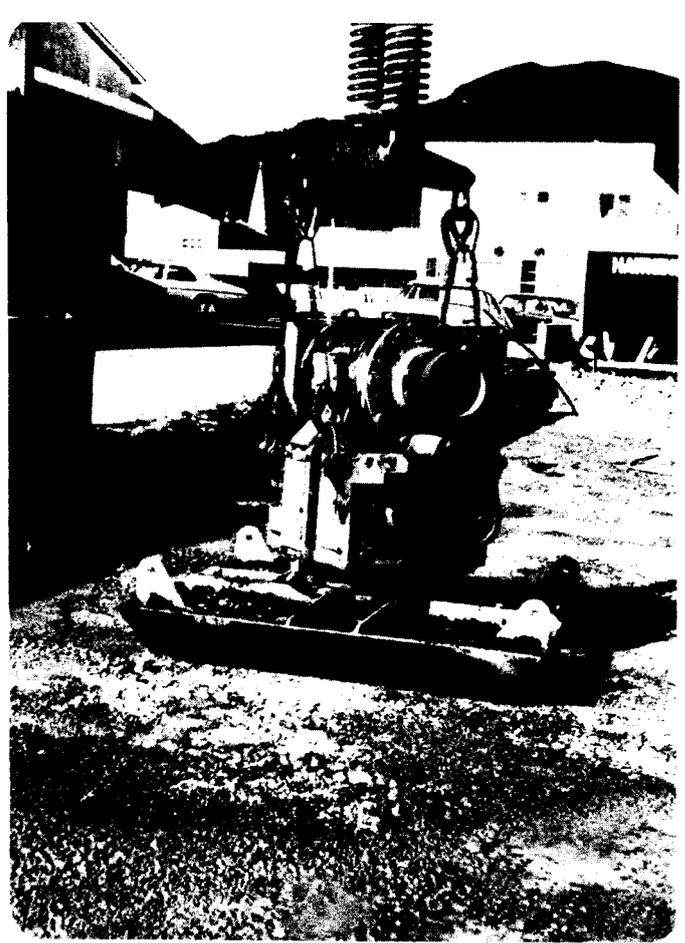# *N.Z. GEOMECHANICS NEWS*

No. 28

JUNE 1984

A NEWSLETTER OF THE N.Z. GEOMECHANICS SOCIETY

Deep Compaction of Loose Sands –
Thames Using – Gilberd Hadfield
Patented Vibro Probe (Effective Depth: 7
to 8 metres)

Surface Compaction Vibro United Bolted
to Steel Tray (Effective Depth: 1.5000
metres)

GROUND STABILISATION

Open Trench – Showing Stabilised Sands Resulting from
use of above methods.

GILBERD HADFIELD

CONTENTS

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 apply for membership of the Society are invited to complete the application form at the back of this newsletter. The basic annual subscription rate is $20.00 and is supplemented according to which of the International Societies, namely Soil Mechanics ($10.00), Rock Mechanics ($13.00), or Engineering Geology ($6.00) the member wishes to be affiliated. Members of the Society are required to affiliate to at least one International Society.

EDITOR'S NOTES

The feature article in this issue is a paper by Professor I K Lee of the University of New South Wales, Sydney, on the prediction of earth pressures on retaining structures. This is a reproduction of a paper prepared for the Mini-Symposium on Earth Pressures on Retaining Walls, Brisbane, June 1983. It is reproduced with the permission of the Author and the Australian Geomechanics Society.

Both parts of NZ Standard 4402 "Soil Testing for Civil Engineering Purposes", are presently under review by SANZ with a Committee largely the same as that which produced the original standards. Suggestions for amendment are being considered and any comment which members of the NZ Geomechanics Society wish to make should be forwarded to our representative, Dr R D Northey, 127 Knights Road, Lower Hutt.

A draft specification describing test procedures for the Standard Penetration Test prepared by the ISSMFE Technical Committee on Penetration Testing, was reproduced in part in the last (No.27) issue of Geomechanics News. Aspects of the draft document not reproduced in issue No.27 include drilling methods and apparatus, and reporting of results. Members of the Society wishing to obtain a full copy of the draft document should contact the Secretary.

The highest international honour for environmental achievement, the Tyler Prize, is awarded annually by an American Institution. Prizes are awarded for the protection, maintenance, improvement and understanding of ecological and environmental conditions anywhere in the world, and for the discovery, further development, improvement or understanding, of known sources of energy or new sources of energy. Anyone interested in further information on this prestigious award should contact the Secretary.

The ISSMFF Lexicon is a publication providing formal definitions of soil mechanics terminology in a number of languages. The latest revised edition of the Lexicon contains 1,592 entries in eight languages (English, French, German, Italian, Portuguese, Russian, Spanish and Swedish), as well as a list of symbols used in soil mechanics and geotechnical engineering with their definitions and units. It contains an index for each of the eight languages. The Lexicon is available at a cost of Canadian $38.00 including postage. Anyone interested in obtaining a copy of the Lexicon should write to the Secretary for further information.

Contributions to NZ Geomechanics would be welcome. They 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.

G.G. GROCOTT
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 $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 all back-issues of "New Zealand Geomechanics News" are available to members at a nominal price of $2.00 per copy.

- The following back issues of the IAEG Bulletin are available. Price $3.00 to members.

Issue	No. available
14	1
15	10
21	1

(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.

P C McGREGOR
Publications Officer

SOME RECENT DEVELOPMENTS ON THE PREDICTION OF PRESSURES

ON RETAINING STRUCTURES

by

I K Lee, BCE, MEngSc, PhD, FIE, MASCE*

INTRODUCTION

In a Memorandum of the Royal Academy published in 1776, a forty year old military engineer, C.A. Coulomb, developed an analysis for predicting the active thrust on a retaining structure. He introduced concepts such as wall friction, a statically determinate solution based on an assumed failure surface, maximum and minimum soil pressures (passive and active), discontinuities, critical height of an embankment, the shear failure criterion, an expression for shear strength considering both cohesive and frictional components, the need for soil testing, and he foreshadowed the method of slices. It is sobering to realize that even the technological turbulence of the 20th century has failed to detect major fallacies in his reasoning and approach to the prediction of active earth pressures. Modern methods (Lee et al, 1983; Chen, 1975) of analysis have been developed from the theory of plasticity and have led to a basic understanding of Coulomb's method. The significant fact is that "Coulomb's" values of active pressure coefficient are generally within 5 per cent of the correct values. This means that the designer can still use the Coulomb values. If he is only concerned with active pressures his major problems are to determine the appropriate c and ϕ values, to evaluate the drainage conditions, to decide on the depth of tension cracks, and the effects of possible swelling pressures.

Since the active and passive pressures are failure states no *direct* reference need be made to the initial stress state, nor to the magnitude of the wall movement, but it must be recognized that the type of wall displacement and rotation can have an important effect on the soil pressures. It is implied in the analysis that the movement is sufficient to induce a shear failure associated with active and passive pressures. There are practical situations which do not involve the full mobilization of active and, particularly, passive pressures and it is required to follow through an analysis of the stress and movement states during and after construction. The ability to carry out such analyses has been the major development in the last decade and does give the designer the opportunity to model the sequence of field operations and the progressive development of the pressures applied to wall.

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Such analysis require modelling of the constitutive relationship for the particular soil. This aspect is still unresolved in terms of basic knowledge but there are a number of empirical stress-strain laws which have been shown to be applicable to specific soils under specific conditions. Thus the designer is required to make a personal judgement about the effectiveness of the constitutive relationship used in the finite element programme.

The present discussion concentrates on the methods for evaluating the active and passive states and recent developments relating the construction history and wall movements to the pressures mobilized at the soil-wall interface.

LIMIT ANALYSES

All limit analyses commence with the assumption that soils can be modelled as a rigid-plastic material and known strength parameters c and ϕ. Exact solutions can be difficult to obtain but there are several simple approaches to the problem of determining the active or pressure thrust which generally give values very close to known correct values.

The limit equilibrium method as developed by Coulomb assumes a plane failure mechanism and applies the equations of statics to solve for the active thrust (Fig. 1). The analysis is repeated for different planes of failure until the maximum thrust is determined. In order for the problem to be statically determinate the strength parameters along the soil-wall interface must be assumed.

The limit state or upperbound solution also assumes a failure mechanism but uses virtual work to establish the active thrust for a given mechanism (Fig. 2). As with the limit equilibrium method the maximum thrust is considered to be the active value. The assumed failure mechanism does not necessarily have to be a single plane surface. Indeed it has been known since the 1920's that the plane surface is inappropriate for the passive analysis and some combination of surfaces are used to model the observed failure surfaces (Terzaghi, 1943; Chen, 1975).

A more rigorous method establishes the lower and upper bound solutions of active or passive pressure. The stress field (Fig. 3) is defined by the equilibrium equations of an element combined with the Coulomb failure criterion. Furthermore, the velocity field (upper bound solution) can be derived from the stress field and the kinematic

Figure 1 : Limit Equilibrium Method

$a\,b = H$

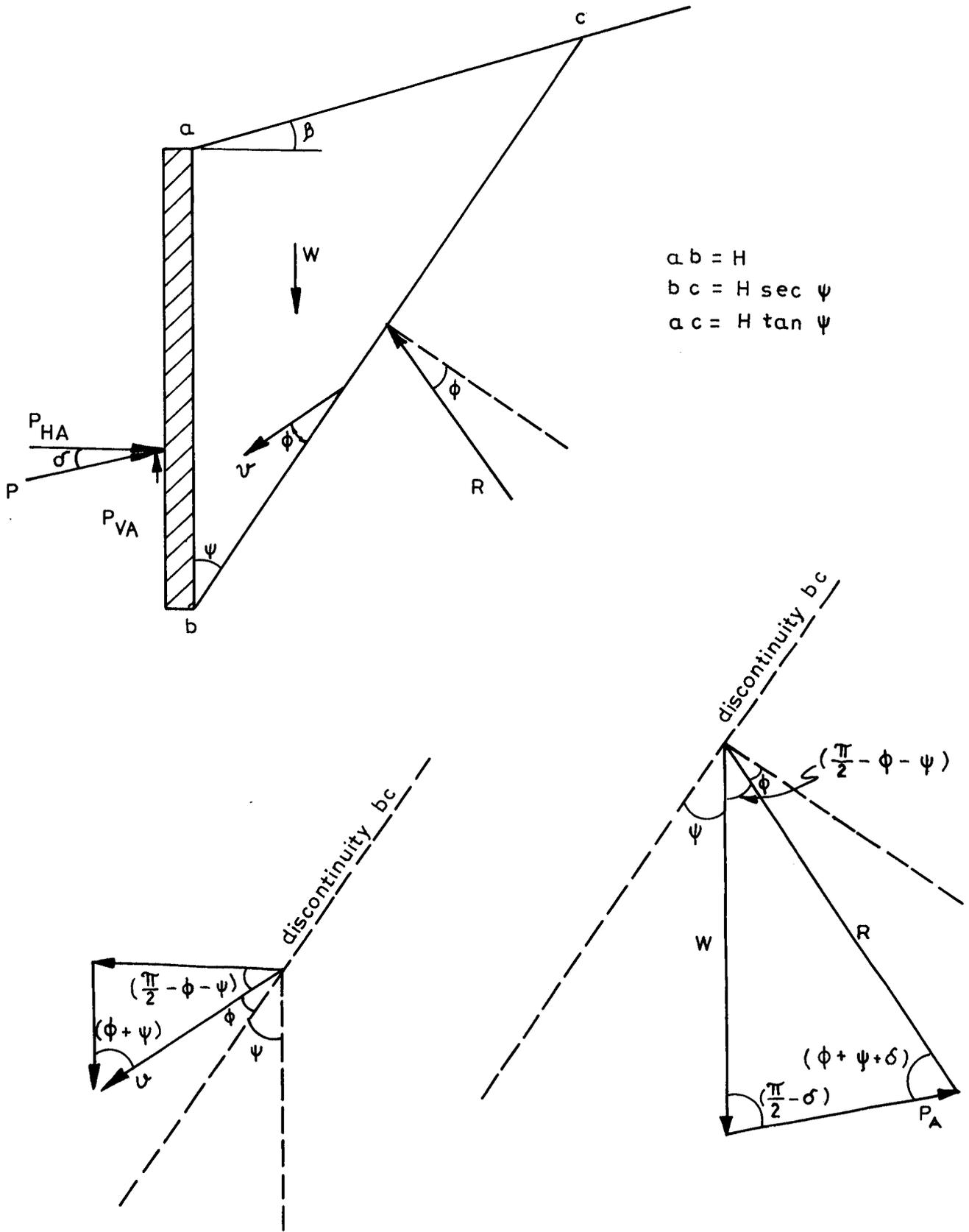$b\,c = H\,sec\ \psi$

$a\,c = H\,tan\ \psi$

Figure 2 : Limit State Method

criterion examined for given velocity boundary conditions thus establishing whether the derived velocity field is kinematically acceptable, that is, whether the solution is an upperbound solution. Within the concepts of applied mechanics this is a rigorous approach which gives the correct solution within defined limits. It does not involve the arbitrary assumption of a specific failure surface.

