NZIE

THE NEW ZEALAND INSTITUTION OF ENGINEERS

(Incorporated)

PROCEEDINGS OF TECHNICAL GROUPS

VOLUME 1
Issue 5 (G)

THE NEW ZEALAND GEOMECHANICS SOCIETY

Proceedings of the Symposium

on

STABILITY OF SLOPES IN NATURAL GROUND

Nelson, November, 1974

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PROCEEDINGS OF THE SYMPOSIUM ON STABILITY OF SLOPES IN NATURAL GROUND

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- Symposium on -

STABILITY OF SLOPES IN NATURAL GROUND

The Symposium took place in Nelson, on 8th and 9th November, 1974 at the Rutherford Hotel, and was attended by 120 participants. Included in the Proceedings are the papers from invited authors, together with reports of the discussion. Thanks are due to Mrs. D. Cryer for her transcription from the tape record.

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SYMPOSIUM ON STABILITY OF SLOPES IN NATURAL GROUND

- NELSON -

November, 1974

OPENING SESSION

The Mayor of Nelson, *Mr. R.A. McLennan*, in officially declaring the Symposium open, welcomed all present to the city. He felt sure that those attending would be able to show the way to develop land and provide the standards to bring this land to the use for which it was required.

Mr. D.K. Taylor, Chairman of the N.Z. Geomechanics Society and Convener of the Symposium Organising Committee, added his words of welcome to those of the Mayor, and also thanked the Mayor for his address. Speaking to Keynote Address (see p.1.1) Mr. Taylor said that the N.Z. Geomechanics Society was a technical group of the N.Z. Institution of Engineers and included in its membership Engineers, Scientists, Geologists, Technicians, Contractors - in fact anyone who had an interest in the mechanics of the behaviour of the earth's crust as it affected man's efforts to use it. This was the fourth of the Symposia organised by the Society to study a specialised aspect of engineering. In some of the previous symposia he had felt that there had been a tendency to a rather introverted discussion amongst those present - preaching to the converted perhaps - but this time the audience was the largest and most diverse there had been. From it there would surely be a discussion ranging widely from the more specific to the most general and least technical aspects of the tendency of steep ground to become flatter. Without exception, the papers which had been written for this symposium were of a high standard. The topics and authors had been carefully chosen to cover the significant factors which affected the stability of slopes in natural ground, as the organising committee had seen them. Each participant, he said would see the problem in a different light. Illustrating his remarks with slides he said that his own belief was that any picture of land stability should first be viewed from a distance at which the broad geological and geomorphological form of the landscape could be seen objectively. If this was done then one would not so often be surprised and dismayed by a particular occurrence of landslip. Examination should then sharpen its focus on progressively more localised factors finally attempting a quantitative assessment of stability based upon the mechanical behaviour of very small and carefully selected pieces of ground. He stressed attempting a quantitative assessment because he thought it must be admitted that there were very severe limitations upon the validity of such an assessment, and one would be doing a disservice if one forgot those limitations. While all this general and specific viewing was being done, somebody was waiting for the answer - or perhaps just getting on with the use of land without waiting for an answer. Life went on; people needed houses, roads, harbours, power supply, and there would always be some risk in using steep ground for such purposes. If it was felt that all the risks should be removed then nothing would be built. In considering the consequences of landslip, one could think of three very broad types of situation in each of which the approach to an appraisal could have a different emphasis:-

1. A large aggregate capital investment spread over a large area. Here the accent might be upon a broad regional study, leading to a fairly conservative judgement backed by some form of community insurance.

2. A large capital investment concentrated in a small area. In this case the cost of a detailed investigation and quantitative analysis is warranted.

3. Quantitatively small, but personally vital, private investment in a specific area - the average house owner for example. The cost of detailed assessment in these cases is prohibitive and the consequences of misjudgement can be tragic. The most conservative judgement is required.

Mr. Taylor then postulated some general questions to which he hoped to have answers by the end of the symposium.

(a) Was the "cost" of landslips to the community such that there was a serious problem - or were people preoccupied with this aspect of their own speciality?

(b) Were they considering the possibility of landslip at the right time in relation to the commencement of planning and construction?

(c) Had they a balanced view of the importance of landslip in relation to the consequences in differing circumstances?

(d) Were they using the best available techniques which they could afford, to forecast the likelihood of landslip?

(e) Were new or modified administrative or legislative processes necessary to reduce the incidence of damage to property?

(Attempts to answer some of these questions are given in the closing session - Ed.)
In conclusion, as it was a very full programme and he suspected they would be short of time, he thanked members for attending, the authors of the papers, his own organising committee and particularly Mr. Bruce Hands and his Nelson committee for the work put into the symposium.

Professor P.W. Taylor, then addressed the meeting. He said it gave him great pleasure to be in attendance in the role of Vice-President of the International Society of Soil Mechanics and Foundation Engineering. He extended the good wishes of the International body for an interesting and fruitful seminar. The N.Z. Geomechanics Society had only about 250 members throughout the country and almost half that number were present. This provided testimony to the wisdom and foresight of the Geomechanics Society Management Committee that it should select a Symposium topic of such great interest, such a suitable time and such a beautiful city as Nelson in which to hold it. He said that in this age of technological advance, disasters such as landslides, rather than being considered acts of God, were now more often than not considered to be negligence of the engineer. Such was the price paid for claims to expertise. He said his main task as editor of the Proceedings was to collect an adequate record of the discussion. He hoped there would be a frank and open discussion by comparing experiences, not necessarily of the most successful jobs, but of those which had caused most concern.

THE STABILITY OF SLOPES IN NATURAL GROUND

D.K. TAYLOR

Chairman, N.Z. Geomechanics Society

The object of this symposium is to review our capability, in New Zealand, to evaluate the stability of natural ground with all its heterogeneous variations, so far as that stability affects the design construction and maintenance of civil engineering works and urban development. Dominant in this review is the assessment of stability in advance of construction and development, although we must consider the precautions to be taken in modifying those slopes and the means of arresting movement after it has started.

In the course of development, slopes often are altered by excavation and by filling. We will not dwell upon the techniques of computing the internal stability of bodies of filling which we may construct to standards which are within our control.

Vegetation plays a vital part in the stability of natural slopes but we do not intend to consider as a separate topic, the stability of land used for agriculture, pasture or forest.

In most of New Zealand we live in a relatively steep landscape, subject to periodic earthquakes and high rainfall intensity. From our pioneering days we have accepted, for obvious economic reasons, all the risks that lie in constructing our communication routes and our towns in that topography; we are quite used to clearing up landslips after they occur and this, I believe, has reduced our awareness of signs of potential instability which are there to be read.

Those risks which have been acceptable in the past have now to be viewed in a changing context of more intensive development and larger capital investment. We can afford to be more cautious.

In the open countryside our pastoral economy and our waterways are damaged by erosion and we are more aware of the eyesores which are generated.

Our communication routes are built now to a much higher standard at vastly greater cost and disruption of traffic or loss of life become progressivley less tolerable. Our economic progress and physical comfort depends more and more upon the generation and transmission of power without interruption from, amongst other causes, collapsing land slopes.

It is in our urban areas, however, where the impact of land slope failures bears most directly upon the private citizen. For most people their greatest, perhaps only, material asset lies in their house, its contents and the land upon which they stand. Their reserves of capital usually are not enough to pay for fixing the sort of damage that results from ground movements on a slope. As our towns and cities grow, flat land is scarcer, the standard and cost of services rises, and residential subdivisions are pushing into steeper and steeper ground; a progressive process which dulls our appreciation of the risks.

Having made arguments for conservation I consider it essential to throw some weight onto the other side of the scales. As well as being very conscious of the consequences of over-estimating the stability of a slope, we must realise that we will get no thanks for playing it too safe. It is not our function as professional advisers to play the role of saviours of the human race and we should be very careful that legislative processes do not force this role upon us! The human race cannot exist without taking risks - after all, most of us drive motorcars. Asking one of us for an assurance that a piece of sloping ground will not become flatter is like asking a doctor of medicine for a certificate that we will not die - the only question in both cases is "when".

Under the following general headings, the papers which will be presented in this symposium will deal with the intensely practical questions of how we should make use of sloping ground.

RISKS, LEGALITIES AND INSURANCE

In face of the many pressures to use steeper ground, who should be responsible for decisions which must always contain an element of risk which is hard to quantify; what are the consequences of making a wrong decision; at what stage should the residual risk be covered by an insurance fund, and how should that fund be built up?

CLASSIFICATION AND MECHANISMS OF SLOPE FAILURE

No classification will suit every point of view, but it is useful at the outset to list the fundamental causes of slope failure so that we can recognise better the circumstances in which they occur; decide the best means of remedying failure; and, to some extent, use a common terminology in our discussions.

GEOLOGICAL ASSESSMENT

In examining land before we use it we must, in my opinion, start with the Geologist's broadly based view of it. All too often laymen and even civil engineers look only at the small area which is of immediate concern to them. The landscape is not made up of separate small pieces.

A study of the lithology and structures of the large masses of underlying rocks and of the processes which form the landscape upon them, leads to an explanation of the shape of the ground, points to areas which will be less stable than others, and gives relevance to any more detailed examination which may follow. Without an appreciation of regional differences of geology we may try to apply standards which are safe in one region, to another area in which they are not safe. Contrast similar angles of slope in Wellington and in Auckland, for example.

SOIL SLOPES - ENGINEERING ASSESSMENT OF STABILITY

Quite sophisticated methods of sampling, testing and quantitative analysis of failure mechanism have been developed in the science of "Soil Mechanics". In this context "soil" has come to mean fine-grained sediments, products of residual weathering and even rocks, in which the discontinuities are effectively encompassed in samples up to about 4 inches in diameter.

These sophisticated methods are valid only so long as the samples represent the whole mass of ground being considered. In most cases of natural ground this is a very severe limitation and a valid quantitative analysis by these methods often may become very expensive or even impossible.

Nevertheless, the studies of "soil mechanics" have provided valuable insight into the mechanisms of slope failure, and no consideration of land stability would be complete withot reference to those studies.

ROCK SLOPES - ASSESSMENT OF STABILITY

The much more recently developing science of "Rock Mechanics" takes over from "Soil Mechanics" where the discontinuities in the ground cannot effectively be ecompassed within samples of manageable size and the study is concentrated upon the forces acting upon discontinuities.

Many cases of slope failure patently involve sliding on existing rock discontinuities whose properties are very different from those of the parent rock.

Subdivision into these groups is artificial of course; any study of one overlaps into the other and no more so than in the matter of landslope stability. This is why our society is named "Geomechanics" and seeks to provide a vehicle for integration.

STABILISATION OF SLOPES

Under the above headings of Soil and Rock, we will have considered already the sort of things one should or should not do in modifying a natural slope by construction or development.

In this section we will examine the more positive actions which can be taken to stabilise a slope either as

A PRECAUTION, or

A REMEDY.

Surface protection can often be done cheaply but when it comes to ground improvement (by grouting for example) or to retaining structures, the masses of ground involved are commonly so large that the costs are prohibitive except in relation to works to be protected which have a very large and concentrated capital investment in them.

The difficulties of providing valid <u>quantitative</u> evaluations of stability also bear upon the cost which can be justified for stabilising works. Finally, we came back to specific practical problems. Who suffers if our assessment is wrong, what level of responsibility should we accept as professional advisers, who has the necessary authority to allow or forbid use of a particular piece of land, how much assurance should our clients expect - in short, how should we advise them?

* * * * * * * *

RISKS, LEGALITIES AND INSURANCE OF SLOPE STABILITY

J.L. Gill

1.0 INTRODUCTION

This paper is intended to outline some of the consequences of slope failure as affecting residential sections, and to briefly set out how the Earthquake & War Damage Commission has become actively involved with land instability problems since automatic landslip insurance was introduced under the Earthquake & War Damage Act, 1944.

This landslip legislation was designed to assist homeowners when dwellings are damaged by land failures not within the reasonable control of homeowners to prevent. It was not intended cover should extend to relieve homeowners of instability problems affecting sections.

2.0 HISTORY OF INSURANCE UNDER THE EARTHQUAKE & WAR DAMAGE ACT 1944.

During the second world war there was threat of invasion of this country and a war damage insurance scheme was introduced under the War Damage Act 1941. This scheme was essentially to compensate for tangible property losses arising from the activities of war, whether by enemy action, allied action in defence or as a result of measures to mitigate the spread of hostilities in this country.

The scheme was very simple. A war damage premium of 5/- was levied on each £100 of cover under contracts of fire insurance and payable by the fire insurer into a war damage fund established under the War Damage Act. Thus a fire insurance policy also attracted automatic insurance against war damage.

Basically the function of the War Damage Commission was to receive premiums collected by insurers and to determine claims. No record of insurance was maintained by the Commission, the fire insurer furnishing all necessary information on request at the time a claim was made.

2.1 Earthquake Insurance: For some years there had been pressure to establish a fund to compensate property owners for earthquake damage. In 1944 when the end of hostilities was in sight the Earthquake & War Damage Act was introduced repealing the War Damage Act 1941 and transferring the then balance of some $8 million in the war damage fund to a new fund for earthquake shock and earthquake fire damage in addition to war damage. The principle of the new scheme remained the same, but the premium levy was reduced from 5/- to 1/- for each £100 of fire cover.

2.2 Subsequent extensions to cover: In 1949 a new fund, the disaster fund was established within the earthquake and war damage fund to provide cover against extraordinary disaster damage, defined as:

> "Damage occurring as the direct result of storm, flood or volcanic eruption (excluding damage caused by landslip, subsidence of earth or rock, or erosion by the sea) where the storm, flood, or volcanic eruption is of an abnormal and unforeseen nature and is of extraordinary effect."

No extra premium was levied for this extension of cover, but 1/10th of future premium income was merely diverted from the earthquake fund into the new disaster fund. In 1970, again without any increase in premium, cover under the Act was further extended to permit payments from the disaster fund for damage caused by landslip.

2.3 Voluntary Insurance Provisions: In 1954 the Commission was permitted to underwrite landslip insurance on a voluntary basis. Each risk was inspected and a premium commensurate with the landslip hazard was fixed. Since 1967 the Commission has also been able to accept geothermal activity insurance on a voluntary basis.

2.4 Accumulated Fund: The earthquake fund from which earthquake and war damage losses are met currently stands at $185 million. Since inception in 1944 nearly 35,000 claims for earthquake damage have been made on the Commission with payments under claims of less than $5 million.

The disaster fund which provides for extraordinary storm, flood and volcanic eruption damage and landslip damage losses stands at only $3.5 million with payments since 1949 totalling $9.5 million under some 42,000 claims.

