parameters are more appropriate than total stress parameters in the evaluation of the frictional shear strength that can be mobilised on a potential sliding surface.

3.1.3 Defect Surveys

In summary, the following defect properties require quantitative analysis in assessing rock mass stability (after Piteau, 1971):-

1. the type of geological structure (shear or tensional joint; fault; etc.)
2. its position in space (coordinates and elevation)
3. its orientation (with respect to the coordinate system, usually true north)
4. defect intensity (spacing, measured in frequency per metre)
5. defect wall hardness (intact rock strength, as modified by "skin" weathering)
6. asperities (roughness and waviness of the defect surface)
7. continuity of defect sets (failure may be along continuous defects, or partly through intact rock and partly along discontinuous defects)

8. distribution thickness and mechanical properties of gouge or defect infilling material
9. the presence or absence of water on the defect plane.

3.1.4 Stability Analysis

Analysis of rock mass stability (for example, in batter design) involves the following stages:-

1. Collection of geological (defect) data; identification of structural domains; and the determination of a rock mass model.
2. Preliminary analysis of the major defect sets to determine if the slope is kinematically stable or unstable (i.e. to decide if failure is possible).
3. Laboratory and/or field testing of the controlling defect surfaces to determine strength parameters that can be applied in mathematical analysis.
4. Mechanical stability analysis of the "assumed" geotechnical model to determine a factor of safety against failure.
5. Design and/or remedial measures for the problem at hand can then be realistically implemented.

For both kinematic analysis and force vector analysis, widespread use is made of stereographic projection, in which planes in the lower hemisphere are projected onto the horizontal diametral plane of a sphere. This technique is widely used in structural geology. For mechanical stability analysis, a combination of stereographic projection and graphical vector addition can be employed. The underlying assumption in all such analyses is that a realistic geological model has been selected, and that suitable mechanical parameters have been determined or adopted.

3.2 In Situ Rock Mass Properties

Because of the dependence of the stress response of rock masses on the geological defects present, it is clear that laboratory tests on *rock material* specimens cannot realistically model likely in situ behaviour. The existence of high horizontal in situ stresses has also been known for some time, and these obviously will influence rock mass behaviour in large underground openings (such as powerhouses or mines). There have been many recent developments in the field of in situ rock mass testing, and some of the more important are:-

1. Borehole Strain Instruments: These techniques involve stress-relief (by drilling a borehole) and measurement of the resultant strain. Instruments include borehole deformation meters, borehole inclusion stressmeters, and borehole strain gauge devices.

2. Hydraulic Pressure Cell Measurements: "Jacking" techniques have been in use for some years, and the basic technique involves cutting a slot and then applying an hydraulic pressure to cancel the resultant strain. The data obtained can be used to compute elastic parameters for the rock mass at the site tested.

3. Geophysical Methods: The velocity of the longitudinal wave (V_p) is a function of E and υ, and hence theoretically dynamic moduli can be determined using seismic techniques. However, no material is perfectly elastic, and hence V_p is stress dependent. Nevertheless,

moduli such as E_{dyn} are readily determined, and are often used for design purposes.

4. <u>Static Mechanical Tests</u>: In these tests, loads are applied in large underground excavations by hydraulic rams with cross-sectional areas of about $1m^2$. Such tests are considerably more expensive than conventional laboratory tests, but provide much more realistic design parameters.

It is clear that many of the above in situ tests are only really appropriate on large projects, such as large dams or underground powerhouses. Nevertheless, the limitations of tests carried out on small samples "abstracted" from the in situ environment dictate such techniques, and they will be used increasingly in the future.

3.3 <u>Engineering Implications</u>

The fundamental properties of rock masses can now be summarised as follows (after Piteau, 1971):-

1. Failure will tend to be confined to defects, the strength of which will be significantly less than that of intact rock material.
2. Strength and deformational properties are directional, and the rock mass is therefore <u>anisotropic</u> in its stress response.
3. The physical and lithological (composition) properties of rock materials are generally variable (i.e. the rock mass is not homogeneous).
4. A rock mass is analagous to a partitioned solid body composed of individual blocks, and is thus a <u>discontinuous medium</u>.