There are also several other methods involving some combination of statics, assumed failure surfaces, and kinematics (for example, Brinch-Hansen). Some publications give useful numerical values of the active and passive coefficients (Caquot and Kerisel, 1948; Chen, 1975; Lee and Herington, 1972 a,b; Lee et al, 1983). The important conclusion made evident by a study of values derived by the different methods is that the numerical values of active pressure are very consistent. Table 1 shows, for example, the values of the active pressure coefficient for a range of values of the soil friction parameter, ϕ, and soil-wall friction parameter, δ. These results apply to a cohesionless soil, vertical wall and horizontal backfill surface, but the correlation is typical of all practical situations.

$$P_{an} = \tfrac{1}{2} \gamma H^2 K_a$$

Table 1

Comparative Values of the Active Earth Pressure Coefficient K_a. Limit Equilibrium, Upper Bound and Lower Bound Solutions for Vertical Wall, Horizontal Backfill Surface, Cohesionless Soil. (Source Chen, 1975; Lee et al 1983).

Angle of Friction $\phi°$	Angle of Wall Friction $\delta°$	Values of K_a		
		Coulomb	Upper Bound	Lower Bound
10	0	0.704	0.704	0.700
	10	0.643	0.635	
20	0	0.490	0.490	0.490
	10	0.447	0.446	0.450
	20	0.426	0.426	0.440
30	0	0.333	0.333	0.330
	10	0.309	0.307	0.320
	20	0.296	0.297	0.300
	30	0.297	0.302	0.310
40	0	0.217	0.217	0.220
	20	0.199	0.200	0.200
	40	0.210	0.214	0.220

Values of passive pressure coefficients calculated by a limit equilibrium method, for example, the conventional log spiral method (Terzaghi, 1943), the best limit state (upper bound) method (Chen, 1975) and the lower bound method (Lee and Herington, 1972a; Lee et al, 1983) are reasonably consistent. Chen quotes, for example, a comparison of upper and lower bound solutions - see Table 2.

Table 2

Comparison of Upper and Lower Bound Solutions for $K_{p\gamma}$. Cohesionless soil, vertical wall, horizontal back-fill surface. (after Chen, 1975).

ϕ°	$K_{p\gamma}$ Lower Bound			$K_{p\gamma}$ Upper Bound
	Vertical Component	Horizontal Component	Resultant	
30	3.30	5.65	6.55	7.10
40	12.00	14.20	18.60	20.90

Although comprehensive tabulated values of earth pressure coefficients are now available (for example, Lee et al, 1983; Chen, 1975) it is recognized that there are specific combinations of geometry and soil properties which necessitate a specific analysis. The important conclusion is that the designer can use the Coulomb active thrust analysis with some confidence, and, similarly, the conventional friction circle, log spiral passive pressure, or the plasticity upper and lower bound analyses give adequate accuracy.

The major difficulties do not arise from the limitations of the "applied mechanics" analyses but the choice of the relevant values of the c and ϕ parameters. Consideration may have to be given to the type of wall movement. This latter aspect will now be considered in more detail.

INFLUENCE OF THE TYPE OF WALL MOVEMENT

In the conventional active pressure analysis it is appropriately assumed that the backfill moves downwards along the soil-wall interface, and vice versa in the passive pressure analysis. It is clear from the vector diagrams, however, that a reversal of the direction of relative movement would significantly alter the magnitude and direction of the thrust on the wall. The effect of the direction of wall movement is best represented by a polar diagram showing the resultant thrust for
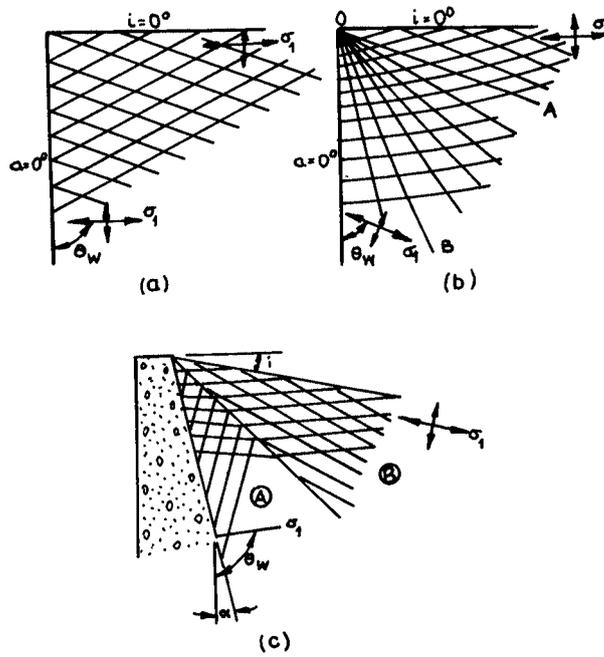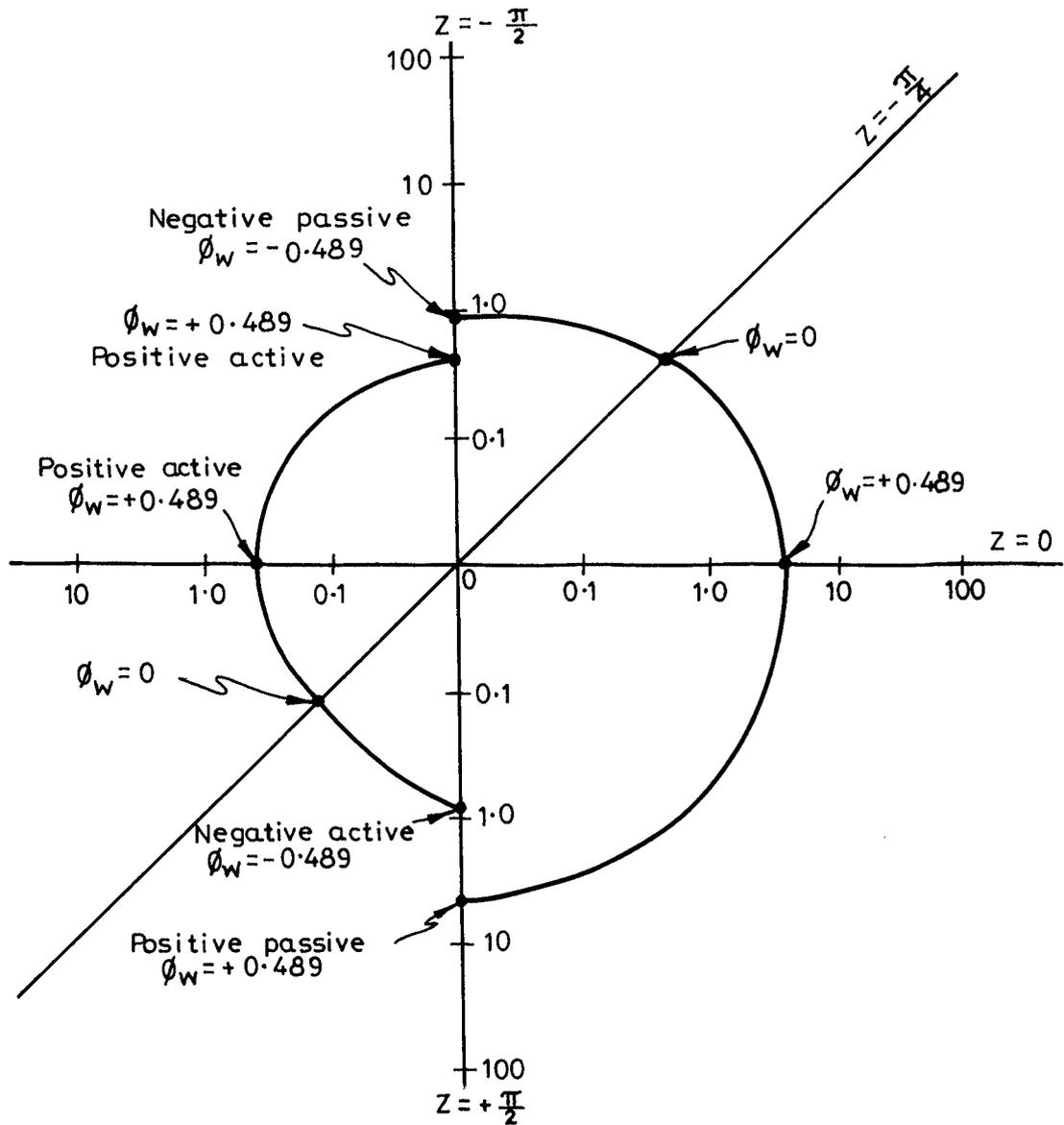
Figure 3 : Stress Fields

Figure 4 : Effect of Direction of Wall Displacement on Active and Passive Pressures

the complete range of possible directions of movement Fig. 4 shows such a plot based on the plasticity analysis (Lee and Herington 1972a, b; Lee, 1974). Consider the passive state, for a c = 0 soil and the conditions existing at large wall displacements. A "positive" state of friction is developed for wall angles, z, from $+ \frac{\pi}{2}$ to 0, that is, from the range of directions from vertically downward to horizontal. When the wall direction is upwards relative to the soil, the wall-soil friction is not fully developed and, in fact, at $z = \frac{\pi}{4}$ the wall friction is theoretically zero. A further increase in z decreases the passive thrust with a minimum value for vertically upward wall movement, the latter corresponding to the "negative" wall friction. There is a similar sequence for the active state as shown in Fig. 4. This applies to large strains ($\psi = 0$)*.

This approach also predicts the value of the angle of wall friction. It was shown (Lee and Herington, 1972b) that the wall adhesion and friction angle are given by the expressions

$$c_W = \frac{c \cos \phi \sin 2(\theta_W - \alpha)}{1 - \sin \phi \cos 2(\theta_W - \alpha)}$$

$$\tan \phi = \frac{\sin \phi \sin 2(\theta_W - \alpha)}{1 - \sin \cos 2(\theta_W - \alpha)}$$

$$\theta_W = \alpha + \frac{\pi}{4} + \frac{\psi}{2} \quad \text{positive, passive}$$

$$\theta_W = \alpha + \frac{3\pi}{4} + \frac{\psi}{2} \quad \text{negative, passive}$$

$$\theta_W = \alpha + \frac{\pi}{4} - \frac{\psi}{2} \quad \text{positive, active}$$

$$\theta_W = \alpha + \frac{\pi}{4} - \frac{\psi}{2} \quad \text{negative, active}$$

θ_W is the angle between the soil-wall interface and the direction of the major principal stress and α is the wall angle relative to the vertical (Fig. 3). ψ^* is defined as

$$\frac{\varepsilon_3^o}{\varepsilon_1^o} = \tan^2 \left(\frac{\pi}{4} + \frac{\psi}{2}\right) \qquad \phi > \psi > 0$$

The important conclusion is that the actual thrust lies between the limiting positive and negative values. For intermediate directions of movement the likely value can be estimated from Fig. 4. However, in practice it would be anticipated that the positive state will be developed except when some type of constraint imposes "unconventional" movements.

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*an associated flow rule material is defined as $\psi = \phi$, and the bound theorems only apply to this model.

ACTIVE AND PASSIVE THRUSTS AND PRESSURES

The conventional method of expressing active (a) and passive (b) pressures is

$$P_{an} = K_{an}\, \gamma Z - cK_{acn}$$

$$P_{pn} = K_{pn}\, \gamma Z + cK_{pcn}$$

the subscript, n, referring to the normal component. These expressions imply a pressure increasing linearly with depth Z. An alternative, but equivalent method is to write the expressions for the horizontal and vertical thrust components, viz. for the active thrust components,

$$P_{Ha} = H(cN_{cHa} + qN_{qHa} + \gamma HN_{\gamma Ha})$$

$$P_{Va} = H(cN_{cVA} + qN_{qVp} + \gamma HN_{\gamma Va})$$

and for the passive thrust components

$$P_{Hp} = H(cN_{cHp} + qN_{qHp} + \gamma HN_{\gamma Hp})$$

$$P_{Vp} = H(cN_{cVp} + qN_{qVp} + \gamma HN_{\gamma Vp})$$

where N_c, N_q, and N are earth pressure factors analagous to the bearing capacity factors and express the effects of cohesion, surcharge and body force respectively. H refers to the horizontal component, V refers to the vertical component. Values of these coefficients for the active state are given in Lee et al (1983) (see Tables 6.6) for a range of values of wall angle, backfill slope, friction angle, depth of water table, and depth of tension cracks. It will be noted that multi-layered deposits can be readily evaluated given the earth pressure coefficients. A limited range of values for the passive state are also given (Table 6.8). Chen (1975) gives a more extensive range.