3.0 LANDSLIP INSURANCE AMENDING REGULATIONS

Amending regulations to provide automatic landslip insurance cover under the Earthquake & War Damage Act became effective from 16 July, 1970. These regulations were designed to provide cover on a basis very similar to that which for some years had been provided for earthquake damage and extraordinary disaster damage.

3.1 Definition of Landslip Damage: For the purpose of these regulations landslip is defined as meaning:

> "Subsidence of a substantial land mass other than by settlement, soil shrinkage or compaction; and including the movement from any hill, mound, bank, slope, cliff or face of earth or rock of a substantial mass of earth or rock which before movement formed an integral part of the hill, mound, bank, slope, cliff or face".

The regulations further provide that in determining whether any damage is landslip damage the Commission shall have regard to the following matters:

"(a) In the case of damage to any building, or to the contents of any building, whether the building complied with the requirements of any applicable New Zealand standard model building-bylaw relating to foundations declared or continuing in force under the Standards Act 1965:

(b) Whether the basic principles of site investigation and foundation design have been observed, and the construction of foundations and earthworks have been property supervised;

(c) The standard of repair and maintenance of the insured property or of any building containing the insured property;

(d) Any neglect or carelessness of the insured person;

(e) Any other matter of any kind whatsoever that the Commission considers relevant in the circumstances of the particular case."

3.2 Scope of Cover: Reading the landslip amending regulations in concert with Section 14 of the Act and the principle regulations thereunder, only such property actually insured against damage by fire is deemed to be insured for damage caused by landslip. A minimum franchise of $200 applies and only damage to an insured building in excess of this statutory minimum franchise can fall as a liability on the Commission. The landslip provisions do not extend to cover a property owner against loss of land, damage to paths, fences or retaining walls, or for any part of the cost of stabilising the land or resiting a building. The cost of clearing landslip debris is covered only to the extent that clearing of debris is essential to enable landslip damage to an insured building to be repaired.

The Commission is permitted to accept voluntary landslip insurance cover on property not covered under the automatic provisions. Voluntary cover is, however, selectively underwritten and has been limited to such items as engineer designed retaining walls, water reservoirs or swimming pools.

Landslip insurance is not underwritten by the insurance industry in New Zealand, but a form of cover is provided under the Contractors' All Risks type of policy issued for large contract works. To what extent the overall premium is loaded for the landslip risk is not defined.

4.0 FUNCTION OF THE EARTHQUAKE & WAR DAMAGE COMMISSION

Legislation charges the Commission with the duty of executing the provisions of the Act and with exercising such other functions as are conferred upon it by the Act or any regulations thereunder. The Commission cannot exceed this authority and is given no discretionary powers which would permit assisting a property owner with site works and other measures to preserve an insured building from damage notwithstanding such work could have the ultimate effect of relieving the Commission of a substantial liability at some future time.

Cost of remedial or preventive measures is not a function of an insurer who normally is under liability only when damage to the insured item actually occurs. The responsibility rests on the insured to take all reasonable steps to preserve insured property from damage. However, the insured is only expected to do what can reasonably be expected of him and in many cases of slope failure on residential sections this is limited. The cost of site works could be well beyond the owners means or it could be that the threat arises from neighbouring properties. Thus in many cases the affected property owner is forced to live with his problem and the Commission to carry the risk.

5.0 SUSCEPTIBILITY TO LANDSLIP The regulations provide safeguards whereby the Commission can be relieved of liability in certain circumstances but it was not the intention homeowners with no control over instability problems be adversely selected against and denied a right to landslip insurance protection.

5.1 Safeguards under Claims: The various factors the Commission is required to take into consideration when determining a claim (para.3.1) are designed to ensure there is not a reduction in the standard of care necessary to minimise the risk of landslip on residential lots. Cover was not to be available to those in a position to avoid or prevent landslip damage. A homeowner who had no control over developing the site or building the dwelling is not denied indemnity for failure due to earthworks or foundation inadequacies. The success of a claim could however, be prejudiced if it is found landslip resulted from some neglect or injudicious act by the homeowner.

5.2. Classification of Property: The regulations provide also that for the purpose of a claim the insured person shall carry one percent of the amount of the insured loss or damage as a franchise with a minimum of $200. After causing a survey to be made the Commission may classify property according to susceptibility todamage from landslip thus increasing the amount of franchise to be borne by the claimant. Normally property is classified with an increased franchise only after a loss has occurred and it is found damage would have been avoided had preventive measures within the control of the insured been implemented. In such a case the Commission would classify until preventive measures such as retaining a cut, planting a slope or providing for safe disposal of stormwater had been carried out. Where preventive measures are not within the control of the homeowner concerned, classification would seldom be considered.

5.3 Cancellation of Cover: Landslip insurance can be cancelled by the Commission in its discretion at any time. The normal policy of the Commission is to contract out of further Act liability in respect of a particular site only after a total loss settlement has been made and there is evidence to believe the site is no longer suitable for housing.

6.0 LANDSLIP CLAIMS EXPERIENCE

In four years since automatic landslip insurance was introduced more than 700 claims for landslip damage on residential sections have been recorded. A number of these have not been sustained as a claim in terms of legislation, either because damage was not the result of landslip as defined, the dwelling itself was not damaged or damage to the dwelling did not exceed the franchise.

The cost of these claims including an estimate for liability still outstanding will be in excess of $500,000. Although only 23 claims involved dwellings which were a material total loss, in many other cases cost of repair has exceeded $5,000.

With the nature of movement affecting some sections recurrence of damage is inevitable, but on completion of repairs the Commission has continued to carry the risk on normal terms and without any special classification. A few properties have already been the subject of more than one claim.

7.0 ADEQUACY OF PRESENT LANDSLIP PROVISIONS

There has already been pressure to extend the landslip provisions under the Act and so reduce the uninsured burden on homeowners that inevitably arises following landslip. Some sectors believe the Commission should meet the cost of measures necessary to preserve insured property from damage. There have also been suggestions loss of use and loss in re-sale value should be compensated.

The Commission is not anxious to become involved in remedial or preventive measures and feels that any form of consequential loss should not be met from a fund subsidised by a compulsory levy on all property owners.

Very few homeowners to-day have any control over selection and development of building sites and should not have to bear the cost of hidden defects or inadaquacies. The cost of restoring sections in the Wellington area damaged by landslip early this year has been estimated at about $10 million. Newer sub-divisions suffered considerably and in nearly every case damage would have been avoided had better development standards been enforced.

If property owners should not have to bear this loss, it must then fall on those responsible. For this type of protection to come from the Commission would mean changing the whole concept of cover under the Act. Homeowners also face a considerable uninsured loss following earthquake or extraordinary flood and compounded by the aggregation of homeowners so affected by these events a much more deserving case is presented than the few affected by landslip. If cover is to be widened such additional benefits would need to apply to Act cover as a whole and to meet this need a vast fund would be necessary with a substantial increase in premium for all property owners.

It would not appear unreasonable for the Commission to be granted some discretionary power in certain circumstances to carry out emergency measures where property is placed in dire peril of damage and this would remove some of the frustration now being experienced under landslip claims. Exercising discretion in administering legislation can prove dangerous however as invariably small inconsistencies creep in and in time are regarded as the law itself.

The Commission would prefer the present concept of cover under the Act to be retained but some attempt be made to help the homeowner by attacking the landslip problems at the very roots by enforcing more realistic sub-division standards with perhaps absolute liability on those responsible when failures do occur.

8.0 LEGAL RESPONSIBILITY

With greater use of hillside areas for housing development stability problems are inevitable but the incidence of damage on new sub-divisions makes it evident local authority standards in some areas are far from adequate.

A recent survey over 100 consecutive claims showed that 55 dwellings affected had been erected within the past 10 years and of these 32 had been built since landslip insurance was introduced. A matter of some concern is that 17 were less than 12 months old and 5 actually still in course of erection. One dwelling ($22,000) occupied only 3 months and another ($30,000) nearing completion were total losses. This suggests standards for hillside sub-divisions have been relaxed now that automatic landslip insurance protection is available.

8.1 Subrogation of Rights: Circumstances giving rise to damage on some sections could provide good grounds for action against some other party, but few homeowners have the resources to take action to seek redress. Where the Commission is under a liability the rights of the homeowner to any relief or indemnity from some other party are subrogated and legislation makes it obligatory for the Commission to exercise any such subrogated right of action.

8.2 Local Authorities: Control over housing sub-divisions is vested in local authorities and by-laws dictate standards for earthworks and buildings. The prime responsibility must rest with local authorities to ensure not only these standards are adequate, but also complied with.

Ordinance V of the Town & Country Planning Regulations 1960 requires local authorities to have regard to several factors, including landslip, when zoning areas for housing and also when placing a building on a site. To the average homeowner local authority compliance with this Ordinance would be a tacit guarantee against instability problems on new sub-divisions and if a failure did occur the local authority would be to blame.

8.3 Land Developers: A land developer will do only such work on a proposed new sub-division as is required by the local authority and a certificate of compliance would satisfy these obligations to the local authority. It is problematical whether short comings in local authority standards will reflect vicariously on a developer but if earthworks carried out do not conform with the certificate of compliance the developer or perhaps his professional advisers could be held responsible. It would appear a professional adviser to the developer could be responsible only in a case where absolute control over all aspects of development of an area was vested in the professional adviser.

8.4. Builders: Well placed fill is frequently cut about to suit a split level type of building and invariably steep unretained cuts are left beneath and adjacent to the structure being erected. A few claims from builders for failure of such cuts during course of erection have been declined. A builder of a spec house could be liable to a subsequent owner for damage from collapse of a hidden unretained cut.

8.5 Neighbours: A property owner could be liable to a neighbour if through an act of neglect or carelessness a neighbour's property is damaged by landslip. This could arise through failure of earthworks being carried out or lack of proper stormwater control. Some property owners have indemnity for such liability under a homeowners insurance policy, but many insurers specifically exclude the landslip risk from public liability contingencies under this type of policy.

9.0 CONCLUSION Landslip damage on residential sections can be avoided. The homeowner should not have to suffer for an instability problem not of his own making. From reports furnished to the Commission following investigation of claims almost without exception someone somewhere down the line has done something that should not have been done or, has failed to do something that should have been done.

9.1 Local Authority Responsibility: Many of today's problems on residential sections could have been prevented had greater control over hillside developments been exercised at local authority level. A local authority is vested with the power and holds the only key to the long-term stability of future developments for housing. It is charged with the welfare of the people and it would appear to follow that the local authority has a moral obligation to the homeowner if not a legal liability to make good damage on any section it has approved for home building.

9.2 Local Community Involvement: If accepted that local instability problems should be resolved at local community level, some local community scheme must operate and funding of such a scheme should fall only on those within that community who could be affected by landslip. It is suggested each local authority administer a local fund from which local homeowners can be reimbursed for the cost of repairing landslip damage on sections or to meet the cost of any preventive measures which may prove necessary after development.

9.3 Finance: Not all local authorities will need a local scheme. Some will need only a small fund and others may need a much larger reserve. Each local fund could be financed by a nominal levy on developers for each new section approved for housing, supplemented by a nominal annual rating based on the unimproved value of only those sections in such areas to be zoned by each local authority as liable for the levy. The amount of this levy can vary according to the drains on the fund. The demands made on a local fund must eventually be controlled by the development standards demanded by that local authority and the control it maintains over all facets of development and the subsequent long-term housekeeping on each developed section.

It would also appear reasonable there should be some provisions for contribution by the Earthquake and War Damage Commission where work carried out with the approval of the Commission will mitigate the Commission's loss by saving an insured building in dire peril from damage or further damage as a result of landslip on the section.

CLASSIFICATION AND MECHANISMS OF SLOPE FAILURES IN NATURAL GROUND

R.D. Northey, J.G. Hawley and P.R. Barker

"Classification is the arrangement of things in classes according to characteristics that they have in common .... Classification may also occur when the only act of arrangement done is the giving of a common name to things of the same kind." (Encyclopaedia Brittanica)

INTRODUCTION

There is a great need for a common language of classification of slope failures in natural ground. This need is felt not only in communication within the geotechnical profession but also in dealing with others particularly the general public and the law. Within the profession the principal purpose of classification must be to provide an adequate framework about which we can accumulate common experience for use in controlling or avoiding instability of slopes.

The aim of the first part of the paper is to draw up a recommended classification system and to present a glossary of the terms used in it. The second part consists of a checklist of various factors which may contribute to slope failure. The paper has been deliberately kept short, confined to the setting down of a common terminology to be used at the symposium. Its purpose is one of listing and brief description rather than analysis or explanation.

CLASSIFICATIONS

Many systems of classification have been put forward by various authors though more recently there has been a fairly general tendency towards a consistent terminology. From a review of past systems, Sharpe (1938) put forward a basic system of relating the many types of slope failure to one another, with rate of movement and water content as important criteria. This recognition of the essential continuum of natural phenomena so that any classification in some degree imposes arbitrary boundaries, is important since in many cases it is difficult to fit a particular occurrence to a particular category. Intergrades will always occur and a particular earth movement may partake of aspects of different classes at various times during its development.

Carson & Kirkby (1972) also use speed and water content as controlling variables in their study of mass movement as a rate process. This system has the merit of uniting what superficially appear to be quite different phenomena in a simple comprehensive manner, but does not provide the common terminology.

Varnes (1938) classified landslides into categories of falls, slides and flows, within which he considered the shape of the failure surface, the type of material, speed of movement, and the degree of internal deformation within the moving mass. This system has been widely used but has been superseded in some degree by that of Hutchinson (1968) upon which the system advocated in this paper is based. Modifications have been made to exclude the essentially stable geomorphological features, at least to the bulk of engineering works, and to deal more fully with types of failure which are of particular concern in New Zealand. The category of creeping earthflow as introduced by Campbell (1951) to describe many failures in mudstone country of the central North Island, Hawkes Bay, Gisborne and North Canterbury has been included, together with a further subdivision of Hutchinson's climatic mudflows (Hutchinson 1968) as suggested by Hutchinson & Bhandari (1971).

RECOMMENDED CLASSIFICATION AND GLOSSARY OF TERMS

The recommended classification is as shown in Table 1 (after Hutchinson 1968), and the various categories described below in note form.