The implications for rock mass engineering are clearly significant, and some of the more important are:-

A. <u>Rock Slope Engineering</u>: By making certain assumptions about significant rock defect sets (in controlling stability of a particular slope) and about the validity of strength parameters obtained from small-scale testing, it has been possible to develop mathematical analysis techniques to a high degree of sophistication. Hoek and Bray (1977) give numerous charts on which to base the design of highway batters or open-pit mine slopes, providing that the slope in question can be treated by a plane, wedge, toppling or circular failure mode. It is, however, the weakness of the technique that in assuming a particular failure mode, or in ascribing particular C and Ø values to a single defect surface, the *real* geological complexities of the site may not have been fully taken into account.

Although there must always be reservations about slope stability calculations which produce a factor of safety close to 1.0, and a probability approach to stability analysis may yet come into wide acceptance, there can be no doubt that the geomechanics approach in current use is considerably superior to an empirical basis of design. In designing any cut batter in "hard" rock, account must be taken of the nature and orientation of the geological defects; of any kinematically unstable defect combinations; of shear strength parameters appropriate to the various defect surfaces; of groundwater conditions and resultant pressures; and of any long-term deterioration of the slope which may result from a particular design. An empirical approach to the design of batters which does not take account of rock mass defects (especially jointing and any primary rock fabric) is an invitation to slope failure, or to excessive costs during either construction or subsequent remedial measures.

B. Underground Excavations: The determination of rock mass properties is an essential pre-requisite in the design and construction of large underground openings, such as those for mine workings and hydro-electric powerhouses. Likewise, tunnelling for transportation routes (railway and highway, in particular) requires a clear knowledge of both engineering geology and rock mechanics to ensure adequate safety and optimum economy. Overseas experience in this field is considerable (see, for example, Obert and Duvall, 1967; C. Jaeger, 1972; J.C. Jaeger and Cook, 1976), although there is also considerable expertise available within New Zealand, both in hard and soft rock tunnelling (see, for example, Prebble, 1977; Pender, 1977; Millar, 1977). By careful application of rock defect studies, ground support requirements can be optimised, and some of the available methods include rockbolting, anchored cables, and shotcrete lining; this last technique forms the basis of the so-called "Austrian" method of tunnelling, in which rockbolts and shotcrete are applied to the underground cavity immediately after blasting (C. Jaeger, 1972).

C. Dam Foundations: The satisfactory analysis of dam foundations requires both engineering geology and rock mechanics investigations, and again such methods are well documented (see, for example, Attewell and Farmer, 1975). The basis techniques of rock mechanics, and development of such mathematical theory as finite element analysis, provide new methods in the field of dam foundations (and, of course, of underground openings). In addition, analysis of some of the relatively recent major dam failures, such as Malpasset, has provided an impetus to develop new methods of foundation investigation (see, for example, Stapledon, 1976).

4. Site Investigations

4.1 Engineering Geology Mapping

One of the major developments in engineering geology in recent years has been the standardisation of methods for the presentation of data on maps and plans. Engineering geology mapping may be carried out at the reconnaissance, design, and/or construction stages of a particular project, providing essential links between geotechnical investigations at specific locations. Differing map scales are selected depending on the nature of the project and the stage of investigation, and these may range from 1:50 for underground detail to 1:10,000 for the slopes of a reservoir. The need for engineering geology mapping at the reconnaissance stage (to predict foundation conditions), at the design stage (to facilitate and interpret geotechnical data), and at the construction stage (to record actual foundation conditions), cannot be too highly stressed. There is no doubt that many smaller engineering projects would benefit considerably from engineering geology mapping such as is routinely carried out at major sites, and reconnaissance engineering geology investigations (including mapping) may save considerable expenditure on unnecessary site investigations.

4.2 Geophysical Methods

Seismic refraction and electrical resistivity surveys continue to be major tools for engineering site investigations, and the only significant developments have been in improved instrumentation (giving better resolution of data). However, there have been considerable advances in recent years in downhole logging techniques, principally in the oil and mining exploration industries. Borehole geophysical logs can also be of considerable use in engineering investigations, particularly in assisting with correlations between boreholes.