MOBILIZATION OF SOIL PRESSURES

It is now possible to predict the relationship between wall displacements and the pressures developed as a consequence of these displacements (Lee et al, 1979). Horizontal displacements of the order of 0.1 to 0.2% of the wall height are sufficient to develop the active state in a dense sand fill, whereas the full mobilization of the passive state requires a wall displacement into the fill of about 0.05 times the wall height (see Lee et al, 1983; Tables 6.3a and 6.3b). Displacements to produce the active and passive states in loose sand fill, are an order greater.

The most effective analytical technique involves the representation of the backfill and the underlying soil by isoparametric finite elements (for example, Zienkiewicz, 1971; Desai, 1972), and a modelling of the soil-structure interface by a one dimensional element (for example, Mahtab and Goodman, 1970). The designer is faced with the problem of modelling the stress-strain relationships for the soils, and to establish the size of the finite element programme which can be accommodated by his computer. There have been numerous constitutive equations prepared for specific soil types and in the present discussion consideration will be limited to the linear elastic-plastic model and a non-linear elastic-plastic model - the latter specifically developed for medium to dense granular backfill.

The designer may not have to consider wall movements in detail, for example, it is usually accepted that the wall movement away from the fill is sufficient to develop the active state. There are, however, many situations where the movements need to be evaluated with reasonable accuracy. This is particularly important when considering the support which can be mobilized as a structure moves or rotates into the soil mass, for example, when existing gravity dams are strengthened by soil or rockfill it is necessary to determine the support offered by the backfill for a specific downstream movement of the existing structure.

The designer may be interested in evaluating the progressive development of earth pressures as the backfill is placed. Fig. 5 shows the distribution of earth pressure immediately after completing construction when the displacement of the wall is considered to be zero - the so called K_0 state. In this analysis the soil was considered to be placed in a series of layers and as each layer was "placed" gravity was "turned on". Such an analysis models an unyielding support system so that the pressures exceed the active values. Small movements or rotations away from the fill will lead to the full development of the active state. It will be noted that the use of the linear elastic-plastic model predicted a stress state very close to the elastic distribution. It will also be noted that the elastic-plastic model included a no-tension criterion.

A series of analyses will now be discussed to show the relationship between the mobilized earth pressures and the wall movements or rotations. For this purpose the following conditions are considered (see Fig. 6):
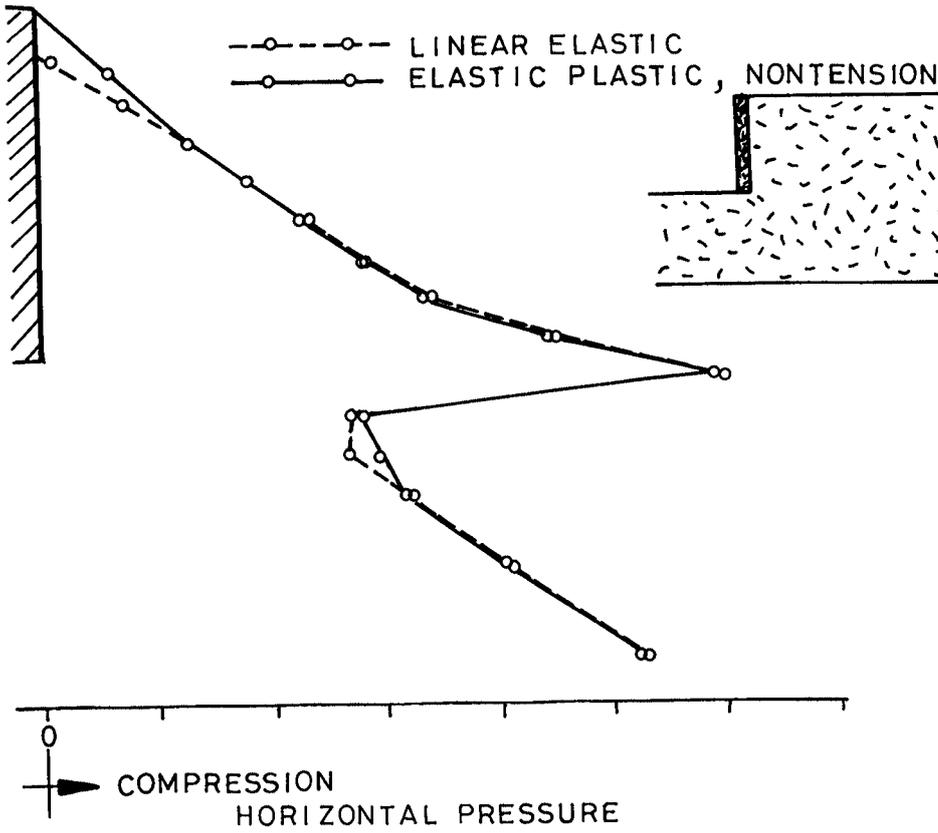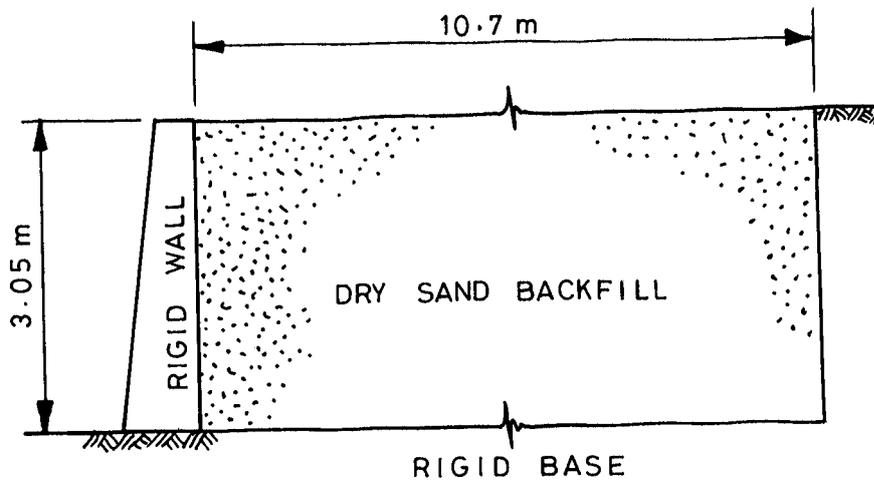
Figure 5 : Construction Earth Pressures

Figure 6 : Details of Wall

Wall height	3.05 m
Rigid base	
Rigid wall which can translate, or rotate about the top or the base	
Soil modulus	48.2 mPa
Poisson's ratio	0.3
Cohesive parameter	0
Friction parameter	35°
Unit weight	17.5 kN/m³

The one dimensional interface elements inserted between the rigid wall and the backfill, with assigned stiffness values depending on the interfacial condition to be modelled. An interfacial element is also placed between the base and the backfill to model a rough interface. Initially the wall is considered to be held rigidly as the backfill is placed in layers. An elastic-plastic analysis modelling of the construction sequence established a coefficient of lateral pressure of 0.48, and a hydrostatic distribution of pressure with respect to depth.

Fig. 7 shows the progressive reduction in the earth pressures as a smooth wall is progressively rotated about the base away from the backfill. It is seen that the pressures approach the classical active values except in the vicinity of the base where the strains are not sufficient to induce a failure state. Fig. 8 shows the relationships between the earth pressure coefficient and wall rotation (expressed as a ratio of the lateral displacement, Δ, at the top of the wall to the wall height, H. Empirical data suggests an active state is fully developed at a ratio of Δ to H of about 0.06.

The corresponding predictions for horizontal translation away from the fill are shown in Fig. 9. The Δ/H ratio in the (essentially) active state can be compared with the previously quoted empirical range of 0.1 to 0.3%.

Partially mobilized passive states for rotation about the base and for horizontal translation are shown in Figs. 10, 11 and 12. The effect of small strains in the lower portions is particularly evident in Fig. 10. This distribution is consistent with field and laboratory observations.