TABLE 1

RECOMMENDED CLASSIFICATION OF SLOPE FAILURE

(after Hutchinson 1968)

CREEP	(1) Shallow, predominantly seasonal creep;
	(a) Soil creep
	(b) Talus creep
	(2) Deep-seated continuous creep; mass creep
	(3) Progressive creep
FROZEN GROUND PHENOMENA	(4) Freeze-thaw movements
	(a) Solifluction
	(b) Rock glaciers
LANDSLIDES	(5) Translational slides
	(a) Creeping earthflow
	(b) Rock slides; block glides
	(c) Slab, or flake slides
	(d) Detritus, or debris slides
	(e) Mudflows
	(i) Mudslides
	(ii) Mudspates
	(iii) Volcanic mudflows - lahars
	(f) Bog flows; bog bursts
	(g) Flow failures
	(i) Loess flows
	(ii) Flow slides
	(6) Rotational slips
	(a) Single rotational slips
	(b) Multiple rotational slips
	(i) In stiff, fissured clays
	(ii) In soft, extra-sensitive clays; clay flows
	(d) Successive, or stepped rotational slips
	(7) Falls
	(a) Stone and boulder falls
	(b) Rock and soil falls
	(8) Sub-aqueous slides
	(a) Flow slides
	(b) Under-consolidated clay slides

CREEP

Used to describe the very slow permanent deformation of a slope regardless of cause or mechanism involved.

(1) Shallow creep: Largely confined to the weathered surface zone of fluctuating temperature and water content: Speed increases with increase in variations in temperature and water content

(1a) Soil creep: cohesive shallow creep

(1b) Talus creep: frictional shallow creep

(2) Deep-seated creep: Sometimes called mass creep; much slower than soil or talus creep; due to stresses in rock and soil masses at depth, which in turn are to be expected to arise from temperature changes with depth and with time, and tectonic movements. (There is as yet scarcely any direct field evidence for its existence).

(3) Progressive creep: Occurs in slopes which are approaching failure. The creep movements work-soften the materials (such as overconsolidated clays). Rates of movement are liable to increase dramatically, leading to landslides (see below).

FROZEN GROUND PHENOMENA

(4) Freeze-thaw movements

(4a) Solifluction: slow downslope movement of surface material under the influence of freeze-thaw processes. One of the main agents of denudation in periglacial environments (such as the 'high country' in N.Z.): varies between widespread creep-like movements and more active local and faster forms which grade into mudflows: most active when shallow thawing produces saturation of a layer above a frozen subsoil.

(4b) Rock glaciers: glaciers with much more rock than ice in them. Ice may be limited to intersticial pockets at depth. May exceed 1 km in length and 30 m in depth. Lower reaches may have lobes on surface. Occur in currently glaciated terrain.

LANDSLIDES

Relatively rapid movements involving failure. In contrast with creep movements in which there is generally a continuous gradation between the stationary and moving material, the movement in landslides take place characteristically on one or more discreet surfaces.

(5) Translational slides: Involve failure on a surface roughly parallel to the surface slope. Sliding mass is usually shallow (less than 10% of the length of the slide). Usually occur in frictional material in which deeper movements are inhibited by frictional strength increasing with depth. Also occur in cohesive soils (see creeping earthflow) where some plane of weakness defines a surface of sliding.

(5a) Creeping earthflow: slow or intermittently rapid: often occurs over the full length of comparatively gentle slopes. The appearance is hummocky and often creviced, with slipping at the upper margin, pools in the depressions and pressure ridges at the sides and base of the flow. Occur after heavy rains and frequently involve a complete catchment. Gullying can develop if the drainage pattern becomes defined - as commonly happens in the Gisborne district. On bentonitic clays particularly mobile earthflows develop.

(5b) Rock slides or Block glides: rock mass moves as one or breaks up to produce a multiple failure: generally fairly rapid: slip surface commonly formed by bedding cleavage or joint plane, which is frequently occupied by an argillaceous filling.

(5c) Slab or Flake slides: similar to (5b) but involve somewhat uncemented materials, usually retrogressive (movement moves uphill as face left unsupported) but may be progressive (movement moves downhill as slope is overloaded by initial movement).

(5d) Detritus or Debris slides: slide material behaves as a more or less cohesionless mass; generally occur on fairly steep slopes (15 - 40°); usually fairly rapid. Depth of movement and amount of distortion influenced largely by cohesion of debris - at one extreme heavily weathered clay debris may approach the nature of a "slab slide" - at the other, dry cohesionless material may resemble a "sand run" in which only the grains of a surface layer move. Extremely rapid debris slides often result from heavy rainfall in tropical regions; both this and the soil mass movement involved in dirty avalanches in cold mountainous regions grade into mass transport (as by rivers).

(5e) Mudflows: With the exception of lahars these develop characteristically beneath steep, bare slopes of deeply weathered fissured or jointed rock which serve as a debris source. If a sufficient amount of such accumulated debris breaks down to a clayey paste mudflowing will occur when the essentially granular mass becomes saturated.

(5ei) Mudslides: relatively slow moving lobate or elongate masses of softened argillaceous debris, essentially viscous in character like soft tar or honey. Some creeping earthflows after substantial movement exhibit a similar appearance.

(5eii) Mudspates: an overloaded torrent; a rapidly moving highly fluid slimy mud carrying abundant coarser material; generally of very short duration and thus rarely observed. Only by rapid movement can such a mixture of liquid and solid be maintained, if slowed a temporary dam may form only to surge on again even on slopes as flat as 1°.

(5eiii) Volcanic mudflow - lahars: associated with volcanic eruptions and arise from the sudden supersaturation of (great) accumulations of volcanic dust and ashes: large amounts of water necessary - which may be derived from, ejection of crater lakes, the condensation of steam clouds or the melting of snowbanks. Frequently very destructive: some of their extreme mobility may be due to included gases.

(5f) Bog flows; Bog bursts: predominantly translational downhill movement of masses of saturated peat. Scarcely known in New Zealand - see Hutchinson (1968) for occurrence in U.K.

(5g) Flow failures: originate through collapse of the metastable structure in certain loose, predominantly non-cohesive uniform silts or fine sands, which are generally also saturated.

(5gi) Loess flows: appear to involve failure of virtually dry deposits; collapse usually caused by earthquake shock. Mobility of flow suggests that positive air pressures are present; maintenance of such pressures is possible only at high speeds.

(5gii) Flow slides: occur in saturated or nearly saturated non-cohesive uniform silts or fine sands. Transient positive pore pressures due to shearing of an essentially normally consolidated, material gives the failing material a semi-fluid character. Coarser materials are not prone to this phenomenon because higher permeability allows rapid dissipation of pore pressures; finer materials are not prone because of their cohesion; unsaturated materials are generally not prone because they can contract when sheared without developing significant positive pore pressures. They also commonly occur on the sea bed as subaqueous slides.

(6) Rotational Slips: Failure takes place (usually fairly rapidly) by shearing on a well-defined curved slip surface: usually more deeply seated than translational slides. Occur principally in fairly thick and homogeneous beds of clay or shale.

(6a) Single rotational slip: Single concave slip surface upon which the slipping mass moves as a virtually coherent unit.

(6b) Multiple rotational slip: two or more slipped blocks, each with a curved slip surface tangential to a common, generally deep seated, slip sole. Two types can be recognised

i. in stiff fissured clays - multiple rotational slips occur only where a harder cap rock prevents degradation of the rear scarp of earlier slips. If this scarp is degraded quickly stresses are unlikely to rise again high enough to trigger a deep rotational slip.

ii. in soft sensitive clays - the slip movements remould the quick clay forming the lower part of the initial slip, to the consistency of a heavy liquid. This runs away leaving the steep rear scarp unsupported thus inducing another rotational slip. Retrogression is extremely rapid.

(6c) Successive or Stepped rotational slips: occur in stiff fissured clay at slopes approaching their angle of ultimate stability (i.e. fairly flat). Each slip is of limited extent down the slope but of considerable extent across it, so forming terraces.

(7) Falls: The more or less free descent of masses of soil or rock of any size.

(7a) Stone and boulder falls: falls of rock bodies which were already separate from the slope mass; limited in magnitude.

(7b) Rock and soil falls: progressive separation (frequently protracted) of a mass from its parent cliff leading to collapse. Commonly, the growth of tension cracks leads to shear failure at the root of the mass. Occur on rock surfaces on which the effects of pressure release (due to excavation) and of variations in temperature and cleft water pressure are significant. Many examples to be seen in cuts in weathered greywacke in the Wellington area.

(8) Subaqueous slides: Occur in bottom sediments in seas or lakes.

(8a) Flow slides: as in (5gii) above, the collapse of structure leads to contraction and therefore to positive excess pore pressures. Occur in cohesionless or slightly cohesive silts and fine sands.

(8b) Underconsolidated clay slides: high rates of sedimentation of very fine material may lead to high excess pore pressures due to lag in consolidation; underconsolidated slides may result, even on very gentle slopes.

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As a final cautionary note of the need for a common language concerning slope stability problems the following extract from an explanatory leaflet to property owners by the Earthquake & War Damage Commission is quoted.

> "Landslip damage means damage caused by the subsidence of a substantial land mass other than by settlement, soil shrinkage or compaction and includes the movement from any hill, mound, bank, slope, cliff or face

of earth or rock of a substantial mass of earth or rock which before movement formed an integral part of the hill, mound, bank, slope, cliff or face."

How many readers would be prepared to defend this at law?

FACTORS CONTRIBUTING TO SLOPE INSTABILITY

Before listing the many factors which may contribute to slope instability in soils and rocks it is appropriate to emphasise the importance of water. It is no exaggeration to state that in the majority of situations where instability has occurred an increase in water present in the area has been the decisive factor. Commonly this increase has lead to increased pore pressures at depth with consequent reduction in effective stress and shear strength (B 3i in Table 2). Of the thirty one factors listed in Table 2, water is operative in nineteen.

TABLE 2

FACTORS CONTRIBUTING TO SLOPE INSTABILITY IN SOILS AND ROCKS

(after Varnes 1958)

A. Factors contributing to increased shear stress		
1. Removal of lateral support	i.	Erosion by water
	ii.	Weathering, wetting, drying, thermal stresses, and frost action
	iii.	Slope steepness increased by mass movement
	iv.	Manmade excavations
	v.	Sudden drawdown
2. Overloading by	i.	Weight of water, snow, talus, or other mass movement from above
	ii.	Fills, wastepiles, structures, vehicles
	iii.	Growth of vegetation
	iv.	Stress concentrations caused by fissures
3. Transitory stress	i.	Earthquakes
	ii.	Manmade vibrations
	iii.	Wind action on trees
4. Removal of underlying support	i.	Undercutting by running water
	ii.	Subaerial weathering, wetting, drying, thermal stresses, and frost action
	iii.	Subterranean erosion (eluviation of fines or solution of salts)
	iv.	Mining activities
5. Lateral pressure	i.	Water in fissures
	ii.	Freezing of water
	iii.	Swelling by hydration of clays
	iv.	Seepage forces (viscous drag of moving water on soil grains)
	v.	Wedging action of roots
6. Tectonic movement	i.	General steepening of slope
B. Factors contributing to reduced shear strength		
1. Composition and texture	i.	Presence or development of planes of weaker material
	ii.	Unfavourable pattern of jointing in rocks
	iii.	Loosely packed materials - spontaneous liquefaction, sensitive soils
	iv.	Overconsolidated clays - long term stability
2. Physico-chemical reactions	i.	Cation exchange
	ii.	Chemical weathering, decomposition, oxidation- reduction, leaching
	iii.	Physical weathering, breaking of particles by frost, capillary and thermal stresses
3. Effects of porewater	i.	Excess porewater pressure
	ii.	Reduction of capillary tension

Failure occurs when the total shear force acting on a surface within the materials exceeds the total shear strength which can be mobilised on that surface. The surface must of course divide the mass of material into two or more distinct regions. The factors contributory to slope instability may therefore be divided into those leading to increased shear stress and those leading to decreased shear strength (after Varnes 1958). The factors listed will be generally self explanatory to most readers of this paper particularly if all levels of headings are read, e.g. "instability due to increased shear stress due to removal of lateral support due to sudden drawdown" is more informative than just "sudden drawdown". Some amplification of a few of the factors listed, however does seem appropriate.

A.1.ii. This category refers to increases in shear stress at depth within a slope which may result from removal or weakening of material on the slope face.

A.1.iii. Steep slopes and cliffs are commonly created by the movement of part of a slope.

A.1.v. The time lag (particularly in fine grained materials) between the lowering of say, a lake water surface and the lowering of the water table in the soils on the shoreline, will leave a mass of water in the shore slopes effectively without the lateral support it had prior to drawdown.

A.4.ii. As for A.1.ii. in situations where the weathering or fretting material is undercut rather than on the upper surface of the potentially mobile mass.

B.1.iii. Loosely packed materials tend to compact when disturbed (i.e. when subject to a shear stress). Contraction implies expulsion of liquids and/or gases from the voids. If the permeability of the material is insufficient to permit immediate explusion of these fluids, significant positive pore pressures may develop which can greatly reduce shear strength.

B.1.iv. Tightly packed materials will tend to expand when disturbed. Expansion implies the intake of fluids into the voids. If the permeability is insufficient to permit immediate intake of fluid significant negative pore pressures may develop. These can markedly increase the immediate shear strength of the material. Immediate failure may thereby be prevented but in the long term the negative pore pressures will be relieved and the slope may then fail.

B.2.i. This occurs in clay materials and commonly involves the replacement of divalent (Ca, Mg) with monovalent (Na, K, H) ions. The resulting clay structure is more readily dispersed and may ultimately have a lower shear strength.

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Chairman: *Mr. D.K. Taylor*

Section 2: INSURANCE and Section 3: CLASSIFICATION

PRESENTATION AND DISCUSSION

Mr. J.L. Gill, presented his paper "Risks, Legalities and Insurance of Slope Stability". (p.2.1.) He said that in the paper he had endeavoured to outline the scheme of insurance operating under the Earthquake and War Damage Act, leading to the introduction of automatic landslip insurance. Prior to 1970 landslip insurance was only available from the Commission on a voluntary underwriting basis at a premium commensurate with the landslip risk. Only homeowners with a known instability problem showed any real interest and the cost was generally high. Very few voluntarily purchased this protection. The incidence of landslip damage on residential sections had increased with greater use being made of hillside areas.

It was only to be expected that eventually there would be some move to provide a measure of assistance to the unsuspecting homeowner with an instability problem not of his own making. The present landslip regulations spelt out the thinking of a special committee set up by the Government of the day to investigate and report on the feasibility of providing automatic insurance cover. It was felt automatic cover under the act was desirable, but it should follow the same concept of cover as provided for earthquake damage and extraordinary disaster damage and provide indemnity only in respect of material damage to the building itself, the item under the fire policy and on which Earthquake and War Damage premium is levied. It was considered the Commission should not become involved in siteworks or remedial measures to restore stability on the section and that it was not the responsibility or function of the insurer to meet the cost of preventive measures to preserve insured property from damage.