4.3 Drilling and Core Logging

Diamond drilling with maximum core recovery still provides the major information on foundation conditions for the design engineer. It is from the cores that the site geology is deduced, and samples are made available for laboratory determination of geotechnical parameters. However, drilling from which only a fraction of the total depth is recovered as core may be of very limited use, since the nature of the material *not* recovered is unknown. One of the more important drilling developments has been the triple tube method for the recovery of soft or erodible cores, such as fault zone clay "gouge". Yet another development for site investigations in soft sediments (such as glacial till) is the large diameter hole (c. 1m), which allows visual inspection of the subsurface and permits large-scale sampling or in situ testing.

One interesting development (Deere, 1968) in core logging is the RQD or *Rock Quality Designation.* This is a measure of the length of core in excess of 10 cm expressed as a percentage of the total core run, and is regarded by some as a more sensitive indicator of general rock quality than is the gross core recovery percentage.

5. Active Geological Processes

A survey of new developments in engineering geology would not be complete without at least brief discussion of research trends into active geologic (and geomorphic) processes. Active geological processes, in the context of which many civil engineering structures are built, include landslides and associated erosion processes; stream transport of sediment; coastal erosion and wave action in general; earthquakes and tsunamis; and volcanic eruption and associated problems. For many engineering structures active geological processes constitute *hazards,* and it is in the context of geological hazards that many recent studies have been undertaken (see, for example, Bull. I.A.E.G., No.14, 1976). Likewise, environmental geology has developed as a science in the similar context of land use planning and resource management, but probably the most important trend has been the "quantification" of active processes. No longer are studies of active processes purely descriptive (as in classical geomorphology), and research is carried out by many "non-geological" organisations (see Bolt et al., 1975).

The following examples serve to indicate recent trends in research into active geological processes:-

1. Mass Movement and Erosion Processes: There are many recently published studies of landslides which have affected the siting of engineering structures, or necessitated remedial measures on a massive scale (see, for example, Coates, 1977). The most spectacular landslide of recent times is still the 1963 Vajont rockslide of some 250,000,000 m^3 which entered the hydro reservoir in less than one minute, sending a 30 m wave over the dam crest and into the valley downstream (Stapledon, 1976; Jaeger, 1972). In New Zealand considerable research is being undertaken into active landslides

(see, for example, N.Z. Geomechanics Society, 1974), where problems range from deep-seated rockslide problems through creeping earthflows in bentonitic materials to shallow regolith (weathered mantle) failures triggered by high intensity rainstorms (Bell, 1976). The associated problem of erosion is also being carefully studied in a variety of engineering and envonmental contexts, such as that studied in a variety of engineering and environmental contexts, such as that of forest removal (O'Loughlin and Pearce, 1976) and mountain land management (for example, Hayward and Sutherland, 1974). A major research effort has recently been mounted by Water and Soil Division, Ministry of Works and Development, into catchment erosion problems and associated downstream sediment effects, and in fact soil conservation as such has been a major concern of Catchment Boards in New Zealand for many years.

2. Earth Deformation and Earthquake Prediction: Considerable advances have been made in recent years (particularly in China, Japan, USSR and USA) in earthquake prediction, and techniques employed include geophysical studies, precise ground surveys, and well monitoring adjacent to active faults. In the period 1926-1950, loss of life from earthquakes was about 350,000 and damage estimated at $10,000,000,000 (Bolt et al., 1975), and there has certainly not been any decrease in seismicity around the world since that time. The possibilities of earthquake prediction and control have thus given considerable impetus to research, and in New Zealand a number of earth deformation networks have been (or are still being) installed across known active faults (Lensen, 1976).

3. Volcanic Eruption and Prediction: Hazards presented by episodic eruptions in volcanically active parts of the world are well documented (see, for example, Barberi and Gasparini, 1976), and as with earthquakes major research programmes are under way into volcanic prediction. An associated hazard of the volcanic environment is that of lahars (Neall, 1976), and detection schemes have been devised to protect engineering works in the Central North Island (e.g. parts of the Tongariro Power Scheme).

No doubt recent developments into other active processes will be presented during the Seminar (e.g. in coastal erosion processes and stream sediment transport).

6. Synthesis

1. Engineering geology is an integral part of *Geomechanics*, and as such is of fundamental importance in *foundation engineering:* adequate geological input is essential at the reconnaissance, design, construction, and post-construction stages of civil engineering projects.

2. Developments in the field of rock mechanics, in particular defect studies and in situ rock mass investigations, constitute the major recent advances in engineering geology practice: these techniques have application in rock slope engineering, underground excavations, and dam foundation investigations.