All of the above cases considered the wall-soil interface to be free of shear stress. It was shown earlier that the shear stress free condition can only occur when there is zero relative movement between the soil and the interface and this specific condition is not fulfilled
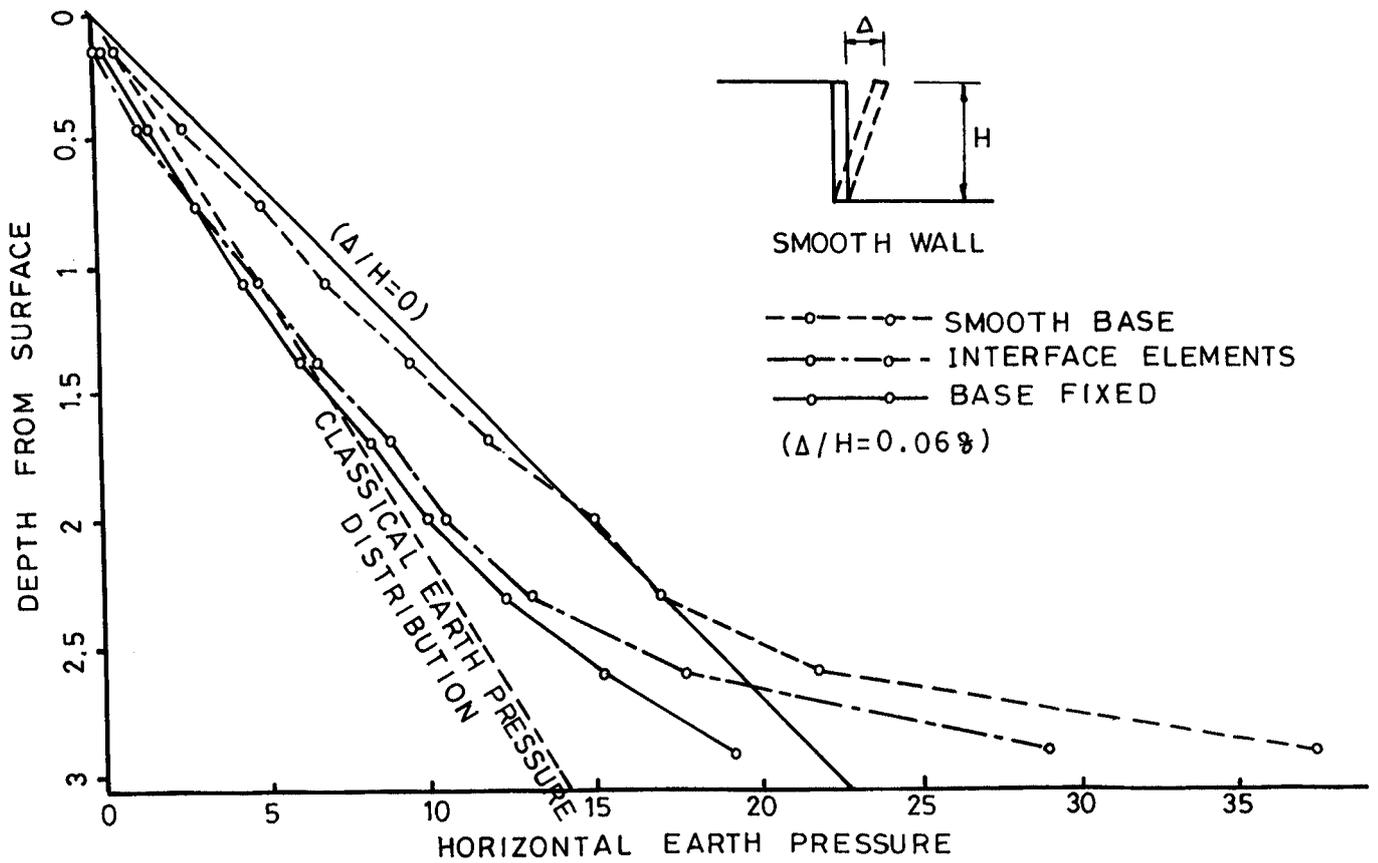
Figure 7 : Mobilisation of Active Pressure Smooth Wall Rotated About Base
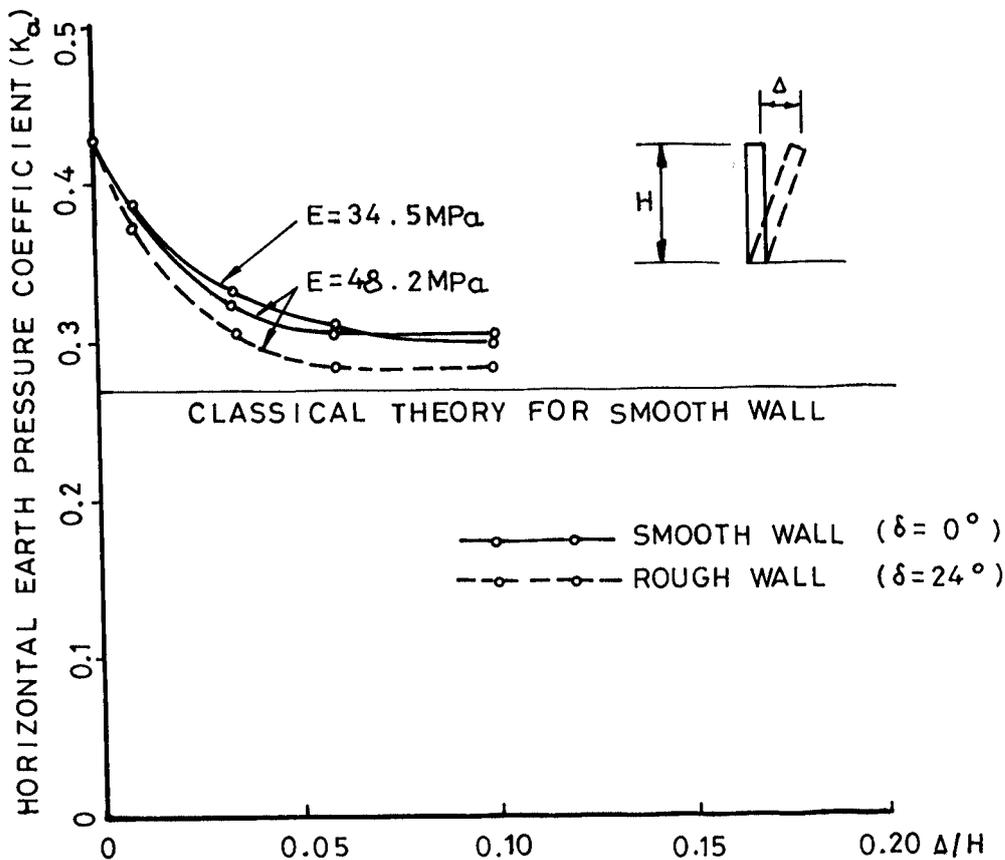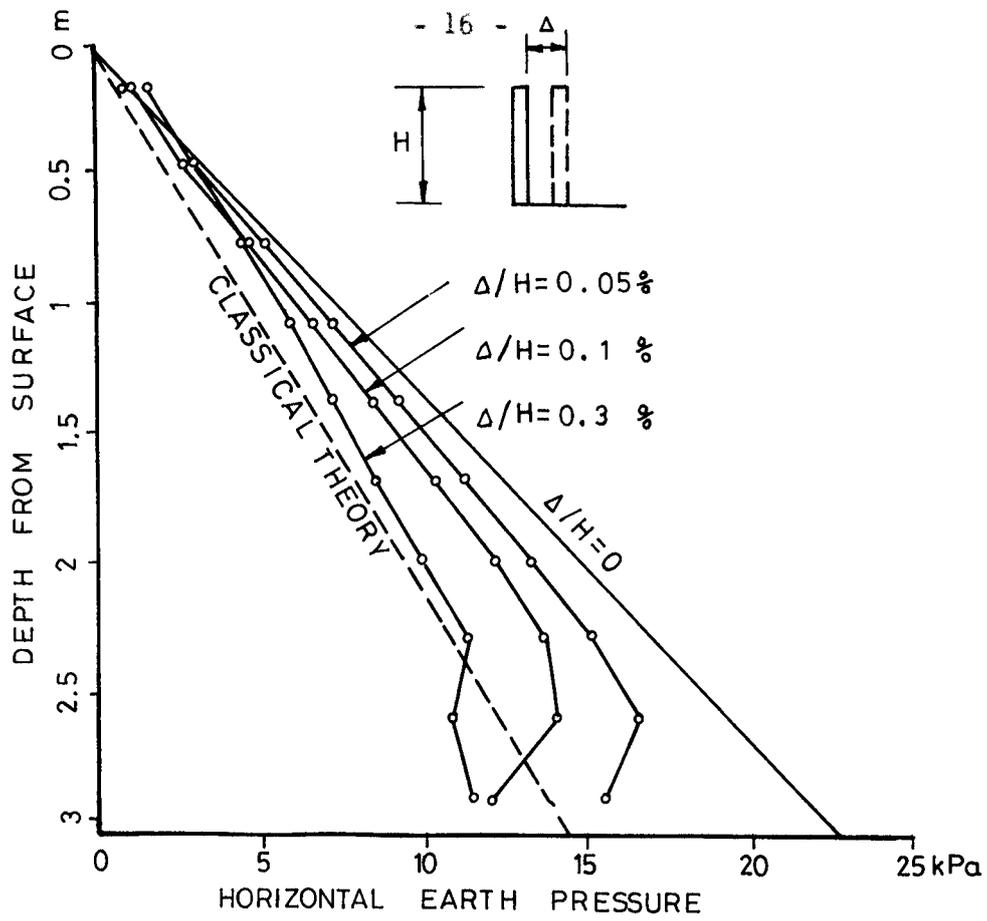

Figure 8 : Development of Active State

Figure 9 : Mobilisation of Active Pressure - Smooth Wall.
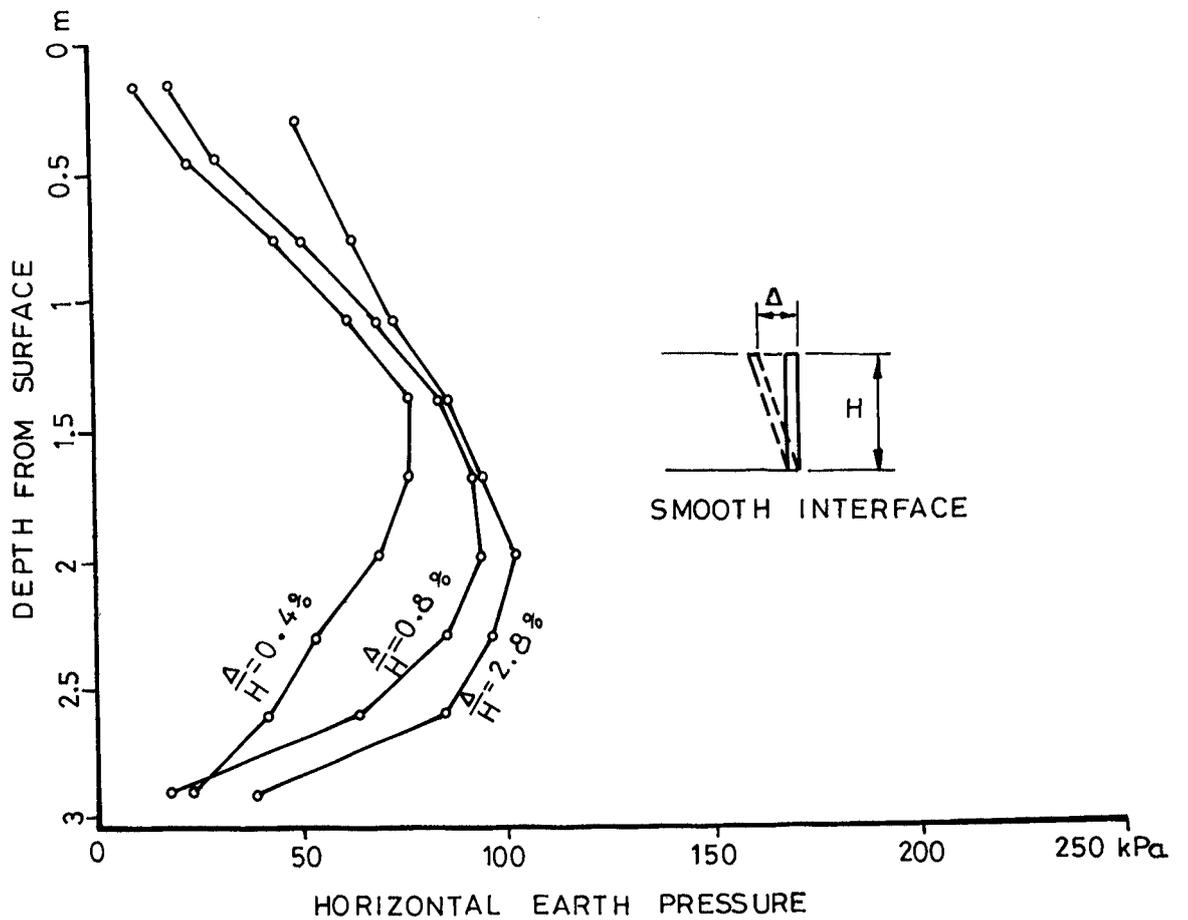Horizontal Translation.

Figure 10 : Mobilisation of Passive Pressure - Smooth Wall
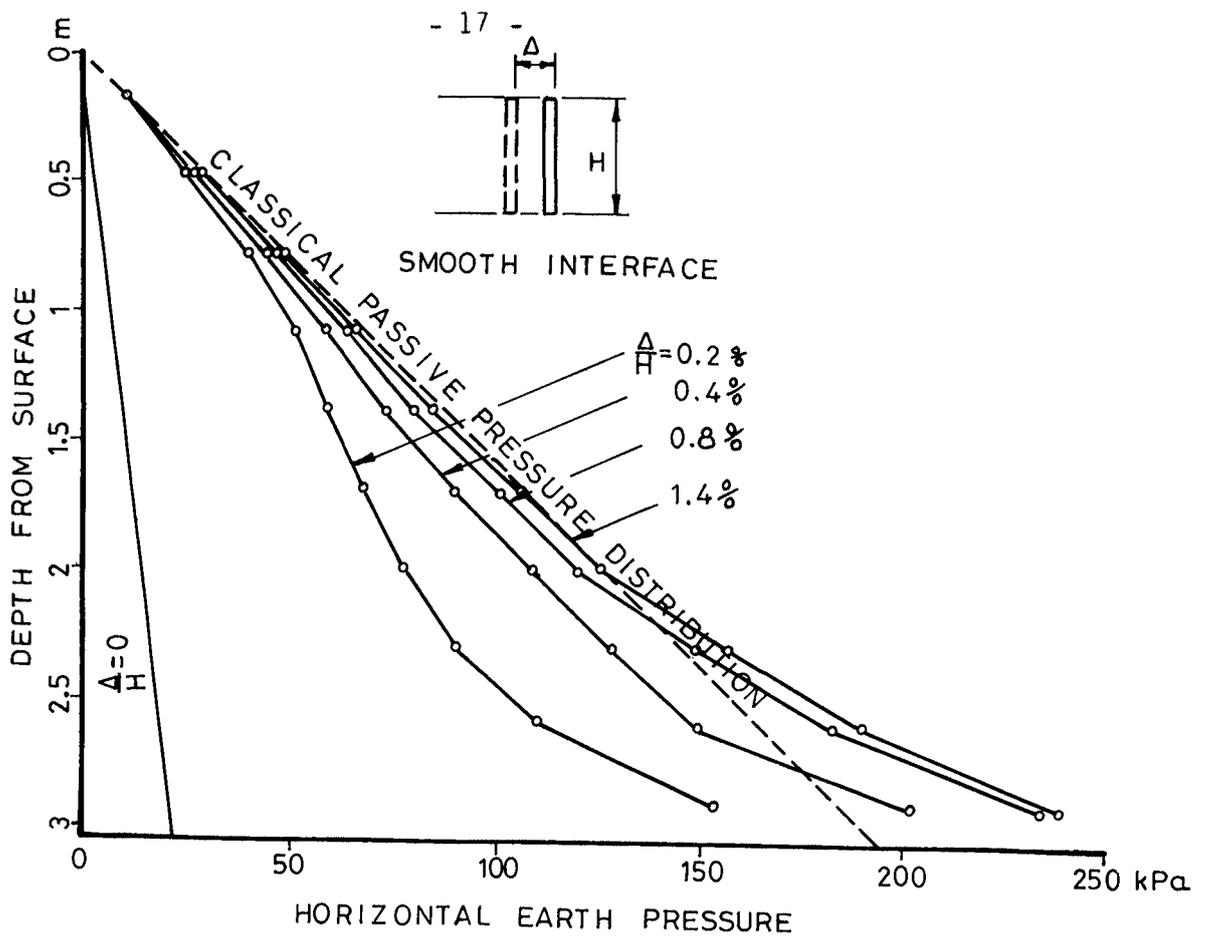Rotated About Base