Some people may have felt that landslip cover which had been provided fell a long way short of easing the landslip burden for homeowners. Some may also have felt a more prudent and economic approach would have been for the Commission to also assist with preventive measures to prevent costly claims. The answer must lie in the practicability of extending the Act provisions to provide such assistance for landslip claimants and the cost involved. It is inevitable after any disaster, be it earthquake, storm, flood or landslip, many property owners will suffer some financial loss not recoverable from insurance. Only a small proportion of those who pay into the fund administered by the Commission could be affected by landslip. It would be inequitable to widen the landslip provisions and not extend a comparable cover to other claimants.

Compounded, the uninsured loss arising from storm and flood is considerably greater throughout the country than that arising from landslip. The cost of maintaining a fund to meet even a part of the damage on residential sections would be considerable. The cost would be astronomical if the fund was also to meet the cost of preserving property from damage requiring a substantial increase in the present rate of premium.

The avenues open to the Commission to opt out of some or all Act liability by classifying property or contracting out altogether are outlined in the paper. The landslip provisions were introduced to meet a social need for existing homeowners as well as those of the future. The Commission must carry the known less stable risks and apply any cover restriction with discretion. The present landslip cover was not intended to remove all the aches and pains of landslip damage on residential sections - merely to soften the blows. However, some further assistance does appear desirable administratively and would remove some frustrations experienced - particularly in the case of a dwelling which is slowly moving to inevitable destruction. If it were possible for the Commission to meet the cost of resiting on some other section provided by the owner, the ultimate cost of the claim could be less. Again in a case where resiting is not possible economically, it would assist the homeowner to get rehoused if a total loss settlement could be effected without waiting until the building had reached the stage of becoming a material total loss.

Without pointing the finger of responsibility in any one direction Mr. Gill felt that it should be accepted that with nearly every failure affecting a residential site, someone somewhere down the line had done something that should not have been done or failed to do something that should have been done. The loss should therefore be carried by the party responsible and only in a case where the homeowner had been the author of his own misfortune should he suffer the consequences. From the many reports received by the Commission following investigation of landslip claims, it is clear local authorities hold the key to controlling the incidence of damage on sections. It is equally clear that many of today's problems could have been avoided by the local authority. Local authorities are charged with the welfare of the community. It would appear they have a moral obligation if not a legal liability to make good damage on sections approved for home building. Local instability problems should be resolved at local community level and should not be a national burden.

Mr. Gill suggested local authorities could administer a fund from which homeowners could be reimbursed for uninsured losses and the cost of any remedial or preventive measures necessary after development. This fund could be financed by a nominal levy on developers for each new section and by a nominal rating based on the unimproved value of hillside residential lots. The demands which would be made on each district fund would depend on the standards the local authority demands and the control maintained over all facets of development of hillside areas for housing.

<u>*Dr. J.G. Hawley*</u>, then presented the paper on "Classification and Mechanisms of Slope Failures in Natural Ground". (See p.3.1). The first half of the paper centred around Table 1. In the earlier systems reviewed, there was the tendency to put various types of failure into diagrams, plotting increasing water content one way, and speed of movement another way. It was found that when one tried to put particular occurrences on to the diagram, they did not plot just as a point. There were various aspects in the particular occurrence. Therefore the diagrams were of fairly academic interest and what the symposium needed was a definition of terms. Table 1 was a collection of terms based on a list produced by Hutchinson, followed, in the paper, by notes on the meanings of the terms. It was hoped that the Table would help people to communicate with one another and help with communication with the general public and possibly even come into the legal langauge. In this respect it was necessary to have terms defined. Inter-grades would always occur between the types listed. Particular occurrences may begin as one thing and continue as something else. From Hutchinson's list stable relics (of past instability) like stone streams had been excluded. All the items listed in Table 1 deferred to present instability. Included were such items as creeping earth flow - something of great interest in New Zealand. Really the main interest was in classes 5, 6 and 7. There was some doubt as to where to put the clay flows. They were shown as 6b(2) but could equally as well have gone in under 5g(2). On page 3.3 was a note under deep seated creep "there is as yet scarcely any direct field evidence for its existence". Dr. Hawley felt that in Wellington they were getting to the point where there was some direct field evidence for its existence. He mentioned the problem surveyors were having in Wellington with the gradual distortion occurring over the years between the trig stations. This was the sort of evidence Hutchinson said they did not have on the other side of the world. Ubiquitous in the world of geology was the fossil evidence of bent strata.

In the second half of the paper there was a list of factors contributing to slope instability in soils and rocks. The effect of water in most cases was the key thing. It was hoped that Table 2 (p.3.6) would serve as a check list. In reading the check list all levels of heading should be noted. This would give a proper picture of what was being discussed. The criterion given was useful: 'Failure occurs when the total shear force acting on a surface within the material exceeds the total shear strength which can be mobilised on that surface. The surface must of course divide the area into two distinct regions'. This definition of failure had led to the division of the table into two halves, A, "Increased Shear Stress" and B, "Reduced Shear Strength". He emphasised 'increased' shear stress, not 'high' shear stress and 'reduced' shear strength, not 'low' shear stress. In dealing with natural country the shear stress might in fact still be very low. It was the changes and not the absolute values which were of interest.

Mr. Olsen, said that examples of the "creeping earthflow" category of translational landslide as defined in the paper by Northey et al., could be found in the Poverty Bay - East Coast region.

Some of the hillsides in this area were covered with ancient landslide debris - an unsorted mass of soil and rock, distinct from the underlying bedrock (Bishop, 1966). Recent earthflow failures had taken place within this regolith (Stout, 1971) and some flows covered whole hillsides. The motions of others, however, were confined within discreet boundaries and these slides were similar to glaciers in appearance.

Three of the discrete creeping earth flows were then being studied in some detail by Ministry of Works and Development staff at Central Laboratories in conjunction with Water and Soil Division staff from Gisborne Residency. The rear scarps of the flows were characterised by areas concave in elevation, where sliding had occurred with some rotational component. The accumulation of material from this zone appeared to feed the main body of the flow which in general consisted of saturated hummocky ground with broken turf cover, confined between exceptionally well defined lateral shear planes. A pressure ridge was formed at the base of the flow, and in some cases this material was eroded from the toe by water flowing from the failed mass of soil. Slope angles range from 5°.

It was possible, Mr. Olsen said, that the movements during the initial failure were fairly large and rapid. A state of quasi-equilibrium was reached and subsequent downslope movements were associated with periods of heavy rain. These movements had been monitored and varied from a few centimeters to several meters per year and were variable along the axis of flow. This fact suggested that there may be active and passive zones within each flow. The glacier-like shape of the discreet creeping earthflow excluded anlysis using conventional infinite slope assumptions and consideration must be given to the movement of a mass of soil within a channel of either rectangular or parabolic cross-section. Attempts had been made to analyse flows using viscous flow models. Analyses of the movements using the theory of plasticity, as had been done in the case of glaciers (Paterson, 1972), probably provided a more sound basis for study of earthflow mechanisms. However, since most of this work assumed a zone of gradual transition from zero velocity to some finite velocity within the flow, some modifications were necessary to provide for slip at the well defined lateral and basal flow boundaries (Brukl, and Scheidegger, 1973).

References: Supplied by Mr. Olsen

BISHOP, D.G., 1966 "Landslides and Earthmovements in the Gisborne District". Unpublished New Zealand Geological Survey Report.

BRUKL, E and A. SCHEIDEGGAR, 1973 "Application of Theory of Plasticity to Slow Mudflows", Geotechnique, Vol. 23, No. 1.

PATERSON, W.S.B. 1972 "The Physics of Glaciers", Pergamon Press.

STOUT, M.L. 1971 "Engineering Geologic Observations - East Cape Area". Unpublished New Zealand Geological Survey Report.

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Mr. Brickell, expressed his pleasure at hearing Mr. Gill suggest a locally administered and locally raised fund to deal with problems of slips involving homes. When the practice of subdividing hills in Wellington was getting a hammering in the press he had written an article proposing just this in the Dominion. He joined issue with Mr. Gill concerning his remarks (Para. 7), 'by enforcing more realistic subdivision standards with perhaps absolute liability of those responsible when failures do occur'. He said that although he admired the justice system, it was seldom seen to greater disadvantage than when a problem of this kind was brought before the Court. The costs tended to escalate and there were cases where the total costs of the action were out of scale with the values involved. The adversary system of cross-examination of the problem was not the right way to determine responsibility. The determination of responsibility was so difficult that he leaned towards 'no fault' correction of the problem and perhaps some system of fining those who may be wholly or partially responsible.

Mr. Beach said that he considered it to be in the national interest that hill country areas be urbanised. If this was not done Wellington, as an example, would spread into the Waiarapa and the West Coast, costing the country valuable farm land and increasing the cost of servicing and transport.

However, as the Chairman had pointed out in his key note address, hill country could only be developed with some degree of risk - geomechanics can reduce but not eliminate the inherent risks.

If these risks are accepted and hillside development allowed to continue then it follows that, despite all due care, stability problems will occur from time to time. These problems will represent a very small percentage of the total development although they will be very serious to the householders concerned.

When difficulties arise Mr. Gill said that he considered the local body in the particular area to be responsible for allowing the development. Local bodies often consider that the designer is responsible and designers have been known to blame the contractor - a 'buck passing' exercise which does little to advance the science of geomechanics.

Mr. Beach said that in his view the risks should be covered by an extension of the Earthquake and War Damage Commission cover. From an actuarial point of view the risks are acceptable and from a social aspect it would be of the greatest advantage to the unfortunate few householders involved in these problems each year.

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Mr. Ireland, taking issue with Mr. Gill on the points raised by both Mr. Brickell and Mr. Beach disagreed that it was an insurance risk. It was his opinion that the standard of engineering and construction in much of the subdivisional work must improve if construction was to be extended to the higher risk areas. Quite a number of the failures that had occurred in Wellington were due to lack of knowledge in the general field and lack of engineering in the more difficult country where the problems were greater. He did not believe the local authority should be held responsible but the engineers in the local authority must ensure the standards of engineering were met. He then referred to Mr. Hawley's paper - Table 1: "Ancient land movements" which were now completely excluded, may be over 100 years old. They may not be creeping at the present time but they were in fact failures of an ancient sort. The older failures should also be included in Table 1.

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Mr. Costello said he would like to take issue with Mr. Gill in laying blame for slope failures affecting houses on local authorities. This he regarded an unfair oversimplification.

The problem was partly historical. There were many people other than the local authorities who make a major contribution to the problem which, it should be recognised, is a natural phenomenon on where steep slopes inevitably become flatter.

Over the last 10 years the number of residential developments with large mass earthworks on steep ground has greatly increased and he thought it would be widely admitted that use must be made of this land.

Local authorities may have lagged in some areas in setting and applying proper standards for these subdivisions but even when these were established and properly administered results were dependent on the competence, honesty and effectiveness of the Developer and his engineering adviser.

Local authorities could not, without greatly increasing their staff levels, provide "Clerk of Works" supervision of these jobs or detailed geological and engineering analysis of development proposals and could only lay down principles and must rely on the Consultant to the subdivider to do his job.

Mr. Costello described a situation in Whangarei where in spite of the local authority's strongly expressed concern to the developer at all stages of the proposal and its best efforts in administering the proposal, including the certification of the proposals by a registered engineer, major slips had occured which would possibly have endangered 3 houses despite insistance that the house be kept on original ground - in cut area, as far back as possible from the possible slip area.

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Dr. Hawley referred to paragraph 5.3 of Mr. Gill's paper where mention was made "that the Commission may contract out of liability in respect of a particular site after total loss settlement" when it was believed the site was no longer suitable for housing. He asked Mr. Gill whether this appeared on the title of that land. He believed that it did not and that there were instances in Nelson where, had this been done, houses would not have been built on those particular areas.

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Dr. Parton referred to Mr. Gill's local scheme for offering aid to those suffering damage from landslip. He asked Mr. Gill what the Commission proposed to do about aid to those who had suffered damage to sections from landslips but where the actual dwelling had not been affected materially but had resulted in the dwelling being uninhabitable - until such local body scheme as suggested, was set up. In three cases in Wellington, damage through landslip had resulted in evacuation of a dwelling and in one case stabilisation of the land alone was estimated at $12,000. In another case the house was a write-off. In general those involved were innocent parties who had done nothing to precipitate the landslip. If the homeowner must bear the cost how Dr. Parton asked, would he raise the finance? He also asked why Mr. Gill proposed the fund be of a local nature and not on a national basis.

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Mr. Gill in answer to Dr. Parton's question as to why he suggested that the fund should be on a local basis, said he believed that if it was on a local basis and the local authority was taxing the local people for a local problem, the local authority would make sure that their standards were upgraded sufficiently to meet the local problem. Different areas had a different problem which was magnified in the larger areas and some of the larger centres had a very large instability problem. Any national fund contributed by all would be inequitable for the smaller contributors paying in to it. The levy per capita or land values might adjust it but he felt that in time, once the local authority appreciated that it was administering a fund and paying out the locals' money for a local problem, the demands being made on that fund would eventually dwindle down to a reasonable level. If this was financed by a national levy, with all property owners forced to pay, then there would be no incentive to local authorities to upgrade their standards. The standard of care in developing areas for residential purposes would drop unless it was mandatory on local authorities to accept, implement and police a national code. He did not think that would ever come.

For the Commission to raise the money to meet the landslip problem of a few would not only be a large scale change in the present concept but also the loss of the present simplicity of the scheme whereby a premium was levied on all insured property, irrespective of the hazard. The big organisation with no landslip risk was the one contributing mostly into the fund administered by the Commission. Regarding property owners who had suffered damage on their section without any damage to the house, the Commission proposes to do nothing because it has no authority to do anything. The Commission's function was merely to administer the legislation. In reply to Mr. Brickell's remarks about 'absolute liability' Mr. Gill said that perhaps 'liability' was a wrong term. The concept of his thoughts was that there should be an absolute responsibility. A local authority could still sue against a third party. To Mr. Beach he said that by all means they could use the hillside areas providing they were suitable or made suitable. From reports on some of the newer subdivisions it was obvious that some buildings on some sites were doomed. Sometimes it was the builder or homeowner who was at fault. It cost less to make these safe at the time of development rather than take remedial measures later. In answer to Mr. Ireland, Mr. Gill said that everyone must be educated. It was not only the local authority but all those involved. The episode of landslip over the last six months throughout New Zealand were serious and it was not only the Commission but also the Minister of Works and all the other experts who felt that what had happened should not have happened. In reply to Dr. Hawley's remarks, he said that with regard to the Commission contracting out, this was not registered on the land title. He said that their involvement was only after a building had been sited and damaged.