3. The refinement of engineering geology mapping and logging techniques is of immediate relevance to engineering design and construction: without satisfactory communication between engineer and geologist sound geomechanics *practice* is not possible.

4. There have been many recent advances in the study of active geological processes, such as landslides, floods, coastal erosion, earthquakes and volcanic eruptions not all such developments come within the scope of *Engineering Geology*, but their relevance is considerable because these active processes constitute *hazards* to engineering structures, transportation routes and population centres.

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REPORT ON 1979 I.S.R.M. MEETINGS, MONTREUX, SWITZERLAND

1. I.S.R.M. Council Meeting

1st Session, Sunday, 2 September 1979

When the roll call was taken, New Zealand, who had not been represented at the Council for several years, was very warmly welcomed.

The Secretary General, Dr Silverio, presented the report of the Secretariat - this will be available from our Rock Mechanics Vice-Chairman, who should be congratulated on the fact that New Zealand was named as one of the very few countries to have sent news of our activities.

The six regional Vice-Presidents presented reports (some very lengthy, Australasia quite brief).

ISRM Commission Chairman presented reports. Lombardi's proposed commission "Dilatation of rocks due to failure around tunnels and other large underground openings," in which NZ has expressed interest, is still not off the ground. The Commission on Swelling Rocks - headed by W.E. Bamford, Australia, reported in some detail (again available from V.C. when it arrives).

Co-ordinators reported on co-operation with other bodies: Permanent Co-ordinating Secretariat, International Bureau of Rock Mechanics (which is the Eastern European group; I seconded a motion for formal collaboration with them), International Commission on Numerical Methods in Geomechanics, International Tunnelling Association, World Mining Congress.

Organisers of forthcoming ISRM sponsored meetings reported.

Item 8 on the agenda was the decision on location, time and scope of the next ISRM Congress. The Australian delegation (L. Endersbee, W. Bamford) put on a superb slide show, embracing both the technical and tourist attractions of Australia, I supported their proposal that the Congress should be held in Melbourne in 1983. Actually they were unopposed, although an invitation had been expected from India. The date of the Congress is yet to be set, April, May or August 1983 - this decision will be made by Australia after consultation with all National Groups of ISRM. The major theme of the Congress will be rock mechanics applied to mining. I told Dr Bamford that we would certainly hope to organise pre and post congress tours in NZ. At the closing session of the Congress, when he (as newly elected Vice-President for Australia) issued his invitation to congress participants, he extolled the beauties of NZ and included us in the invitation.

Finally, the next Council meeting of ISRM was set for June 1980 in Stockholm, Sweden - during Rockstore 80 (23-28 June). The 1981 meeting will probably be in Japan during the Congress on Weak Rock.

2nd Session Thursday, 6 September, 1979

Walter Wittke was elected President and Arnaldo Silverio continues as Secretary General.

At this meeting a telegram was read from Pergamon Press, promising that the ISRM list of members should be published as soon as practicable. There had been some anxiety at the first meeting that they were not going to publish it. I had asked at the first meeting that the agenda should include NZ's proposal for a combined list of members of ISRM, IAEG, ISSMFE. Dr Silverio told me afterwards they did not include this item, as it is already under discussion - also there had been quite a lot of difficulty in publishing ISRM's own list. I believe when this has been done we should raise again the question of the combined list. Australia supports it - in fact they are in favour of amalgamating the three societies.

Finally I must thank the NZ Geomechanics Society Management Committee for inviting me to act as their representative. I enjoyed it! May I put in a plea that we should make every effort to be represented at future Council meetings.

2. 4th International Congress of ISRM, 2 - 7 September, 1979

The Congress was divided into four main themes with a general reporter and three lecturers for each:

I "Rheological Behaviour of Rock Masses," M. Langer, general reporter (in German). Lecturers were J. Hardin, "Rheological properties of rocks at high temperatures;" R. Houpert, "The Behaviour of Rocks at Fracture;" K. Mogi, "Flow and Fracture of Rocks under General Triaxial Compression."

II "Use of Tests and Monitoring in Design and Construction," J.A. Franklin. Lecturers were B. Kujundzic (same title); G. Lombardi, "On the Choice of Tests in Rock Mechanics," (in French); W. Wittke, "Interpretation of Results of a Rock Mechanics Program for an Underground Power House with the help of Numerical Calculations." (in German). (The latter was outstanding - the agreement between finite element calculations and measured deformations was most impressive.)