Figure 11 : Mobilisation of Passive Pressure - Smooth Wall. Horizontal Translation.
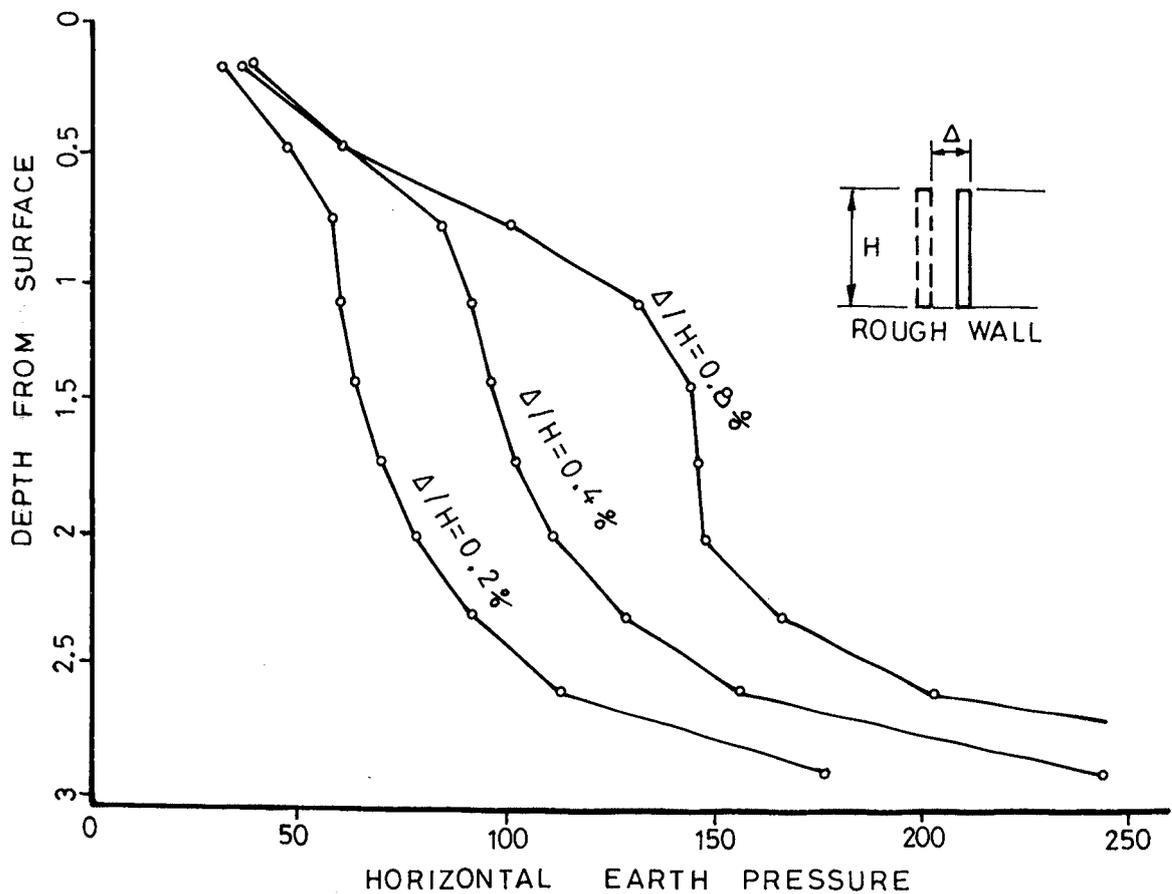

Figure 12 : Mobilisation of Passive Pressure - Rough Wall. Horizontal Translation.

in the cases considered to date. The more realistic model for horizontal translation, for example, is the rough interface (see Fig. 4). A value of wall friction, δ_w, was assumed since this is the usual (reasonable) design value. As one would anticipate the rate of mobilization of pressure for the rough interface is greater than for the shear stress free interface.

To further examine the question of the development of wall friction as related to the direction of wall movement, reference can be made to Figs. 13 and 14. These results apply to the usual situation where there is a layer of soil supporting the wall. For simplicity it will be considered that this soil is the same as the backfill. It is emphasized, however, that the analysis is equally applicable to a multi-layered deposit.

This analysis differs from the preceding cases in that the soil model adopted is the non-linear elastic relationship (Duncan and Chang, 1970) combined with a failure criterion which restricts the stress level at higher strains. Such a model appears to be a reasonable representation of a cohesionless soil.

The tangent modulus for the soil is given by the expression (Duncan and Chang, 1970)

$$E_t = K \, p_a \, \left(\frac{\sigma_3}{p_a}\right)^n \left(1 - \frac{R_f \, (1-\sin \phi)(\sigma_1 - \sigma_3)}{2\sigma_3 \sin \phi}\right)^2 \quad \ldots(1)$$

and for the interface the tangent shear modulus is

$$K_{st} = K_i \, \gamma_w \, \left(\frac{\sigma_n}{p_a}\right)^n \left(1 - \frac{R_f}{\sigma_n \tan \delta_w}\right)^2 \quad \ldots(2)$$

where p_a =   atmosphere pressure

   R_f =   factor

   σ_1, σ_3 =   principal stresses

   ϕ =   friction angle

   σ_n =   normal stress

   K_i =   stiffness number

   n =   stiffness exponent

   γ_w =   unit weight of water

   ϕ_w =   angle of wall friction

The failure criterion is introduced to avoid an overestimation of the post-failure strength of sand. Thus the non-linear elastic model represented by equation (1) will be terminated at some "failure state"
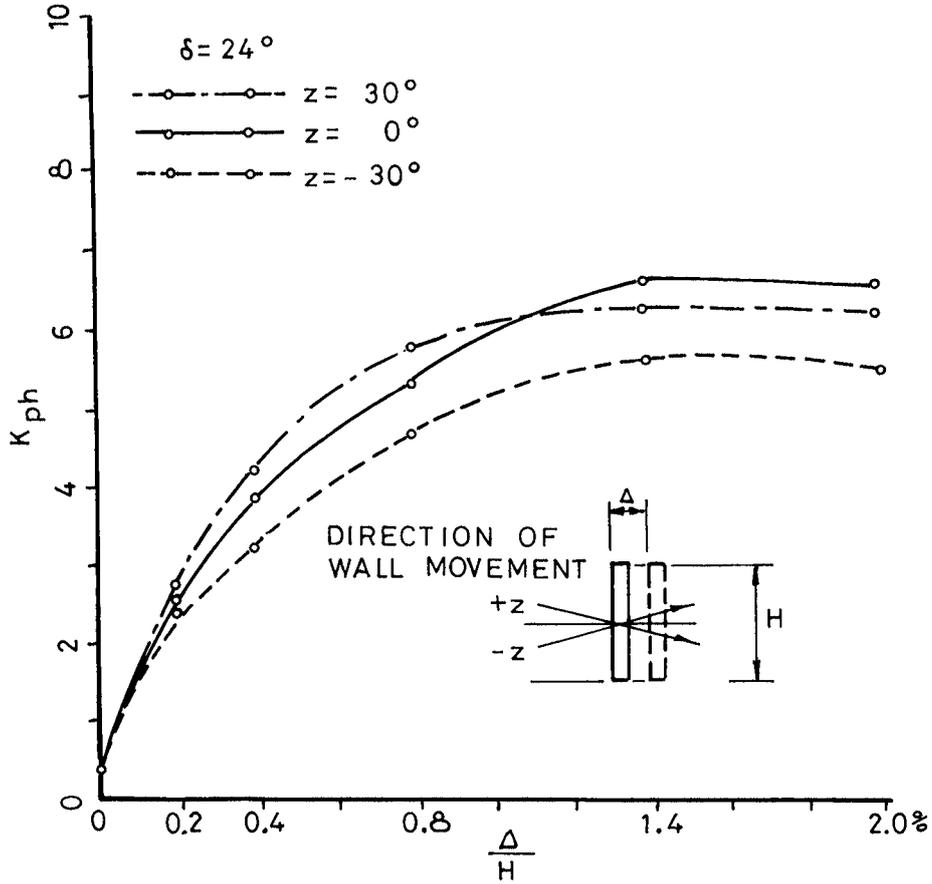
Figure 13 : Effect of Direction of Wall Displacement on Mobilisation of Passive Pressure
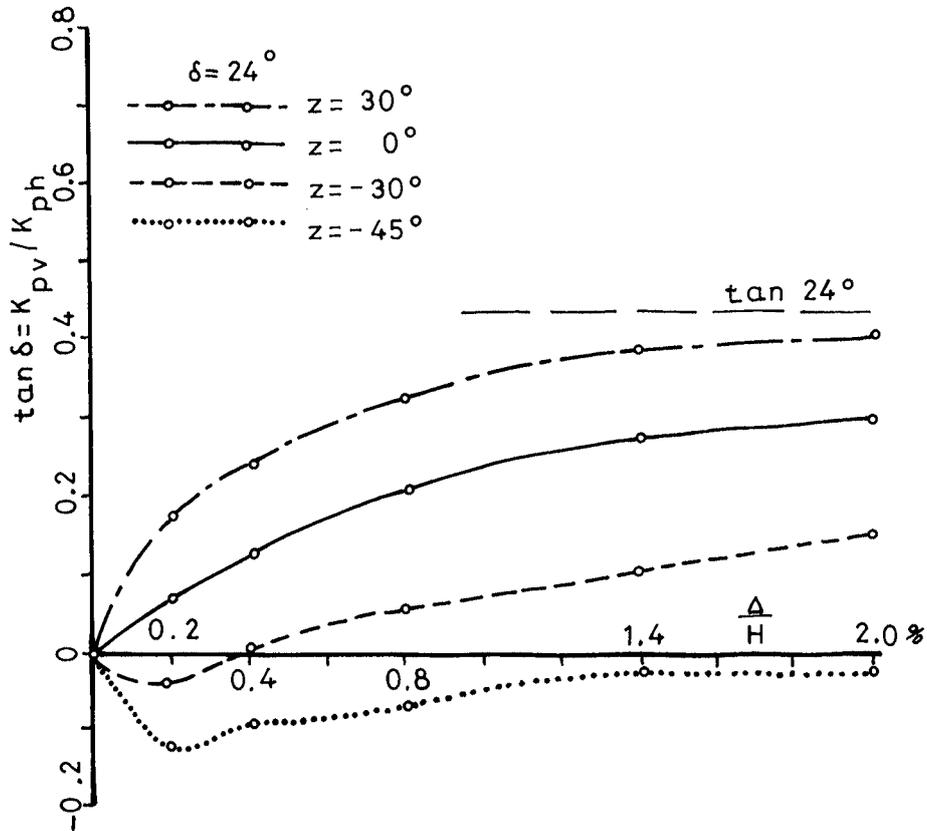

Figure 14 : Effect of Direction of Wall Displacement on Mobilisation of Wall Friction

and at larger strains the deviator stress will be considered to remain constant.

Returning to Figs. 13 and 14 it is seen that the trends are in agreement with the behaviour anticipated from the earlier discussion on the effects of the direction, z, of the wall movement. In particular, it is seen that the mobilized interfacial stress is quite small when the direction of movement, z, is about 45° and this corresponds very closely to a zero interfacial (smooth) stress state. This fact also emphasizes that a condition of a smooth interface can only occur under very special conditions. The assumption of δ_w of $^2/3\phi$ is usually quite reasonable as it applies to the usual types of wall movements one would generally encounter in practice.

APPLICATION OF NUMERICAL METHODS TO SPECIFIC RETAINING STRUCTURES

The discussion in the preceding section established that the designer can now attempt to predict the performance of the retaining structure during construction and during any subsequent changes in the loading state. There are, however, definite limitations to the numerical methods so that one cannot assume that the analyses are routine. Solutions do not necessarily converge, there may be some difficulties with boundary conditions, the capacity of the computer available may be inadequate for the number of finite elements used to represent the system, and the computational time can be excessive when any non linear characteristic is incorporated into the analysis. If the structure must be represented as a three-dimensional system the computing time is generally increased by at least an order. However, it is usually possible to extrapolate from a two-dimensional solution except when dynamic loading is considered.