They paid total loss and implemented the authority they had to contract out of any further liability in respect of the section. They could not enforce it. All they could do was give notice to the owner and expect him to tell any future owner of that section that the Commission accepts no responsibility If he did not, then he could be personally liable. Usually notice was given to the local authority but the Commission had no power to register it on the title.

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Dr. Hawley replied to Mr. Ireland's remarks on the ancient landslide. He said that Table 1 was a classification of occurrences, not a classification of dangerous ground. Probably what Mr. Ireland was thinking of was something that would come under Table 2, that there may be reduced shear strength due to the composition and texture of the ground due to the presence of planes of weaker material.

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Mr. Depledge referred to the apportioning of blame. He felt it was preferable to think in terms of giving assistance to various people who were involved: the local council, the consulting engineer and the contractor. He reiterated the need for a "code of practice" in establishing slopes in cut ground. There was already a code for filled ground.

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Mr. Slimin suggested with regard to Wellington in particular, that the limit of sensible expansion had been reached; future development was simply pressing on in a ridiculous fashion.

Such towns or areas should accept that they had reached, probably over-reached their maximum size and development should therefore cease.

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Mr. Morris suggested that slope stability maps be made and relevant areas be zoned and levied accordingly. This would -

(a) Provide necessary funding,

(b) Draw public attention to the problem existing.

This would be in the main a Local Authority function on the principle that the user pays.

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Mr. Gillespie said his firm had examined about 350 landslip cases in the last six months. Most of these were in the greater Wellington area extending to the Waikanae River on the West Coast and up the Hutt Valley to the Rimutakas. Almost half of the examinations were on behalf of the Earthquake and War Damage Commission and the remainder were for individual owners.

"A preliminary review of these cases reveals that 7% of them were houses not even finished and a total of 55% were houses no more than 10 years old. There was then a decline in percentage claims for houses 10 to 50 years old but a sharp rise again at the 50+ years age. This suggests that in Wellington which do not fail in the first 5 to 10 years may well last 50+ years. Even this latter age does however, fall within the current probable life span of a house.

Note also that all cases refer to private domestic properties - no industrial, commercial or high density housing.

A relatively high percentage of landslips occured in the Plimmerton area of Porirua City (until recently administered by Hutt County Council). All were caused by the complete lack of or the appallingbad quality of stormwater control.

Only two of the cases examined were in controlled, compacted filled areas and both of these were due to neglect by the builders concerned, of stormwater control. The next were in cut ground, non-compacted fill or natural ground, mainly the first. I suggest there are several groups responsible:-

(a) The developer who tries to get as many sections on a given area as possible to keep the sale price down and/or to increase his profit.

(b) The developer's professional adviser (engineer or surveyor).

(c) The Local Authority who allows such work to proceed after insisting on their review of design beforehand.

(d) The house designer.

(e) The builder.

(f) The home-occupier-owner.

My observation is that when a landslip occurs only the last-named really suffers. With few exceptions all other groups tend to look the other way. The suffering home-owner commonly has two or three mortgages and the disaster of a landslip on his property can be heartbreaking to him. Some home-owners now consider those people involved in land development in its broader sense do not have any technical ability or real knowledge and their feelings are understandable.

The Geomechanics Society must realise <u>now</u> that this is <u>our</u> problem, not something that can be left to the oft-referred-to "them" or "they". Mr. Gillespie strongly recommended, urgent consideration of a code of practice for cut ground.

The cost of restoring sections in the Wellington area damaged by landslips this year was given by Mr. Gill as being about $10,000,000. Mr. Gillespie pointed out this refers to the greater Wellington area previously described and is for restoring the status-quo prior to mid April, 1974. It is not just the cost of stabilisation but includes restoration of previous ground surfaces as possible."

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<u>*Mr. Russell*</u> said he did not believe that the increase in landslips and hazards of this nature was because of the development of very steep country. A very cursory inspection of Wellington would reveal that the ancients, decades ago, got into country that would be considered totally economical to handle today and was far steeper than one would dream of touching today. He felt that part of the problem lay in the town planning ordinances as laid down by the local authorities where a minimum section size was laid down for the whole of the district irrespective of the fact of whether it was flat, hilly or very steep land.

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<u>*Mr. Blakeley*</u> brought up the subject of National Standards. The standard for fills was only a draft and very few local authorities were actively using it. Few subdivisions have been finished that were designed to this standard. Local authorities had probably insisted on more engineering control over filling than over cutting, rather than adopting any standards. He had been on the committee which prepared the standard for filled ground and they had discussed whether the code of practice should be widened to include cut ground but it was concluded that this would cause delay so it was better to procede with the code for fills. There were so many different factors which could affect the stability of cuts. However, there was a lot of scope for a set of broad principles which could be applied to excavating cut ground.

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<u>*Mr. Neich*</u> asked if standards should set out in detail how work was to be done or should they be end use or result type documents. If result specifications, the adviser to the developer must accept responsibility for the technical situation and risks. Because local authorities dealt with the broad spectrum of developers, standards between these would seem appropriate. How far did Mr. Gill consider local authorities should go in engineering and geological investigations as opposed to accepting the opinion of advisers to developers? Local authorities' control during subdivision was fairly limited and the time of the final control was at the time of the sealing of the plan. Often then the local authorities were faced with a fait accompli with unsatisfactory work incorporated. Control of excavation was very limited under legislation. He suggested if local authorities were required to have more responsibility, more powers were required.

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<u>*Mr. Sanders*</u> said that in order to protect the Commission from future claims and to protect householders, he felt it was high time there was an extension to the powers of the local authority to

give them responsibility to take prompt action in protection and remedial work in areas where slips involved more than one section. Where movement had taken place over a number of sections it was almost impossible to get anyone to do anything in concert. The only body with any hope of being able to do this was the local authority and therefore some form of legislation was overdue.

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Mr. Read asked, when did 'dangerous ground' become a landslide? A landslide was a landslide regardless of when it occured. Thus the use of the term 'dangerous ground' was sidestepping the recognition of landsliding, and landslide deposits.

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Mr. Gill replying to Mr. Neich said there was an economical limit to what extent a local authority should go in setting a standard demanded for subdivisions. He felt one should settle for a reasonable standard having regard to local knowledge, previous movement in the area and so on. Unfortunately in some of the cases handled, he would have no hesitation in pointing a finger at the local authority. He gave the example of an old slip that had been graded over and the local authority gave the owner of the land a permit to build a house. It was sold and three months later it was a total loss. That permit should never have been issued. Mr. Gill suggested, in another case, a local authority might permit building on a new subdivision provided the applicant for a building permit gave them an indemnity that they would not hold the local authority liable when the house did fall. These things did not go on every day but they did occur. He felt that common sense should prevail as far as issuing a permit for a particular situation. He felt Mr. Sanders had made a good point. It brought to light other difficulties which did arise, for example the fall of a bank of neighbouring land onto a house. If the homeowner removed more spoil in order to erect even a retaining wall and brought down more of the neighbour's land, was he liable? If there was a central authority like the local authority to step in and do these works involving two or more property owners where there was a problem entirely outside the fault of the individual owner then his suggested fund could be used for those works. If there was an objection from an individual concerned, then it was not the local authority's money being used but the people as a whole stepping in, and the individual's objection would surely be overruled.

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Dr. Hawley said that Table 1 described the occurrences and he agreed that a landslide was a bad place to build and he would not attempt to draw a clear line between the occurrence and the dangerous ground. Looking at Table 1 one could say that if any type of landslide had occurred (types 5,6,7 or 8), then a person would be very unwise to build there. All the dangerous ground situations should be describable in terms of Table 2.

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Mr. D.K. Taylor in summing up, suggested that: One could not be held responsible for something over which one did not have control. Careful thought should be given to who in fact had control in the sort of cases discussed. A lot of people were acting, perhaps without reference to each other. If someone was going to be made totally liable then he must be given total control. Far too many of the cases that those very actively concerned look at obviously should never have happened. In about 90 cases looked at, half were pure bad practice, about a third were negligence and only the rest could be regarded as inexplicable by some fairly cursory examination.

GEOLOGICAL ASPECTS OF SLOPE STABILITY

G.T. HANCOX

1.0 INTRODUCTION

Landslides are essentially geological phenomena resulting from natural processes which involve the mass movement of soils, rocks, or complex combinations of both. Although the causes of landslides are complex and varied, geological factors are nearly always one of the basic causes of slope failure. Because of this fundamental relationship between geology, topography and landslides it is clear that a geological assessment should be a critical part of any slope stability anaylsis.

In the past the geologist's approach to landsliding has been largely qualitative and his objectives and approach to the problem differed from that of the engineer. Over the past few years however there has been an increasing tendency to regard slope stability as an engineering geology problem. Although a sound understanding of the geological environment is required, the engineering aspects, based on rock and soil mechanics, and hydrogeology must also be appreciated. The engineering geologist provides the basic data on which the engineering calculations are based and usually, good results depend upon close liaison and understanding between the engineering geologist and the engineer.

The engineering geologist must attempt to present quantitative information on those geological factors which can be used in a stability analysis. The success of the study will depend on the availability of numerical assessments relating to rock and soil types, structures, rock defects, topography, hydrogeology, tectonic history, seismicity and other environmental factors.

The topic, "Geological assessment of slope stability" can conveniently be divided into two sections. In this paper it is intended to outline the geological factors affecting the stability of slopes and to discuss their relationship with various slope stability problems in New Zealand. In the following paper, Mr. Read will discuss the means of collecting, assessing and presenting the relevant geological data.

2.0 GENERAL PRINCIPLES AND TERMS

When dealing with slope stability it is often convenient to divide natural slopes into rock slopes and soil slopes and, because of the difference in parameters which are important in these two types of materials, this seems to be a logical separation. However, in practice this separation is only a convenience and in many areas the gradational transition of soils into rocks makes separation difficult, if not impossible.

There is often confusion over the use of the terms rock and soil, which can involve subtle variations or implications, depending on who is using it - a geologist or an engineer.

In engineering the situation is clear. Terzaghi and Peck (1967) define soil as "a natural aggregate of mineral grains or rock fragments that can be separated by such gentle mechanical means as agitation in water. Rock on the other hand is a natural aggregate of minerals connected by strong and permanent chesive forces ."

In engineering geology the term soil loosely refers to "all unconsolidated material overlying bedrock". This includes rock debris (weathered and fresh) of all kinds, volcanic ash, alluvium, glacial deposits, loess, pedological soils and associated vegetation. Because of this very broad range of materials, regolith is perhaps a more appropriate term in engineering geology and slope stability studies than is the term soil.

Bedrock is the common term that is used to describe rock (usually "solid") that underlies soil or other unconsolidated superficial material. The use of this term could be misleading in certain parts of New Zealand. For example, some of our Tertiary mudstones, sandstones and conglomerates are so poorly compacted and cemented that, according to Terzarghi's definition, they should be classed as soils. There are other "grey" areas of this type, where soils merge into rock, and where slopes comprise both. However, difficulties in communication can be avoided when terms are adequately defined and the materials are fully described.

3.0 GEOLOGICAL FACTORS AFFECTING SLOPE STABILITY

3.1 Geology and Topography:

Gravity is the primary cause of all slope failures, and steep slopes, which are formed by a variety of processes and conditions, accentuate this basic factor. In New Zealand, and most other countries for that matter, we often associate steep slopes with our mountain terrain, where recent uplift has been rapid and subsequent erosion and downcutting by streams, rivers and glaciers has produced very steep slopes (35-80^{o} or more). However, the fact that a slope is steep does not mean it is also unstable. Other factors, such as rock condition and weathering enable some slopes in Fiordland (unweathered diorite-gneiss), and the Southern Alps (unweathered massive greywacke), to be stable at angles approaching the vertical. In these areas recent glacial erosion has removed much of the weaker weathered material, forming sheer faces of massive, unweathered rock. In other parts of the South Island such as the bush covered ranges of the West Coast - Buller area, the instability of the steep slopes is compounded by high rainfall and weathering which has produced thick regolith (slopewash) deposits.

Certain rock types are characterised by typical slope angles that develop during normal processes of erosion. Limestones for example often outcrop as near-vertical cliffs, especially where rock falls have been initiated by erosion of soft underlying sediments such as mudstones or sandstones. Thick massive andesite lavas and ignimbrite sheets in the central North Island have similar topographic expression. Regional differences in geology play an important part in the topography and natural slope angles vary accordingly. In the Wellington greywackes natural hill slopes of up to 45^{o} are considered stable but this value may not apply to other parts of New Zealand and therefore batter angles and cut slopes considered safe in Wellington may not be safe elsewhere.

Slope maps are perhaps the best way to illustrate topography and natural slope angles and are now widely used overseas in slope stability studies. In the San Francisco Bay region (Nilsen and Brabb 1973) colour transparency slope maps (at a regional scale) are used in conjunction with geologic and landslide maps to produce landslide susceptibility maps which are of great value to engineers, urban and regional planners. The construction and use of these maps will be discussed in the following paper.

Following widespread landsliding during the 1968 Inangahua earthquake, a slope stability study in the upper Buller gorge was initiated and a slope magnitude map produced to emphasise present day slopes (Hancox 1969). When compared with plots of earthquake engendered landslides there is a clear relationship between slope magnitude and slope failure. Slope was a controlling factor in many of these landslides, although clearly many other factors were also involved. These will be discussed in subsequent sections of this paper.

3.2 Rock and Soil Types

Rock and soil types vary greatly from place to place and show a wide range of physical properties, texture, mineralogy, and structure. In engineering geology, age is a relatively unimportant factor, except where it has direct bearing on the physical properties of materials. For example a Cretaceous granite, may, with other factors being equal, exhibit the same physical properties as a pre-Cambrian granite; whereas young, poorly-cemented, sediments are often soft or unconsolidated and have contrasting physical properties when compared with older rocks of the same type.

Some rock types are inherently unstable. For example mudstones and siltstones, because of their clay content, frequently exhibit swelling and air slaking due to changes in atmospheric pressure and moisture. Many of our soft Cretaceous and Teriary mudstones have an appreciable content of montmorillonite (a type of swelling clay). Slopes in these areas are notoriously unstable and huge mudflows such as those in Hawkes Bay at Porangahau and Mahia Peninsula are common.

Landslides and slumps are characteristic of the landscape in the Tertiary mudstones between Wanganui and Taihape (Fleming 1953). The formation of clay minerals (including monmorillonite) through the hydration of fine glass fragments is apparently the basic reason for the mobility and incompetence of these rocks. In places, huge areas have slipped (some as large as 8 sq. miles - 2000 ha.) and Fleming concludes that most of the pre-historic slides occurred after the attainment of at least moderate relief. However despite the abundance of landslides much of this so called "papa country" is remarkably stable and vertical and overhanging faces (normal to bedding) are common in the Wanganui and Rangitikei River valleys. This contrasts strongly with other parts of this region where variations in lithology and thin montmorillonite layers have caused hundreds of deep seated landslides.