III "Design of Underground Structures with Respect to Modern Construction Methods." D. Prader (in German). Lecturers were - A.M. Muir Wood, "Some Practical Aspects of Conceptual Models in Tunnelling;" M. Panet, "Les Deformations Differees dans les Ouvrages Souterrains;" S. Branonfors, (same title as Prader; from a contractor's viewpoint).

IV "Surface Displacements as a Consequence of Excavation Activities;" M.A. Kanji. Lecturers were - J.A.J. Salas, "Two Cases of Subsidence in Spain" (in French); O.K.H. Steffen, "Monitoring of Deformations in Open Excavations;" P. Sembenelli, "Secondary Movements during Excavation."

I have copies of the four general reports but none of the lectures - they will be published in Volume 3 of the Proceedings, together with discussions but it will be at least a year before these are available. No doubt the papers could be obtained from their authors - I have all the addresses.

Each theme had a main session consisting of the above and some prepared discussion. There was also a second discussion session of just over one hour for free and prepared discussion. Simultaneous translation into English, French, German was provided.

There were some two hundred papers contributed to the proceedings - by far the largest number (107) under theme II. According to the printed list of participants there were forty countries represented and 650 participants pre-registered. I estimate at least a thousand present.

The general reports and lectures conformed to the usual pattern - some very interesting - some very dull! The discussion sessions were certainly disappointing - too many people simply presented "mini-papers", prepared perhaps too late for the proceedings! There was very little "free" discussion" just once or twice a spontaneous and lively discussion developed. (My inclination would be to ban prepared discussion unless it related directly to one of the conference papers!)

There were 15 exhibitors with stalls in the congress building. As usual, the most valuable feature of the congress was the great number of stimulating people one met, and the opportunity to discuss research interests with others in similar fields.

I was personally quite delighted with the format of the opening and closing sessions - before and after the speeches we were treated to concerts by two wind quartets from the Conservatory of Fribourg (two trumpets and two trombones at the opening session; two flutes, trombone and clavichord at the closing session). Apart from this musical feast, there was on one evening a concert by the Prague Chamber Orchestra, on another a visit to the medieval castle of Chillon, and of course the banquet. For the latter, I was invited by Alan Hargreaves (the retiring V.P. for Australasia) to sit at the President and Vice-President's table. I was next to the retiring President, Professor Habib, who was highly amused when I told him in French, that my French was somewhat rusty as I had not had the opportunity to practice it for <u>fifty</u> years (I meant fifteen!).

Finally I must comment on the beautiful location, the perfect weather, and the friendliness and excellent organisation of the Swiss hosts. I must also record my thanks to the Secretary of Energy, and the Deputy Secretary (Mines) for sending me.

Mary Fama

APPLICATION FOR MEMBERSHIP

of

New Zealand Geomechanics Society

A TECHNICAL GROUP OF THE NEW ZEALAND INSTITUTION OF ENGINEERS

The Secretary,
N.Z. Institution of Engineers,
P.O. Box 12-241,
WELLINGTON

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

I hereby apply for membership of the New Zealand Geomechanics Society and supply the following details:

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

PERMANENT ADDRESS ______________________________

____

QUALIFICATIONS AND EXPERIENCE ______________________________

____

NAME OF PRESENT EMPLOYER ______________________________

NATURE OF DUTIES ______________________________

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

I wish to affiliate to:

International Society for Soil Mechanics and Foundation Engineering (ISSMFE) Yes/No($2.25)

International Society for Rock Mechanics (ISRM) Yes/No($6.20)

International Association of Engineering Geology (IAEG) Yes/No ($2; $6 with Bulletin)

Signature of Applicant ______________________________

Date ______________________________ 19 ______

N.B. Affiliation fees are in addition to the Geomechanics Society membership fee of $6.00.

Nomination:

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

Signed ______________________________ Date ____________________ 19 ______

NEW ZEALAND GEOMECHANICS SOCIETY

NOTIFICATION OF CHANGE OF ADDRESS

The Secretary,
N.Z. Institution of Engineers,
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:

____

____

____

Name: ____

Address to which present correspondence is being sent:

____

____

____

Signature __________________________

Date __________________________