The stability under earthquake loading may have to be considered even under the relatively inactive Australian seismic state. Analyses can be completed for a given earthquake loading using the finite element method (for example, Aggour and Brown, 1974; White, 1975; Valliappan, 1978). Such analyses show that the pressure distribution on the wall is approximately a cosine shape with a maximum value at the top of the wall and the resultant about two thirds of the height from the base. There are dynamic programmes available but again the use of the programmes can lead to difficulties unless the analysis is properly understood. For example, it has been shown that the grid spacing is controlled by the frequency of the disturbing wave as the grid filters out frequencies higher than a value determined by the spacing. (White, et al 1977).

Many retaining structures have been instrumented to record pressures and displacements. There are, however, relatively few "controlled" full scale experiments which provide adequate reliable data for a comparison of predicted and recorded performance. The Japanese Society of Soil Mechanics and Foundation Engineering concluded in 1969 that there was no "reliable" data on "design earth pressure". This conclusion initiated a series of large scale tests on retaining structures up to 11 m high. (Matsuo et al, 1978; Fukuoka, et al, 1975). There was, in fact, reasonable confirmation of the active thrusts and the displacements and rotations producing the states of failure. However, one important question which does not appear to be resolved is the possibility of an increase in lateral pressure with time subsequent to the development of an active state. Such an increase has been postulated on several occasions to explain the failure of a wall after a long period of service. Fig.15 shows the thrust-time measurements made by Matsuo on a 10 m high wall rotated about the base. Three types of backfill were used - silty sand (c = 23 kPa, ϕ = 27°) and two cohesionless slags, types A & B (c = 0, ϕ = 50°), the latter slag composed of uniform gravel sized material whereas type A is a well graded material composed of gravel to coarse sand sizes. There was a steady increase in thrust after the K_0 state had been reduced to the active state, for example, the retaining wall was moved away from the silty sand fill on day 60, and this reduced the thrust from 52 t/m to 20 t/m. There was a significant recovery although the wall was fixed in position.

There has not been consistent evidence showing a rise in lateral pressure following wall movements leading to an active state in cohesionless backfills. Laboratory tests using relatively small retaining wall models have not indicated a recovery of thrust to a value corresponding to the at-rest state. This may be correct or simply a scale effect. Further studies are needed to establish whether Matsuo's observations were a result of the particular conditions or testing technique. It does indicate that the design value of the earth pressure coefficient should be conservatively chosen and combined with a load factor in excess of two.
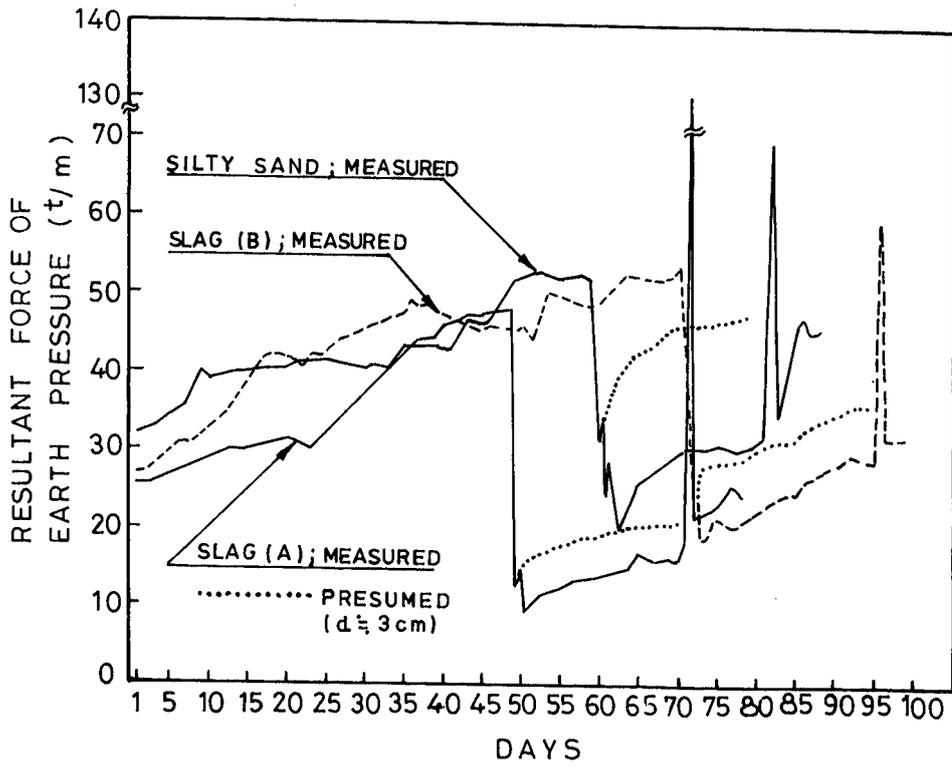
Figure 15 : Change of Resultant Force of Earth Pressure

CONCLUDING COMMENTS

Modern methods of analysis have shown that the Coulomb method for calculating the active pressure coefficient, and the traditional passive earth pressure calculations lead to values of acceptable accuracy. Finite element analyses can be used to predict the relationships between the pressures acting on a retaining structure and the wall movement. Such analyses, however, require careful consideration of the type of constitutive law adopted for the backfill.

In a complex practical situation it may be necessary to follow through the construction and post construction sequence by a finite element analysis, rather than assuming specific displacements will occur and thus adopting the relevant active or passive coefficients.

Evidence of an inevitable time dependent increase in thrust following outward movement of a wall has been established by tests on a 10 m wall under carefully controlled experimental conditions. This observation is consistent with the fact that many early retaining structures collapsed after a considerable period although such instabilities have been structures supporting clay backfill. This question has not been resolved and it appears that a conservative approach is necessary.

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NEWS FROM THE MANAGEMENT SECRETARY

1. MANAGEMENT COMMITTEE

The Management Committee for 1984 is:

T J Kayes	(Chairman)	Wellington
S A L Read	(Secretary)	Wellington
D H Bell	(Australasian Vice-President, IAEG)	Christchurch
H H Galloway	(IPENZ Appointee)	Wellington
G G Grocott	(Editor, Geomechanics News)	Auckland
D N Jennings		Wellington
N S Luxford	(Vice-Chairman, Soil Mechanics)	Auckland
P C McGregor	(Publications Officer)	Auckland
P J Millar	(Vice-Chairman, Rock Mechanics)	Wellington
R D Northey	(Australasian Vice-President, ISSMFE)	Wellington
A J Olsen	(IPENZ Appointee)	Tauranga
B R Paterson	(Vice-Chairman, Engineering Geology)	Christchurch

2. LOCAL GROUP ACTIVITIES CONVENORS

Auckland	P B Riley	Consultant Engineer
Wellington	D N Jennings	MWD, Head Office
Christchurch	D H Bell	Geology Dept, University of Canterbury
Dunedin	W J Henderson	Dunedin City Council

3. NEW MEMBERS

The following new members are welcomed to the Society:

K A Boddie	R A Wilson	J Se Kula	R F Lindsay
D M Graham	D S Coombs	J D Temple	D A J Manhire
R J Lister	W S Lin	J C Yonge	B G Shakes
K J Patterson-Kane	W C Liu	P F Foster	I G Walsh
T J E Sinclair			

4. 4th GEOMECHANICS LECTURE

Professor Peter Taylor has accepted an invitation from the Geomechanics Society to present the 4th Geomechanics Lecture. The award of this 4th Geomechanics Lecture to Professor Taylor is in recognition of the wide respect in which he is held and the significant contribution which he has made to the field of geomechanics.

The 4th Geomechanics Lecture is the highlight of the year's activities and should not be missed by members. Details of dates and venue for the lecture are being finalised and members will be advised.

Tentative dates are:	Wellington	Tuesday	25 September
	Christchurch	Wednesday	26 September
	Auckland	Wednesday	3 October

The possibility of a Dunedin presentation on Thursday 27 September is being investigated. Any enquiries should be directed to your local group convenor.

5. IPENZ 1985 CONFERENCE

The Conference is to be held in Wellington from 11-15 February 1985. As in past years, the Geomechanics Society will contribute to the Conference which this year has the theme, "Innovation in Engineering".

The Conference will cover a wide scope of engineering including aspects such as risk, earthquake damage effects and construction management. Papers are required for presentation during the two Geomechanics Society sessions at the Conference. Intending authors should submit synopses of their papers to the Management Secretary by 31 August. The deadline for submission of draft papers for pre-printing will be 30 November 1984.

The Annual General Meeting of the Society will take place during the Conference.

6. IPENZ AWARDS

The Institution annually makes a number of Awards for papers presented by members. Nominations are being sought from Society members for the following awards.

a) Fulton/Downer Award - for papers presented at the Institution Conference.

b) Furkert Award - for papers in Civil Engineering, particularly the interaction of water on the faces of nature.

c) Rabone Award - general nature subject not qualifying for one of the other awards.

d) Environmental Award - for predominantly engineering work which best exemplifies care for and consideration of environmental values.

Further information on the above awards is outlined in a brochure issued by the IPENZ Secretariat. Nominations should be forwarded to the Management Secretary by 30 September.

The Otto Glogau Award sponsored by the New Zealand National Society for Earthquake Engineering will again be awarded this year. Further details may be obtained from the secretary of that Society.

7. DRAFT METHOD OF SOIL AND ROCK DESCRIPTION

The sub-committee has received three additional submissions since November last year. The committee has met twice this year and apart from discussion on the textural or property usage of the terms silt and clay, has finalized a method for soil. Little progress with the rock description method has consequently been made and unfortunately, the combined draft method does not accompany this issue.

The committee apologizes for this and intends to meet again shortly so that the method may be circulated in the near future.

SAL READ
Management Secretary

FORTHCOMING CONFERENCES

04-14	August	1984	27th International Geological Congress. Moscow, USSR.
03-06	September	1984	Design and Performance of Underground Excavations. Cambridge, United Kingdom.
16-22	September	1984	IVth International Symposium on Landslides. Toronto, Canada.
08-13	October	1984	1984 Annual Meeting of the Association of Engineering Geologists. Boston, USA.
09-11	October	1984	In Situ Soil and Rock Reinforcement. Paris, France.
15-17	October	1984	Strata Control in Mines and Tunnels. University Park, USA.
22-24	October	1984	Fifth Australian Tunnelling Conference. Sydney, Australia.
29 Oct-2 Nov		1984	NZ Institute of Mining, Mining Conference. Westport, New Zealand.
19-23	November	1984	12th World Mining Congress. New Delhi, India.
01-05	December	1984	Geotechnical Aspects of Mass and Material Transporation. Bangkok, Thailand.
11-14	February	1985	Geomechanics in Tropical, Lateritic and Saprolitic soils. Brasilia, Brazil.
	June	1985	4th International Symposium on Tunnelling. Brighton, United Kingdom.
06-08	August	1985	International Workshop on Dam Failures. West Lafayette, USA.
11-15	August	1985	XI ICSMFE. ISSMFE Jubilee International Congress. San Francisco, USA.
15-21	September	1985	First International Conference on Geomorphology. Manchester, England.
	September	1985	The Role of Rock Mechanics in Excavations for Mining and Civil Works. Mexico City, Mexico.

Further information on these conferences may be obtained by writing to the Management Secretary or the Vice-Chairman of the appropriate discipline.

S A L READ
Managing Secretary

FROM THE INTERNATIONAL VICE-CHAIRMEN

1. ROCK MECHANICS

1.1 M Rocha Medal

The second recipient of the medal is S Bandis of Greece for a thesis "Experimental Studies of Scale Effects on Shear Strength and Deformation of Rock Joints" submitted to the University of Leeds, UK 1980.

1.2 6th Congress

The 6th Congress is to be held in Montreal, Canada, in 1987. The first call for papers is due to be announced in June 1984.

1.3 The 1984 ISRM Council Meeting of the Society will be held in Cambridge, UK, in September.

P J MILLAR

2. ENGINEERING GEOLOGY

2.1 The International Institute of Aerial Survey and Earth Sciences of the Department of Mining Engineering, Delft University of Technology, are now offering a new degree course in engineering geology commencing November, 1984. Further details can be obtained from the Vice-Chairman.