Clay minerals have a direct effect on the shear strength of mudstones, among these montmorillonite is widely known to have a deleterious effect on the rocks in which it occurs. With x-ray analyses and other modern analytical techniques and careful field sampling, montmorillonite has become recognised as possibly the most common lithological factor in slope failures. Because of the importance of this material the following description is included (adapted from Grimm 1953):- "Montmorillonite belongs to a group of expanding lattice clay minerals, usually of calcium or sodium type. The basic structure consists of a series of silicate layers which attract water molecules to the interlayer surfaces giving the clay remarkable water adsorbing and swelling properties. If only a little water is adsorbed, the montmorillonite is held in virtually a solid state and the clay has a relatively high shear strength. With increasing amounts of water, lubrication occurs and allows movement between layers reducing shear strength to zero".

In the following briefly described examples of slope failures, the presence of montmorillonite has been one of the key factors involved.

(i) The Utiku "slip", about 5 miles south of Taihape is a well known problem are affecting both the North Island Main Trunk railway and State Highway 1. Since 1964 this old slip area (about 50 acres = 20 ha) has shown renewed movement. Several reports have been written about the problem and considerable money has been spent on investigations and remedial measures to stabilise the area. A recent investigation by Dr M. Stout, a visiting engineering geologist from the California State University, Los Angeles, has disclosed new information on the cause of the problem (Stout 1971, Ref 21). From this study he concluded that the Utiku Slip originally developed as a rotational failure in siltstone, the lower portion of which intersected a bedding plane which is defined by a planar layer of clay (a few mm thick) containing montmorillonite (Str 145°, dip 8° SW). The moving mass consists of relatively undisturbed blocks which are set in broken, crushed and saturated, disturbed material; movement is now on the planar clay layer.

Rates of movement can be directly correlated with groundwater fluctuations (rainfall) but stabilisation methods, aimed at draining and dewatering the area, have been only partly effective.

(ii) In the Dunedin area numerous slope stability problems were encountered during earthworks for the southern and northern motorways. The immediate causes of many of these problems were clearly related to the construction works (steepening natural angles of slope, overloading slopes, removal of lateral support etc.) but the basic factor appears to be lithological. Most of the problems were associated with sections in the glauconitic Abbotsford mudstone (L. Eocene) and the Burnside mudstone (U. Eocene) in which most of the larger landslides were founded. Both are rich in clay minerals, including a large percentage of montmorillonite, and probably as a result of this, both are susceptable to sliding (Stout 1971, Ref 20).

Other minerals likely to cause instability include serpentine, graphite, talc, chlorite, white micas and glauconite.

Serpentine, an alteration product of basic rocks is often found in massive form. It feels greasy and slippery and is as water absorbent and unstable as some pure clay minerals. Because of this it can be very troublesome when wet and has been responsible for many slides. Talc, chlorite, muscovite and graphite are usually restricted to metamorphic rocks, dramatically reducing shear strength along foliation planes.

At Devil's Gate damsite in Tasmania, graphite coatings on joint surfaces in argillaceous cherts were a constant source of trouble during foundation excavations.

Weathering products of glauconite increase pH values which can have a deleterious effect on calcareous cements, reducing the strength of rocks containing this mineral.

3.3 WEATHERING AND ALTERATION:

3.3.1 Weathering

Weathering involves the many varied processes of mechanical disintegration and chemical decomposition that lead to the degradation of rocks and soils and the production of an insitu mantle of slope debris or regolith. Physical weathering involves the loosening and disintegration of rocks by ice wedging, mechanical abrasion, erosion and differential expansion, (crumbling, exfoliation, spalling etc.). These processes form the talus or scree slopes characteristic of mountain regions. Chemical weathering implies rock decay by chemical processes and is achieved largely by the downward passage of rain water and the solvent action of dissolved chemicals which attack minerals by oxidation, hydration and carbonation. The effects involve the alteration of certain minerals, (especially feldspars and other silicate minerals), to form clay minerals. Iron from minerals is oxidated and hydrated to form limonite - the rusty-brown staining characteristic of many weathered rocks. The depth of weathering is controlled by the water table. Quartz and white micas are little affected, hence the predominance of these minerals in sand deposits.

The overall end result of the weathering process is a dramatic reduction in the strength of the rock mass and the formation of a loose mantle of rock debris, weathered rock and soils. On steep slopes these materials are particularly unstable and are the materials most often involved in land-sliding. Though apparently stable under normal conditions, when saturated the shearing forces exceed the frictional forces and failure occurs. Examples of this type of problem are common during periods of heavy rainfall, and news reports of landslides from all parts of New Zealand have become a feature of this type of weather. In failures of this type several factors are usually involved, and although the immediate cause is likely to be heavy rainfall the basic cause is the inherent instability of weak materials lying on steep slopes, often compounded by roading earthworks or similar activities.

Over long periods of time complete insitu weathering occurs. For example, the Wellington greywackes have been extensively eroded and weathered during the last 70 million years, and in places weathered rocks extend to depths of 100 m or more. Within this weathered zone rocks have been reduced to soft residual clays and sands which grade downwards into progressively less weathered rocks. Thick regolith deposits of weathered rock, residual clays, soils and solifluxion debris are sometimes stable on fairly steep angles but cut slopes, fills, and other earthworks are often unstable. The effects of heavy rain during this winter have highlighted these problems.

Andesitic ash, rhyolitic pumice and other unconsolidated volcanic products mantle vast areas of the central part of the North Island. Because of their loose, unconsolidated nature they are particularly susceptible to weathering and erosion. Andesitic ash derived from the Tongariro Volcanic Centre over the last 20,000 years has been highly weathered to clayey-silty-sands. Although surprisingly stable along steep banks of deeply incised streams and road cuts, problems do occur where these deposits lie on steep greywacke slopes in the Kaimanawa mountains. During construction of the Tongariro Power Development few major stability problems have occurred with these materials but several spectacular failures were initiated by the construction of access roads above the Rangipo underground powerhouse.

3.3.2 Hydrothermal Alteration

Hydrothermal alteration is simply an accelerated form of chemical weathering associated with geothemal and volcanic activity. In this context "alteration" implies rapid decay of the constituent silicate minerals, forming andesitic and rhyolitic clays, including montmorillonite, kaolinite with siderite and chlorite etc. Percolation of hot acid waters and steam are the main active processes.

The weakening of hillsides in the vicinity of Tokaanu and Waihi is largely a result of this process and has been a basic factor in the formation of many of the ancient landslides in this area. The most recent major landslides in this area have originated from the Waihi thermal area along the Waihi fault scarp (steaming cliffs), where geothermal activity has extensively altered the andesitic bedrock to form soft, multi-coloured clays. The weakened rock in this area has failed twice in historic times, producing huge rock and mud flows that obliterated the original Waihi Village in 1846 and again in 1910. Although without doubt these slides were triggered by prolonged periods of intense rainfall - and possibly earthquakes or fault movements - the basic cause of these failures was the instability of weak altered materials on steep slopes.

3.4 Structural Considerations

The stability of rock slopes depends not so much on the strength of the rock itself but mainly on the presence of planar defects within the rock mass. Apart from weathering and other strength reducing parameters, the physical properties of the rock mass are controlled by structural discontinuities. The parameters considered important include:- The frequency, attitude, continuity and nature of defects such as bedding planes, joints and foliation (schistosity) surfaces; the nature of the defect surfaces - including smoothness, weathering, coatings (clay, chlorite, limonite etc), slickensides, infilling minerals (qtz, càlcite), and sand, silt or clay infillings; and the nature, attitude and extent of fault planes or fault zones.

The spacial relationship of defects to slopes is especially important, acting both as a controlling influence on slope direction and steepness and as a basic factor in slope failure. When geological structures are important in a stability analysis, a reliable result can only be achieved if the calculations are based on data assembled by careful and detailed statistical sampling of rock defects. This type of study requires more than a casual geological mapping exercise and is usually only possible when the rocks are exposed with the aid of either sluicing, trenches, pits, drives, diamond drilling or similar subsurface investigations.

3.4.1 Bedding Planes

In some sedimentary rocks bedding planes are a basic weakness, causing the rock to split easily into layers or beds. Bedding planes that dip in the same direction as slopes are prone to bedding plane failure, especially in bedded sandstone and mudstone sequences. In clayey beds, even low dips (less than 5^{o}) have been known to be sufficient for sliding to occur. In the Tertiary rocks of the Hawkes Bay - East Cape area (Stout 1971, Ref. 19) landslides are stratigraphically controlled, the most important factors being clay mineralogy and bedding plane orientation.

The Oweka Slide near Inangahua is one example of this type of failure and was one of several bedding plane failures caused by the earthquake in 1968. Further reference will be made to these slides in a subsequent section of this paper.

3.4.2 Jointing

Joints are naturally occurring fractures in both igeous rocks and sediments, and are a common cause of failures, especially small ones, in all rocks. Steep, closely-spaced joints are a problem especially in weathered rocks where joints are likely to be clay coated, with resultant reduction in shear strength. The removal of lateral support by excavations causes rock masses to relax and joints to open. Clearly, open joints can have no shear strength, and the also promote slides by admitting water which can "lubricate" closed joint planes and increase hydrostatic and pore pressures. Ice wedging in joints is common during frosty weather.

Massive joint failures sometimes occur but the main type of failure is from small joint-controlled blocks (wedge failures from intersecting joint sets) or masses that drop out of rock faces along road cuts or similar excavations, a common feature during heavy rain or earthquakes.

3.4.3 Foliation Surfaces

Metamorphic rocks such as the Otago schists are characterised by a parallel arrangement or layering of minerals such as quartz, muscovite chlorite, graphite etc. This textural feature is known as foliation or schistosity and it usually imparts a planar fissility to the rock, causing it to split readily into thin layers. Slatey cleavage in slates is essentially the same type of defect. When weathered, this weakness is accentuated by the production of clays which readily promotes sliding, especially where foliation planes dip in the same direction as slopes.

Recent work in the Kawarau gorge (Stout 1971, Ref 22) has suggested that the landslides in that area are partly due to shearing along undercut foliation planes. For the first time, montmorillonite was identified in schist gouges and unoxidised schist, and is likely to have contributed to the stability problems. However, much more work is required to establish the source of the montmorillonite and its relationship to the mechanism of landsliding in the Kawarau area.

3.4.4 Faults and Associated Features

Slides occur on dipping fault surfaces (crush zones, gouges, shears and clay seams etc) which affect slopes in the same manner as joints or bedding planes. Such slides are usually due to the weakened mechanical properties of material along the fault plane, and only rarely due to actual movement on the fault itself. Slides on relatively narrow clayey crush zones are common in the Wellington greywackes, mainly affecting road cuts and cut slopes for new housing sub-divisions. A recent paper (Riddolls 1974) describes an example of slope failure concentrated along a fault plane, although increased water following heavy rain and the over steepening of natural slope angles were the main or immediate causes of the failure.

Although clay gouges and clay seams are sometimes significant, more often it is the associated zones of crushed and shattered rock (usually loose, weathered and saturated material) that are most prone to sliding problems. In brittle rocks (usually pre-Teriary) fault zones hundreds of feet wide are known and have caused numerous problems in road building and other engineering activities.

3.5 Seismic Activity

Earthquakes are often accompanied by widespread landsliding, especially in areas close to epicentres. Although by no means the chief cause of landslides, earthquakes are probably the most spectacular of landslide triggering mechanisms.

Recent major earthquakes throughout the world (Peru 1970, San Fernando 1971) have vividly demonstrated the effects of earthquakes on rocks and soils. The most dramatic aspect of the Peruvian earthquake was the cataclysmic rock and ice avalanche that fell 3700 m,(12,000 ft.) from the west face of Nevados Huascaran 7300m (22,200 ft.), raced across the landscape at 300 km/hr (200 m.p.h.) and obliterated two towns (Cluff 1971). Between 25,000 and 30,000 poeple were killed in what has been described as perhaps the largest most destructive landslide to have occurred during historic times.

The seismic areas in New Zealand are characterised by more frequent earthquakes and evidence of recently active faulting. Not all areas are equally affected by these phenomena. There are marked regional differences in the levels of seismicity,consequently the importance of this factor in slope stability varies from place to place. Seismic areas such as Marlborough- Nelson- Wellington or Hawkes Bay- Central North Island are recognised earthquake risk areas and seismicity is more likely to be an important factor in slope failure in these regions.

In simple terms earthquakes produce high lateral and vertical accelerations in rocks and soils and can also impart high dynamic groundwater pressures. The increased "g" forces have the effect of oversteepening natural slopes - for example, an earthquake which has an acceleration of 0.2g, effectively increases moderate slope angles by approximately 11 degrees (Oborn 1968). Failure occurs when the shear stresses resulting from these forces exceed the shear resistance (friction) along joints and other planar weaknesses. In loose sands and gravels, shocks cause a disturbance of intergranular bonds, and consequently a decrease in cohesion or internal fiction. In some saturated fine sands or silts, displacement or rotation of grains leads to sudden liquefaction of the soil. "Sand volcanoes" are one result of this phenomenon.

The Inangahua earthquake of May 24 1968, which had a magnitude of 7.0 on the Richter Scale, triggered many new landslides (most within a ten mile radius), and re-activated most of the old slides in the area, many of which originally moved at the time of the 1929 Murchison earthquake. The following brief descriptions of some of these 1968 failures illustrates the importance of earthquakes in slope stability.

Many types of landslides were recognised. A large percentage of these failures occurred in road cuttings and fills along the then newly-reconstructed sections of the upper Buller Gorge highway, causing widespread damage. In general, the size of the slips varied with height and slope of the constructed batters and the materials in which these were cut. Benching of batters, line drilling and different blasting techniques might have reduced this damage (Douglas 1969).

The widespread landsliding emphasised the inherent instability of the weathered near-surface rocks and slopewash debris. In the granitic rocks of the Upper Buller Gorge block slides on joint surfaces were common, but many of the more spectacular slope failures were mainly debris and rock slides incorporating vegetation, soil, slopewash debris and weathered basement. Slides in zones of crushed granite and lower Palaeozoic greywacke were also common, especially along the Glasgow and Lyell fault zones. In many cases the top of the less weathered rock provided a plane on which saturated slope debris and weathered mantle was able to slide.

Several huge debris slides of weathered granite occurred on the south bank in the Upper Buller Gorge. The 1908 (and 1929) "Buller slip" which extends for a ¼ mile downstream from Mountain Maid creek, was reactivated as were similar slides in Little Deepdale creek. Debris from one of these slides was redeposited to form an impressive delta in a slowflowing stretch of the Buller.

Possibly the most spectacular landslide in the affected area was the so called "Big Slip" in the Upper Buller Gorge. This huge debris and rock avalanche originated at the head of a small creek and swept 600m (2000ft.) down the steep southern bank, forming a specatcular scar which stretched from ridge - crest to river. Slide debris travelled as far as the road, about 60m (200ft.) up the northern bank, leaving a huge pile of debris that temporarily dammed the river, forming a lake which extended several miles upstream. Base levels are still high in this stretch of the river and there is noticeable ponding upstream from the slip. Fallen trees and isolated boulders and debris on slopes above the road, suggest that very high velocities were involved and that an air blast accompanied this slide. Aerial photos indicate pre-earthquake sliding in this creek, probably during the 1929 earthquake. After a period of rain in 1971 (Johnston 1974) a further landslide occurred, again blocking the river and severely damaging the highway.

Rock falls from the steep limestone bluffs were common near Lyell and at Whitecliffs, west of Inangahua, and some bedding plane failures from steep ridges were prominent with steeply dipping algal limestone sliding on calcareous grits. Similar failures were reported during the 1929 Murchison Earthquake.

Failures in the Tertiary mudstones (Blue Bottom and Kaiata formations) were often spectacular and examples of slumps, block glides, earth flows, and rotational slides were observed. Perhaps the most spectacular of these was the Oweka Slip, where the entire end of a mudstone spur (estimated at 2.3 million m^3), slid bodily on a clay-coated bedding plane which dipped less than $3\frac{1}{2}^{\circ}$.

3.6 The Effects of Groundwater

Of all the factors which contribute to landslides, groundwater is perhaps the most significant and ubiquitous. Groundwater affects rocks and soils in several ways, by, for example increasing their weight, decreasing effective pressure, softening fine-grained deposits ("lubrication"), and internal erosion.

Lanslides are particularly common during periods of continued heavy rainfall, causing recurrent sliding in some areas. Rain water saturates the soil and rock mass, causing a considerable increase in weight, "lubricating" likely planes of failure and increasing hydrostatic pressures. Increased pore pressure decreases cohesion and internal friction in soils and is probably one of the main trigger causes. The effect of water on a rock mass depends largely on its ability to accept water - either by its previous character or by open defects which admit water to the rock mass. During periods of hot, dry weather, drying out causes cracks to form in clayey soils, facilitating the entry of water. Hence, there is often a spate of landsliding during the first rain following long hot dry spells.

Groundwater movement causes piping or internal erosion in fine sands and silts and removes soluable cements in some rocks, reducing internal cohesion and strength.

Abrupt changes in water level, drawdown of reservoirs and lakes for example, can produce seepage pressures as a result of a steep piezometric surface. Moving ground water causes displacement of grains, especially in fine or silty sands, leading to slumping or other small failures around the edge of the reservoir. In low permeability clayey deposits around the shores of Lake Manapouri (McKellar 1973) low lake levels have reduced the lateral support of the water, resulting in shears, tension cracks and slumps in some areas. Seepage pressures also contributed to these failures.

4.0 CONCLUSIONS

Related and interacting geological factors invariably provide the basic causes of landslides, slowly and subtly reducing the stability of slopes but only rarely triggering slope failures. For this reason there is always a need for a geological assessment in all slope stability studies.

The frequency of failures in some recent urban developments suggests that insufficient geotechnical investigations are devoted to this aspect of engineering. There is little doubt that the number of failures would be substantially reduced if both geological and engineering stability assessments preceeded major earthworks for housing developments and other construction activities. A geological report, based on detailed mapping and subsurface investigations should provide the engineer with sufficient data to more confidently design safe batter angles, cut and fill slopes, and accurately delineate the likely problem areas.

Surely society is entitled to expect a high standard of protection from landsliding. With a little extra effort and expense this could be provided.

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ENGINEERING GEOLOGICAL ASSESSMENT AND SLOPE STABILITY

S.A.L. Read

1.0 INTRODUCTION

The previous paper has demonstrated the main means by which geology can control landsliding in natural ground. This control by geology and geological processes in the reduction of landscape during the mass movement portion of the geological cycle is clearly visible around New Zealand and is continuing at present. Man's interference has often accelerated or complicated landsliding and it is the duty of people involved in geomechanics to understand and control landsliding to the best of their ability.

In the past, as Mr Hancox pointed out, engineers and geologists have studied landsliding for different purposes and in different manners, and it is only recently that the two have started to study landsliding together for the same end. The reason for the differences, stems from basic differences in language and a non-acceptance of each others principles and viewpoints.

An engineering geologist should be able to bridge this gap between the two professions and enable free communication between them. He is essentially a geologist, who describes and interprets geological conditions and presents them as a geological "model", understandable and usable by an engineer. This geological "model" is, hopefully, easily and accurately quantifiable for an engineering analysis. This aim is not always possible but the errors in assumptions for analysis should hopefully be at least be reduced. To do this the engineering geologist on his part must appreciate and understand engineering principles and above all must be able to communicate freely with an engineer. Too many engineering analyses in the past have been based on wrongly interpreted geological information or on geological information not written in a form understandable or usable by an engineer.

The aim of this paper is to present the methods of collecting, assessing and presenting geological data for an engineering geological assessment which may form the basis of an engineering slope stability analysis. The liason abetween an engineering geologist and a geomechanics engineer will be mentioned.

2.0 LANGUAGE

The language used in classical geology is confusing and lacks relevant quantitative data for the geomechanics engineer. Material with different engineering properties are given the same geological name simply because they are of the same geological age. The use of loose terminology can also add to the problem. Thus the first major problem to bridge between geology and engineering is to use a language which combines the powerful interpretive aspects of

Degree of Weathering (Abbreviation)	Diagnostic Features	
	Soil	Rock
Residual soil		The rock is discoloured and completely changed to a soil in which the original fabric of the rock is completely destroyed. There is a large volume change.
Completely weathered (CW)	The soil is discoloured and altered with no trace of orignal structure.	The rock is discoloured and is changed to a soil, but the original fabric is mainly preserved. The properties of the soil depend in part on the nature of the parent rock. Small 'corestones' may occasionally exist.
Highly Weathered	The soil is mainly altered with occasional clasts or small lithorelicts of original soil. Little or no trace of original structures.	The rock is discoloured; discontinuities may be open and have discoloured surfaces. The original fabric of the rock near the discontinuities is altered; alteration penetrates deeply inwards, but 'corestones' are still present.
Moderately weathered (MW)	The soil is composed of discoloured clasts or large lithorelicts or original soil separated by altered material. Alteration penetrates inwards from the surfaces of discontinuities.	The rock is discoloured; discontinuities may be open and surfaces will have greater discolouration with the alteration penetrating inwards. The intact rock is noticeably weaker, as determined in the field, than the fresh rock.
Slightly Weathered (SW)	The material is composed of clasts or blocks of fresh soil, which may or may not be discoloured. Some altered material starting to penetrate inwards from discontinuities separating blocks. Clasts often skin weathered.	The rock may be slightly discoloured; particularly adjacent to discontinuities which may be open and have slightly discoloured surfaces. The intact rock is not noticeably weaker than the fresh rock.
Fresh Rock or soil (F)	The parent soil shows no discolouration, loss of strength or other effects due to weathering.	The parent rock shows no discolouration, loss of strength or any other effects due to weathering.
NOTE: Where possible the ratio of weathered rock or **soil** to fresh rock or soil should be noted for moderate to complete degrees of weathering.		

Fig 1: Table for Field Recognition of the Degree of Weathering of Rock and Soil. From Fookes et. al. (1971) with slight modification.

geological language based on measured values or ranges of physical properties. This language is called geotechnical language.

The language is being developed with the standardization of descriptive terminology and the development of testing techniques to back up geological description. (e.g. point load strength index test). Geological descriptions are being based more on the observed engineering properties of material rather than described in classic geological terminology to try and obtain descriptions closer to a quantitive manner. The terminology used in this paper, although not yet completely standardized, forms the basis for the terminology used by the Engineering Geology Section of the New Zealand Geological Survey.

2.1 Lithology: The division of soil and rock given in Terzaghi and Peck (1967) is normally used as the basis for describing lithology. However areas of overlap exist where this division is not applied (e.g. a Tertiary age mudstone which is normally described as soft rock, but is strictly a soil).

The unified soil classification (U.S.B.R. 1963) is used for describing soils although geological qualification is usually added as part of the description. This is largely because the classification is based on remoulded or disturbed characteristics of a soil and lacks description of 'in situ' structures. (e.g. bedding).

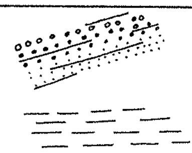
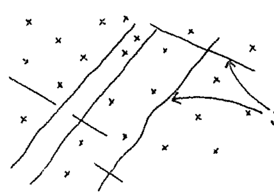
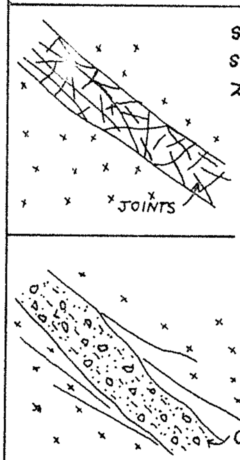
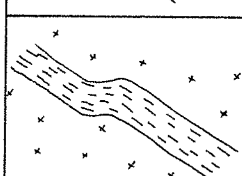
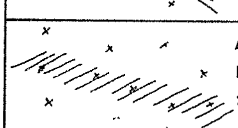
No classification equivalent to the unified soil classification exists for rocks which are described in broad geological terms, e.g. uniform, coarse-grained GRANITE.

2.2 Strength

Soil strength is described by degree of compaction or relative density and consistency depending whether the soil is non-cohesive or cohesive. In many cases these terms are only visual estimates, although laboratory or field testing (e.g. SPT values) are made in more critical areas. The pocket penetrometer is useful for helping assess consistency.

The strength of intact rock has been described by visual assessment or laboratory testing for uniaxial strength. However, the point load tester which measures a point load uniaxial strength index (I_s) of intact pieces of rock is gaining some acceptance as a field correlation for visual assessment of strength. Results from the test have been directly correlated with uniaxial strengths for some rocks (Brock and Franklin, 1972). For anisotropic rocks two tests must be done at right angles across, and parallel to the anisotropy and the strength Anisotropy Index I_s (50) determined. The Schmidt hammer, which measures a co-efficient of rebound, has also been used to assist in assessing the intact strength of 'in situ' rock.

2.3 Degree of Weathering: The degree of weathering of soil or rock is an important factor controlling its strength and is very important in slope stability studies. Weathering occurs in either or both of two ways: 1. Physical breakdown, and

Defect Name	Description
 BEDDING SCHISTOSITY OR CLEAVAGE	Layered or parallel arrangement of mineral grains or microfractures, giving rise to planar anisotropy in the rock.
 JOINTS	Almost planar surfaces or cracks, across which the rock usually has little tensile strength. May be open, air or water-filled, or filled by soil substance, or rock substance which acts as cement. Joint surfaces may be rought, smooth or slickensided.
 SHEARED OR SHATTERED ZONE JOINTS THESE ARE FAULTS IN THE GEOLOGICAL SENSE	Zone with roughly parallel, almost planar boundaries, of rock cut by closely spaced joints (often = 5cm) and/or cleavage planes. The defect surfaces are usually smooth or slickensided when fresh and the defects divide the rock in the zone into small blocks
CRUSHED SEAM	Seam with roughly parallel, almost planar boundaries, composted of disoriented, usually angular fragments of the host rock. The fragments may be of clay, silt, sand or gravel sizes, or mixture of these. The material in the seam has soil properties.
 INFILL SEAM	Seam of soil usually with very distinct roughly parallel boundaries. Formed by migration of soil into open cavity or joint.
 EXTREMELY WEATHERED SEAM	Seam of soil substance, usually with gradational boundaries. Formed by weathering of the rock substance in place.

NOTE: The origin terms are applied only when there is good evidence to support their use. Where it is not possible to deduce the origin type of a seam, an engineering description of the soil in the seam is given, with graphic representation where applicable.

Fig 2: Definitions of Rock Defects

2. Chemical breakdown (including alteration). The classification used for the degree of weathering by the New Zealand Geological Survey is essentially that of Fookes et.al. (1971) which describes the amount of chemical weathering that has occurred (Fig 1). Physical weathering which is breakdown or disintegration without chemical alteration is generally described by the rock defects or the resultant soil. Other classifications exist; e.g. Deere and Patton (1971) which is based more on the pedological weathering column, but are not preferred by the New Zealand Geological Survey.

In New Zealand, Pender (1971) has correlated the degree of weathering with engineering properties and behaviour of moderately - completely weathered 'greywacke' from a site in Wellington. However this type of classification should not be extended implicitly to different geological environments.

2.4 Defects; Rock and soils are penetrated by many defects, which generally weaken them, although in very rare instances they may strengthen them (e.g. limonite cementing of joints). The significant features of these defects include their orientation, persistence, roughness or defect surface, openness, nature of any infilling material, weathering lining and origin. The main defects for rock which are defined in Fig 2 can be extended to the description of soil defects in many cases.

Many attempts at classifying rock defects in drill hole core and exposures have been attempted. Deere (1968) proposed the Rock Quality Designation (R.Q.D.) for expressing drill core quality in terms of a number derived from core lengths and core recoveries. Fookes et. al. (1971) proposed the Fracture Spacing Index (I_f) for describing the natural spacing of fractures in drill core and natural exposure. The Spacing Index is generally preferred to the R.Q.D. by the New Zealand Geological Survey. Heavy reliance is placed on the graphic representation of the defects (with orientations noted), maps and logs, backed by adequate geotechnical description. The interpreted importance of the defects should be stated where possible.

3.0 DATA COLLECTION

The amount of engineering geological data to be collected and the methods used for a slope stability investigation is defined by:

1. Size of job.
2. Purpose of investigation (e.g. regional planning verses cut slope stability).
3. Type of material (s) in investigation area.
4. Cost and time.

In this paper the main methods or possible methods regardless of the scope of the investigation will be outlined. However, to do a job properly an engineering geologist should be told the engineering objectives clearly and be introduced into the job at an early stage before investigations are commenced in earnest. He should then liase throughout investigations and design with the engineer(s) involved and where applicable check the findings of the investigations against the conditions revealed during construction.

3.1 Regional Geology and Background: Existing relevant geological and engineering information on the area should be collected together with a possible literature survey as a first step. The New Zealand Geological Survey's 1:250,000 series maps give very useful background information and the regional geology. The Industrial Series maps (1:25,000) give more engineering based information in very limited areas. Little conventional regional geological mapping on a scale larger than 1:63,360 has been done by the New Zealand Geological Survey.