P B PATERSON

LOCAL GROUP ACTIVITIES

WELLINGTON GROUP

The last meeting for 1983 was a combined Geomechanics Society/Wellington IPENZ Branch meeting on 29 November, which was attended by some 50 people. Guest speaker was Professor David Stapledon who had just attended the Alexandra Symposium on Engineering for Dams and Canals. He gave a very interesting address titled 'The Geotechnical Specialist and Contractural Disputes'.

Professor Stapledon was well qualified to speak on this subject with his involvement in 13 contractural disputes in a variety of works including tunnels (3), dreging (3), piling (2), quarrying (2), dam foundations (1), railways (1) and highways (1).

In these disputes he has acted for both the principal and contractor, with most settled by discussion and negotiation.

He outlined his ground rules before accepting a case:

. He must not know any details of the construction or dispute.

. He must be able to access the data available at the time of tender, including design, specification, general conditions, bill of quantities, geotechnical data.

. He will report on his assessment of the site conditions.

. The client pays the bill regardless of whether the report is favourable or not.

. The client can then either sack or retain his services for further advice.

Two case histories involving the construction of tunnels were discussed by Professor Stapledon. Case 1 involved a 2.5 km tunnel which it turned out was constructed partly through landslide disturbed/weathered rock. Case 2 involved the construction of a tunnel through limestone and dune sand. Both cases involved subtle geological conditions which, when explained by Professor Stapledon, appeared simple and obvious.

Those interested in further details are referred to a paper by Professor Stapledon in 'Engineering Geological Case Histories' published by the Geological Society of Australia, December 1983.

A very successful meeting of the Wellington Group was held on 8 March 1984. The guest speaker was Dr Don Deere, an international consultant in rock mechanics, from Florida, U.S.A. Dr Deere, who was in Wellington consulting to M.W.D. on the Clyde Dam, addressed the meeting regarding 'Some Experiences with Adverse Geological Features at Hydro Projects'.

In his introduction Dr Deere commented, "Adverse geological features are encountered in all types of Civil Engineering Projects, including foundations of buildings and bridges, stability of slopes for highway, railway and canal cuts, stability of tunnels and underground stations, caverns for subway and metro railway and water conveyance purposes.

Problems are often related to certain weak geological features that possess the following characteristics:

. They have a high degree of continuity.

. They have low shearing resistance and often low modulus.

. They have critical orientations with respect to the applied forces of the structure.

New hydro electric projects in particular are subject to some of these geological weaknesses, because hydro electric projects contain many different elements and therefore they are vulnerable to a variety of construction and operational problems.

These elements include the dam itself, its foundation and its abutments, the access roads to the project, diversions, canal or diversion tunnel, power tunnel, tailrace tunnel and possibly a surge tank or surge chamber and massive rock excavations for the powerhouse, either at the face of the dam or possibly as an underground powerhouse."

These comments were illustrated with two examples. In the first case, the project described was located in very steep mountainous terrain and many areas of landsliding had been identified along the river valley. One of these was immediately upstream of the dam on the true right bank. The dam was a gravity arch about 80m high.

During lake filling, a 40,000m^3 part of the slide became displaced into the reservoir. This prompted additional efforts to stabilise the slide before the reservoir was raised to its full operational level.

Dr Deere described measures undertaken to stabilise the slide, including:

. Construction of a toe buttress founded on loose sediments in the reservoir which were stabilised using some 1,500 stone columns (desification resulted in an increase in N from 5 to about 15).

. Installation of 405 80-100m long 100 ton working load ground anchors.

. Tunnel drains which pass through the slide mass into initu rock.

. Unloading of the head of the slope.

Back analysis had shown that the mobilised residual friction of the schist on the sliding surface was 35-36°, consistent with the natural slope angle of 38°.

Dr Deere commented that the hydro scheme was an important power producer and his role as a member of the review panel was to comment on the extent and necessity of remedial works. It was an example of how a secondary feature can grow to dominate a whole project.

In the second part of his talk, Dr Deere discussed the sliding stability of concrete gravity dams. In particular, he discussed the problems associated with weak geological features in foundations such weathered joints, bedding planes or foliation shears, and methods of dealing with these. Dr Deere described the influence of water pressure uplift on stability and drainage measures to control this. He commented that most dams around the world

today are designed for a factor of safety of about 2. The use of shear keys to strengthen weak zones was discussed in relation to Itaipu and Alicura in South America.

A transcript of Dr Deere's talk is available to those interested in further details of this most interesting and technically stimulating address. Dr Deere was accompanied by Dr Lombardi from Switzerland and the audience was treated to some brief comments on dam design problems from him to round off a very successful evening.

D N JENNINGS

AUCKLAND GROUP

The committee has held three meetings during the first five months of 1984, taking advantage of the large pool of expertise in the Auckland region.

In keeping with the current upsurge in interest in mining in New Zealand, our meetings for the first half of the year have all touched on aspects of open cast or underground mining. Two of the activities were joint meetings with the Geological Society of New Zealand, and the Australiasian Institute of Mining and Metallurgy. Much benefit has ensued from discussions between members of the various organisations and we look forward to further joint meetings of a similar nature.

The first meeting on 20 February 1984 was a joint meeting with the NZ Geological Society where Dr Bruce Riddolls of Investigation Geology Ltd and Mr Ken Mills, discussed geotechnical aspects of the Waikato coalfields.

At the second meeting of the year on 17 April 1984, Mr Noel Edkins of ICI Ltd presented an excellent review of the use of explosives in mining and stressed the importance of the geotechnical input into blast design to maximise the effectiveness and to minimise the costs of blasting.

Our third meeting was a joint activity with the Australasian Institute of Mining and Metallurgy and took place on 24 May 1984. A panel of three speakers comprising Mr John Ashby of Applied Geology Associates, Mr Peter McGregor of Beca Carter Hollings & Ferner, and Mr Derek Depledge of NZ State Coal Mines, introduced aspects of geomechanics in opencast and underground mining.

YOLANDA THORP

OTAGO-SOUTHLAND GROUP

A meeting was held on 30 April at which papers were presented on the subject of Land Stability South of Dunedin - Background, Problems and Solutions?

Professor Doug Coombs, Professor of Geology at Otago University, described the Geological History of Dunedin and its environs and the nature of crustal movements and ground formations which cause and are likely to cause problems in the area. Derek Crombie, a consulting engineer for Green Island Borough, spoke on some of the problems in that Borough and described in detail a large slip at Church Hill Road and the steps taken to stabilise it - essentially by deep drainage, with surface protection by way of surface drains and planting. Lynn Sleath of the Ministry of Works and Development District Office, described three particular areas of concern to them in the past south of Green Island - East Taieri, Otakia and Saddle Hill. The first two were major slides essentially stabilised by extensive earthworks. The third, on Saddle Hill, was described by Ross Thurlow, also of the Ministry of Works, who gave a detailed outline of the steps taken to stabilise the slip - a combination of earthworks, deep drainage and fluming away of watercourses running into the area. The work has only recently been done and whether it is successful or not has yet to be determined.

The fifth speaker for the evening was Murray Harris, a Soil Conservator with the Otago Catchment Board, who went through the various procedures and techniques they use to assist farmers in the stabilising of slip areas. Planting of alternative types of trees at different concentrations, seepage drains, contour ditches and field lines, were among some techniques mentioned. He also described their greater involvement with farmers and showed how the incidence of slips in the area had increased over the last 3-4 years.

The meeting was attended by 31 people.

W J HENDERSON

SYMPOSIUM ON ENGINEERING FOR DAMS AND CANALS

Alexandra, 24-27 November 1983

A most successful symposium was held at the D.B. Golden Central Motel Hotel, Alexandra, Central Otago, attended by some 130 delegates from engineering and geological consultants, universities, local authorities and Government departments. The symposium was jointly organised by the New Zealand Geomechanics Society (NZGS) and was designed to improve understanding between geologists and engineers engaged in water retention engineering. The symposium was seen as particularly timely in view of two major failures of hydro-electric projects in the last few years.

The keynote address was given by David Stapledon, Professor of Applied Geology at the South Australian Institute of Technology, who has an International reputation in Engineering Geology and is active in consulting and arbitration work in Australia. His address entitled "Towards Successful Waterworks" provided a valuable summary of many years of experience in investigative engineering, vividly illustrated by case histories drawn from the earlier part of his career in which he gave examples of some of his own misinterpretations based on inexperience. Professor Stapledon contributed much to discussions throughout the conference and the sharing of his experience was deeply appreciated by all delegates.

The first day of papers covered the 'philosophy and methods of investigation'. Papers were presented mainly by geologists who described current investigation techniques and provided case histories illustrating investigations in the very varied types of geological environments found within New Zealand. The papers drew attention in particular, to the various geotechnical hazards inherent in certain rock and soil types and the need for the interdisciplinary team approach to water engineering projects, between geologists and engineers.

The first day was concluded by the symposium dinner, followed by a graphic film on dam safety and concluded by 'off-the-cuff' comments by Professor Stapledon on the work of the New South Wales dam surveillance committee.
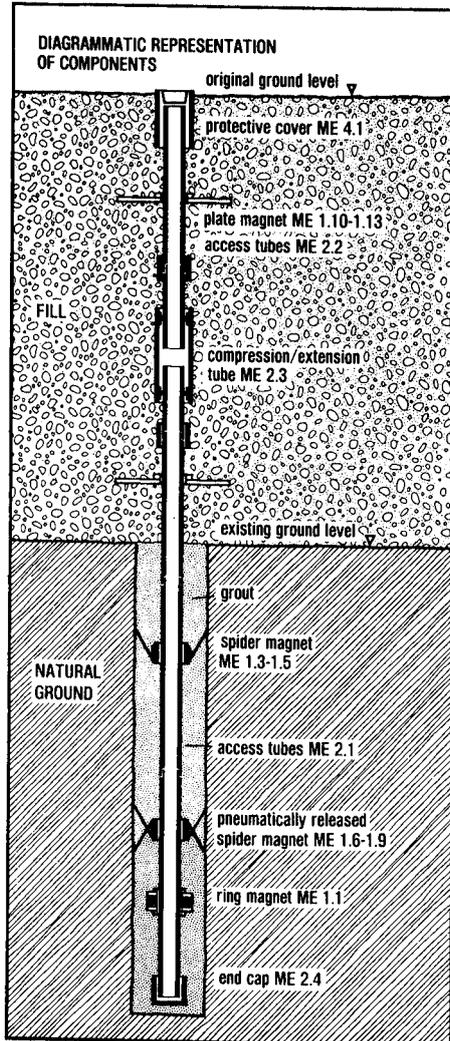
On the second day of the symposium, papers covered examples of design and construction problems and the use of monitoring. The afternoon was occupied by field trips to the Clyde dam site after an introduction to the scheme by MWD project staff. One of the field trips included inspection of underground drives beneath the massive right abutment of the dam, driven to explore narrow shear zones which slope gently below the dam. The evening was taken up by an informal discussion session. Topics discussed included; a proposal to form a data gathering group for volcanic materials in the central North Island, increasing client awareness of investigation costs, soil description codes and safe gate design. All topics provided lively discussion.

On the final day of the symposium a series of short papers was presented by engineers covering preventative engineering. Numerous examples were given of the various engineering hazards associated with dam and water works construction, which proved illuminating to both the geologists and engineers. Following a short panel discussion, the conference was closed at midday on Sunday 27th November.

All who attended the conference no doubt benefited much from the sharing of experiences and knowledge between the two technical groups represented, both from the well presented papers and during the many opportunities for informal discussions. The symposium was most certainly a worthwhile contribution to the campaign for achieving safer dams and has hopefully stimulated greater awareness of the engineering hazards inherent within New Zealands geological environment, parts of which were aptly described as a 'geological rubbish heap' by one of the delegates. The organising committee and all contributors to the symposium are to be heartily congratulated for a well presented and most stimulating conference.

CHRIS GULLIVER

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LETTERS TO THE EDITOR

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

Sir,

I have been invited to join the ISSMFE Technical Committee on Undisturbed Sampling and Laboratory Testing of Soft Rocks and Indurated Soils. The task of this committee is:

1. To make the geomechanics profession aware of the present state-of-the-art practices and associated problems for the undisturbed sampling and laboratory testing of soft rocks and indurated soils.

2. To provide preliminary guidelines and recommendations on the procedures, techniques, equipment and ranges of applicability of a variety of sampling and laboratory testing methods best suited to the range of soft rocks and indurated soils encountered.

3. To identify problem areas and initiate possible directions for future remedies.

A working group in Melbourne, chaired by Ian Johnston, is to act as a co-ordinating body and will prepare the final report to be presented at the XI ISSMFE Conference in San Francisco in May 1985.

To aid discussion, the working party have prepared a draft paper. They would like as much input as possible, but specifically on the following:

(a) Definitions of terminology including classifications.

(b) Undisturbed sampling - because this topic is extremely wide-ranging, we would be most grateful if you and your colleagues could advise us of the techniques that you use, their detailed description, a critical appraisal of their usefulness in conjunction with the material types concerned.

(c) Sample preservation - your experiences of techniques and methods including protection, transportation and storage.

(d) Laboratory testing problems which you have experienced and the methods and techniques you have used to overcome the problems. In particular, your experiences with materials that swell or disintegrate. A consideration of the topics discussed under Section 5 of the draft would be most helpful.

(e) Laboratory testing techniques and your opinion of the existing I.S.R.M. and associated methods as they may or may not apply to soft rocks and indurated soils. Your opinions of the "design" tests would be most appreciated.

(f) Laboratory testing - selection and application to design. A potentially most useful section of the report which requires considerable deliberation and development.

(g) Investigation planning - another potentially invaluable section to provide guidelines linking all aspects, especially sampling and laboratory testing to type of development, range of materials encountered and method of design.

I have contacted a number of people in various organisations for comment, but I am aware that there may well be other people who also have expertise in this field and may wish to make some input. If you wish to make comments and see a copy of the draft would you please contact me as soon as possible. I would like to receive all comments by the end of July 1984 so that they can be collated and passed on to the working group.

Yours faithfully

PETER BARKER

NZ Soil Bureau
Private Bag
Lower Hutt

Sir,

I would like to support the move to change the name of our society to the NZ Geotechnical Engineering Society. There seem to be two reasons for this. Firstly, the term "Geomechanics" is of very restricted usage in a worldwide context, and secondly, the term does not describe the overall scope of the society. As M Pender points out, the term geomechanics logically means the study of the mechanical behaviour of soil, rock, snow, etc. Geomechanics thus forms the theoretical base on which geotechnical engineering operates, but there are many areas of such engineering which are largely outside the field of geomechanics. A particular example of this would be the field of geotechnical processes such as vibrofloatation, dynamic compaction, reinforced earth, lime columns, the use of filter fabrics, etc.

Changing the name from geomechanics to geotechnical is surely emphasising the wide scope of the society and not vice versa as claimed by Mr Galloway. An engineer or technician operating a vibroflot or stressing up ground anchors is not strictly operating in the field of geomechanics, but he is certainly operating in the field of geotechnical engineering.

Yours faithfully

L D WESLEY

Sir,

In a recent letter to the editor (NZ Geomechanics News No 26), B W Riddolls and A J Olsen requested a name change for the Society. As Pender (NZ Geomechanics News No 27) pointed out, there are a number of names that could be adopted. If the reason for a name change is because of the identity problem suggested by Riddolls and Olsen, a change in name may not solve such a problem. It will be difficult to choose a name which adequately covers the wide range of background of our present membership, with the possibility that someone's identity will be affected.

I suspect that the identity problem may exist within minority groups comprising the Society. For example, engineering geologists have long played a secondary role to engineers, and have not been accepted as engineering professionals by IPENZ. Riddolls and Olsen claim that all members of the Society practice in the field of geotechnical engineering, but does this mean that all members are geotechnical engineers? I doubt whether the engineering profession is ready to accept this, although I am aware of consulting work being undertaken by a geologist calling himself a geotechnical engineer. While it is probably clear to most of us that the geologist will not necessarily have the same skills one would expect from an engineer, there is a need to make sure that the public is aware of the distinction.

Perhaps the Society should take a stronger interest in the professional needs of groups such as engineering geologists. Otherwise, it may be necessary for engineering geologists to look to a new organisation; one that would promote public understanding and acceptance of engineering geology and maintain high ethical and professional standards.

Yours faithfully

IAN BROWN

Sir,

I would like to comment on the system for improved description of engineering properties of geological materials as proposed by Messrs Riddolls and Grocott in your issue No.27 (December 1983).

As Riddolls and Grocott point out, some of our most problematic geological materials span, in geotechnical properties, across the interface of soil and rock. For some time there has been difficulties in defining when a soft rock is a soil and vice versa. Terzaghi & Peck (1976) state:-

> "Soil is an agregate of mineral grains that can be separated by such gentle means as agitation in water. Rock, on the other hand, is a natural aggregate of minerals connected by strong and permanent cohesive forces. Since the terms 'strong' and 'permanent' are subject to different interpretations, the boundary between soil and rock is necessarily an arbitary one."

The British Standards Institute (1981) present a widely accepted scale of strength for rocks, based on uniaxial compressive tests.

The Geological Society of London (1972), using the same scale, state that any rock with a uniaxial compressive strength significantly less than 1.25 MN/m^2 should be described and tested as a soil. They suggest that field strength estimation can be done using the point load test and present corresponding point load strengths for the same scale of strength.

However, the use of the point load tester in the field is not all that common yet and the instrument is often unreliable for anisotropic and soft rocks. Therefore, when faced on site with soft geological materials which are near the boundary between soil and rock in terms of geotechnical properties, the field describer may attempt to describe the rock hardness rather than estimate strength (e.g. Ministry of Works and Development). Alternatively, the rock strengths can be estimated using the scheme proposed in the Geological Society of London (1977).

Despite the limitations and difficulties of defining this boundary zone between soil and rock, I still favour retaining the "soil/ rock" concept of description for all geological materials provided an extensive and systematic description of both the material and the material mass is made.

A full systematic description will highlight the problematic characteristics of each material type whether it be Tertiary mudrocks, soft sediments, weathering profiles, volcanic deposits or whatever. Then the identified dominant engineering characteristics, which will vary considerably with different geological materials, can be more fully described. In this way the emphasis is placed on those material characteritics which are likely to be the more significant in terms of the materials engineering performance.

Riddolls and Grocott state:-

"Continued reliance on the "soil/rock" system in geotechnical description can too easily lead to oversimplification, increase the chances of misinterpretation, and so lead to failure."

I disagree. Provided the "soil/rock" description is carried out in a systematic and full manner, using an internationally or New Zealand accepted descriptive system, reliance on a single relative strength scheme as proposed is, in my view, potentially more troublesome. Reasons for this include:

(1) Unless defined at each usage, the proposed terms are bound to cause even more confusion with the already prolific array of internationally published strength terms.

(2) The relative strength of a material will be affected by changes in water content and the occurrence of microfracturing. The proposed definitions do not take these into account.

(3) The proposed scheme is condensed into too few levels and the definitions do overlap. A material described as very weak to weak using the proposed system would vary in strength considerably.

(4) There is an inherent danger that the recipient design engineer of such a relative strength description (or do the proposed terms define disaggregation?) will reply on the concept of the strength of the material rather than focusing on other properties which may affect the

engineering performance of the material more, e.g. slakeability, discontinuities, etc.

Therefore, I would recommend caution before such a new system is adopted. The full and accurate description of geological properties for engineering purposes is often overlooked. There are many varying published descriptive systems, each with their own merits and with the wide variability in New Zealand's geology, it may not be desirable to rigidly 'standardize' a descriptive method or methods. However, I do feel that the system proposed does not add significantly to the existing description systems already in use. The important considerations are that any terms used should be defined (either by reference or in full), and that geological material descriptions be carried out by experienced engineering geologists.

G W BORRIE

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(Accompanying tables omitted - Editor)

DR R D NORTHEY

Dr Roy Northey retired from NZ Soil Bureau, DSIR, after completing 43 years service, on 5 April 1984. The occasion was marked by an informal function at which Roy was presented with a woven wall hanging as a momento of his association with Soil Bureau. A number of speakers made reference to Roy's work and high reputation in soil engineering in New Zealand and overseas.

Roy joined NZ Soil Bureau as a professional cadet in 1941, completed BSc and MSc degrees at Victoria University College by part-time study, and in 1947 was awarded a National Research Scholarship. This made it possible for him to study at Imperial College, University of London, for the degree of PhD. His work there with A W Skempton laid the foundation for an understanding of the sensitivity of fine-grained soils which remains valid to this day. On his return to New Zealand in 1950, Roy assumed leadership of the Soil Engineering Section of Soil Bureau. He held this position until 1981, when he was appointed Deputy Director.

Roy Northey's first encounter with soil engineering took place during the Second World War when Soil Bureau was testing soil-cement mixtures to assess their suitability for airfield construction. After his return from Imperial College he introduced new methods of sampling, testing and analysis of soils based on current British practice. Over the next ten years these were continually improved, and formed the basis for a soil engineering sampling and testing service which was used extensively by other government agencies, local bodies, consulting engineers and contractors. Roy, himself, rapidly established a reputation for knowledge, practical skill, experience and judgement in soil engineering, that he became the ultimate resource person in New Zealand for a generation of engineers.

Roy Northey has made significant contributions to sampling and testing procedures, the relationship between soils and the effects of earthquakes, slope stability, the engineering properties of soils which may be inferred from their pedological classifications and to the assessment of engineering risk. He has taken part in investigations of notable and catastrophic failures of slopes, embankments and foundations. In these investigations his observation, knowledge, analysis and reasoning have often been conclusive in establishing causes and identifying remedial action. He has published over 40 papers in national and international journals and proceedings. Roy has a very extensive knowledge of international literature, and has been able to draw on this to assist him in investigations. He has the ability to see the wider implications of situations and to suggest creative solutions to problems.

Roy's reputation extends beyond New Zealand. He has been asked to provide expert opinion and give evidence in tribunal hearings, both here and overseas. He has been an invited participant in a number of international conferences, and he is currently Australasian Vice-President of the International Society for Soil Mechanics and Foundation Engineering.

SOIL ENGINEERING AT NZ SOIL BUREAU, DSIR

For more than 30 years from about 1950, NZ Soil Bureau established a considerable resource of equipment and expertise for soil sampling and testing for engineering purposes. These were used extensively by government agencies, local bodies and consulting engineers. When Dr Roy Northey became Deputy Director of NZ Soil Bureau, these resources were combined with those of a group working in soil physics to form the Engineering and Physics Section. The combined group is carrying out a number of projects including several related to the restoration of land following mining or other disturbance. However, the specialised soil sampling equipment and testing procedures are available under appropriate conditions to organisations and individuals who may require them.

The range of equipment and testing procedures available, and a brief history of Soil Bureau's soil engineering activity, are outlined in Soil Bureau Labortory Report EP2A, available from NZ Soil Bureau, DSIR, Private Bag, Lower Hutt.
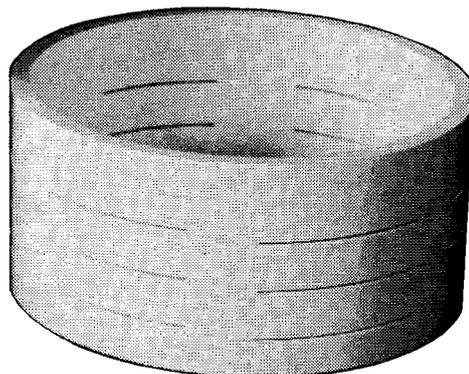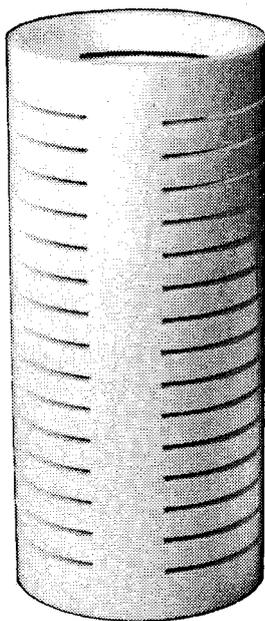
APPLICATION FOR MEMBERSHIP

OF

New Zealand Geomechanics Society
A TECHNICAL GROUP OF THE NEW ZEALAND INSTITUTION OF PROFESSIONAL ENGINEERS NEW ZEALAND

The Secretary,
Instituion of Professional Engineers N.Z.,
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 ($10.00)
International Society for Rock Mechanics	(ISRM	Yes/No ($13.00)
International Association of Engineering Geology	(IAEG)	Yes/No ($6.00)
		$16.00 with Bulletin)

Signature of Applicant _____
                            Date _____ 19_____

N.B.: Affiliation fees are in addition to the basic Geomechanics Society member-ship fee of $20.00. Please do not send fees with this application. An account will be rendered on your acceptance into the Society.

Nomination:

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

Signed _____ Date_____19____

NEW ZEALAND GEOMECHANICS SOCIETY

NOTIFICATION OF CHANGE OF ADDRESS

The Secretary,
The Institution of Professional Engineers New Zealand,
P.O. Box 12241,
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 _____