The aim of this part of the investigation is to:-

1. Establishthe broad geological history of the region.
2. Locate the main geological features, (e.g. faults).
3. Locate any aactive crustal processes (i.e. active faults).
4. Establish the main rock and soil types.

Conventional geological mapping usually neglects the distribution of landslide debris and the study of landslide processes and pays little attention to 'surface geology' because the 'in situ' rock lithology or geological age unit is of prime importance. Because of this, for engineering purposes engineering geological maps based more on engineering properties and interest should be prepared for slope stability studies. These engineering geological maps include the information from the geological maps, and should also make sure that the local or specific geology in the area of the investigation fits into the regional pattern. The Working Party Report (1972) outlines the broad principles of preparing engineering geological maps and plans.

3.2 Surface Data: Surface data is generally collected by two main methods: 1. engineering geological mapping, 2. remote sensing (generally vertical aerial photographs in New Zealand). Limited use has been made of other remote sensing techniques (e.g. infra-red, and radar imagery) in New Zealand. Vertical aerial photographs are generally studied in detail in the office using stereoscopic viewing and as their findings can be interpretative, the doubtful areas should be checked in the field.

The information gained from these methods is generally presented in one of two ways or combination of them:- surface engineering geological maps or landslide maps. Accurate topographic maps are needed as a base to plot this information. However where these are not available aerial photographs can be used as a direct base. This mapping should be done before subsurface investigations are initiated.

3.2.1 Surface Engineering Geological Mapping: Surface engineering geological mapping records all the available geology exposed on the ground surface. In many cases there is little to be seen because of coverings of topsoil, vegetation, etc. but in some areas, especially where engineering development and land use has been done, a lot of useful information can be gained.

Important details that are noted on the maps are:- the degree of weathering, defects present in soil or rock, nature of defect surfaces, distribution of lithologies and any variations in engineering properties, (e.g. clay bedding planes in generally massive mudstone), and structural geological features (e.g. faults, folds).

All the information put on these maps should be explained in terms of engineering geology as they are often used as a basis for subsurface investigations by engineers as well as engineering geologists.

3.2.2 Landslide Maps: Landslide maps are generally constructed from the stereoscopic viewing of vertical aerial photographs with field checking and additional surface geological mapping. The topographic expression of the ground is used to delineate areas of ground movement, and the scale of mapping is dictated by:

1. Type of landsliding visible or suspected.
2. Aerial extent of possible landslides.
3. Nature and scale of investigation. (e.g. preliminary regional verses detailed cut slope stability).

The maps generally show the limits of landslides by recording:

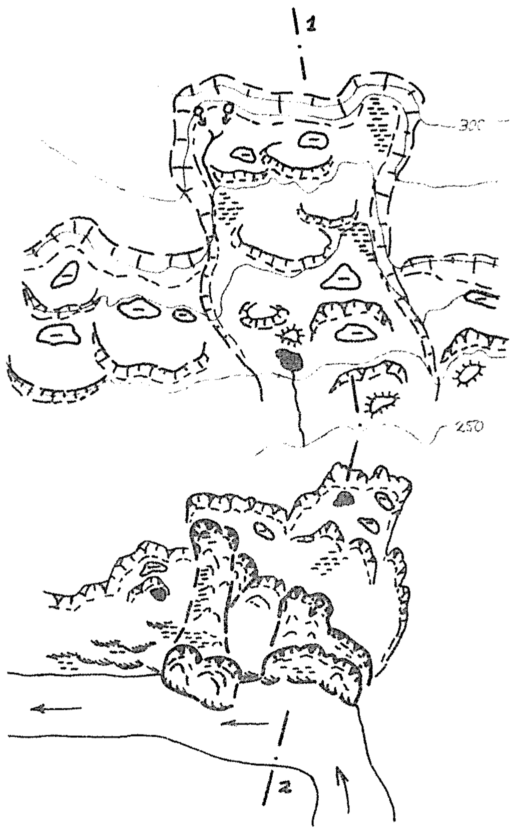
1. Morphological forms, (e.g. slip scars and scarps, tension, shear and pressure joints, depressions, bulges, abrupt slope changes, etc.).
2. Hydrological data, (e.g. springs, waterflows, wet ground, lakes, or alteredstream courses).
3. Anomalous geological structure patterns which record ground movement (e.g. gravity rolls).

Vegetation is often very helpful in defining movement (e.g. tilted trees) and aerial photographs taken with a time interval between them are most useful for recording recent movements.

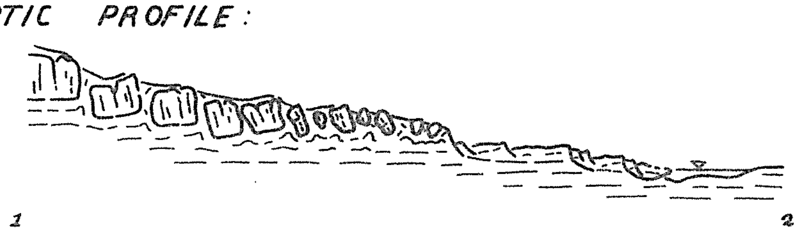
The maps should separate, if possible, the areas which have moved in the past, are moving at present, and those which may move in the future. If done properly these maps can give indications of the type of movement which has occured, the location of failure planes and delineate the areas which should definitely not be developed for engineering or land use purposes. Often projects can unknowingly be located in the middle of large ancient landslides which often necessitates an area larger than the project to be investigated. Slight disturbance of an ancient landslide can cause reactivation of the slide. (e.g. excavation of the toe of the slide).

The production of these maps has become very refined in Czechoslovakia (Rybář 1968) and an example is given in Fig 3. Normally colour is used in the production of these Czechoslovakian maps.

PLAN:



SCHEMATIC PROFILE:



KEY:

SCARP AND OUTCROPPING EDGES OF BURIED SLIDE BLOCK (FOSSIL LANDSLIDE)

OUTLINES OF EMERGING BLOCKS

SCARPS AND ACCUMULATION RAMPARTS OF TWO GENERATIONS OF RECENT POTENTIAL LANDSLIDES

RECENT ACTIVE LANDSLIDE REPRESENTING TWO DEVELOPMENTAL STAGES, WITH MAIN JOINTS

SPRINGS

WATER FLOWS AND LAKES

WET GROUNDS

UNDRAINED DEPRESSIONS

FIG 3: EXAMPLE OF LANDSLIDE REPRESENTATION ON THE SCALE 1: 5 - 10,000. TAKEN FROM RYBÁŘ (1968) FIG 3.

3.2.3 Landslide Susceptibility Maps: Landslide susceptibility maps have been prepared by the United States Geological Survey as a guide to regional urban planning in the geologically complex and landslide susceptible San Francisco Bay Region, (Nilsen and Brabb 1973). The map does not produce an analysis of slope stability but gives an expression of the average susceptibility of slope-material units to landsliding.

The method initially involves constructing, 1. a geological map, 2. a slope map which is contoured in measured slope increments and 3. a landslide map. The percentage of the map area covered by landslides and landslide deposits is estimated and then the geological map units are arbitrarily grouped into classes on the percentage area of them covered by landslides and landslide deposits. This measures a susceptibility of the geological units to landsliding. The existing classes are then compared with the slope map and the susceptibilities to landsliding are raised or lowered according to the abundance of landslides in each geological unit on the slope interval being considered. Landslides and landslide deposits are grouped in a separate class.

The map provides a regional planner with a statistical guide to the most landslide susceptible areas, based on the average landslide failure record in the past. The susceptibilities of flat areas at the base of higher susceptibility areas or landslides is not evaluated by the method.

The method has not been evaluated in New Zealand and should only serve as a guide to planners and not replace slope stability analyses.

3.2.4 Instrumentation: The most accurate and best method of monitoring surface movement of a landslide is surveying relative displacements of points (e.g. by tacheometer). Cruder methods (e.g. displacement of fences) are also used for measuring rapid ground movement. These methods are generally only used for long term studies on larger scale investigations, and maximum value can only be gained by positioning points carefully with reference points on stable ground which will not move.

3.3 Subsurface data: In a slope stability investigation where subsurface information is required, liason between the geomechanics engineer who wants quantitive data and samples for testing and the engineering geologist who wants to define and delineate the variation in lithologies, the depth of weathering, groundwater conditions, unstable zones, weak zones, slide or slip planes, etc, is essential to obtain the most accurate overall data. Although not all land-sliding is caused by either a purely geological or a purely engineering point of view. The investigation programme must also be flexible enough to be changed if subsurface conditions dictate.

The methods of subsurface investigations needed are dictated by:

1. Information needed.
2. Type of material(s) present and groundwater conditions.

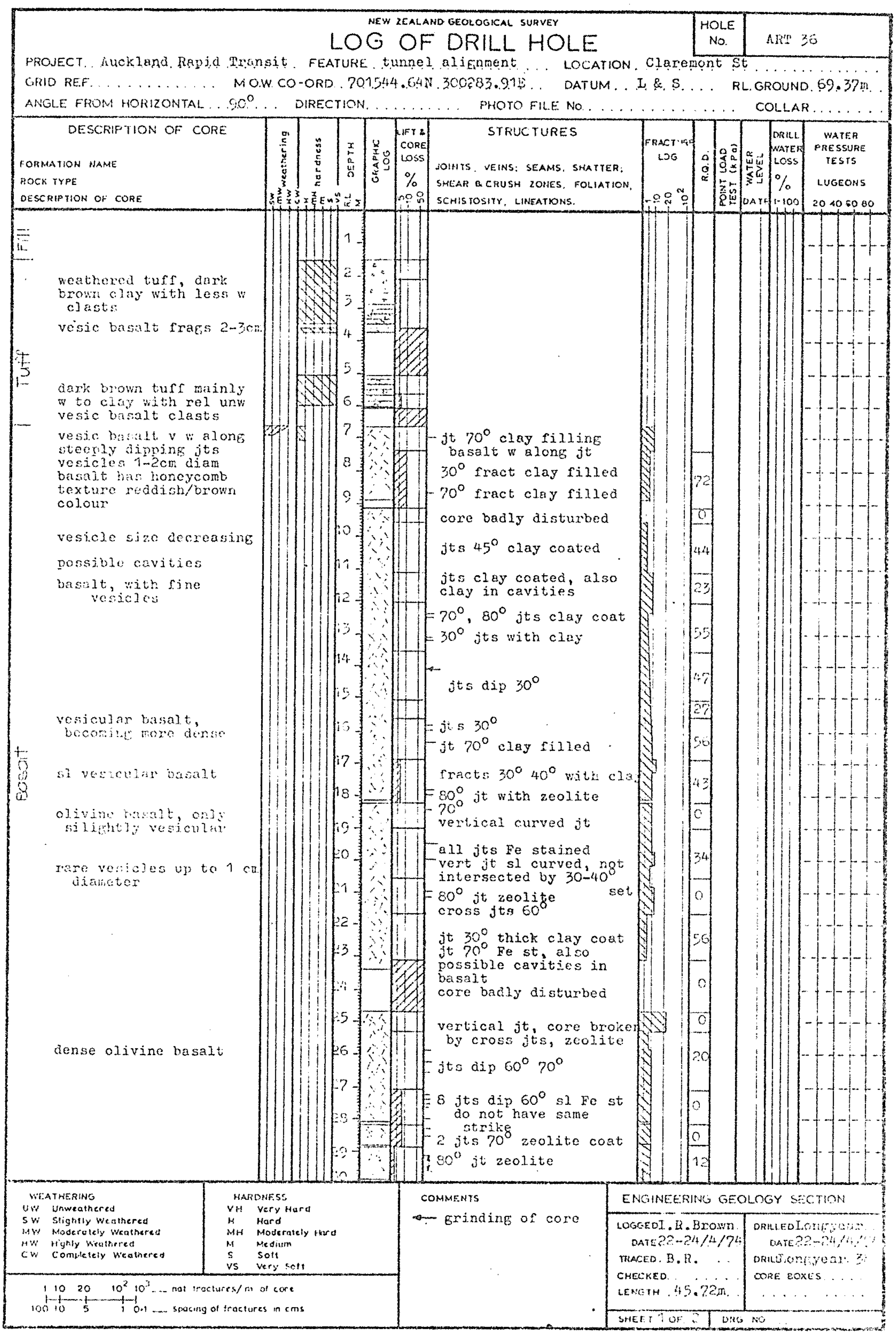


Figure 4 Sample log of Subsurface Investigations

3. Type of landslide (if already existing or may be later possibly initiated).

4. Cost, time, access and available plant.

Subsurface investigations should never be initiated until the nature and location of engineering development and a reasonable indication of the cause or likely cause of any ground instability are known. This is because it is very difficult to see features that one is not looking for (e.g. slide plane).

3.3.1 Methods and Logging: The best methods of subsurface engineering geological investigation are those where information is obtained by logging of material 'in situ'. This is because failure planes are generally very narrow and very difficult to recognise (e.g. Utiku slip plane is 10mm thick) and their true orientations as well as the orientations of other general geological features can be gained. Representative sampling of slide plane material is then also made easier.

For shallow investigations (< 10m) in easily excavatable material (e.g. soil, weathered rock) test pits and cuts give the best information. Where practical, sluicing or air cleaning are good for exposing hard rock. For deeper investigations, shafts (e.g. Calweld) provide the best information in soil, weathered rock and soft rocks, but have the disadvantage of not being able to penetrate easily past seeps, and below the water table. Diamond coring in similar country can be plagued with core loss, but is generally the only method used in rock that is too hard for shafting. Even then core loss can represent weak zones and failure planes. In some types of deposits only very expensive driven adits and shafts can be used to gain reliable direct information (e.g. landslide debris in Central Otago schists).

Geophysics (usually seismic or resistivity methods) is only occasionally used and when it is, adequate accurate correlation with direct means of investigation must be used. When geophysical conditions are favourable geophysics can be a very powerful method of investigation, however, when not used correctly or in the wrong conditions it can be totally misleading.

The extensive use of photography is recommended for recording all the directly observed data which is not sampled continuously (e.g. shafts, test pits). This is essential where direct on site liason with engineers or clients is not possible, and as all the information available is not always able to be recorded.

When presenting engineering geological logs, adequate space must be left for sample locations and results of engineering testing (e.g. permeability testing). The type of log sheet used by the New Zealand Geological Survey depends on the method of investigation used i.e. different format for shafts, diamond drill holes in rock, diamond drill holes in soil, etc. Test pits and drives are normally logged with as much actual graphic representation as possible. A sample log is included in the paper, (Fig 4.).

3.3.2 Instrumentation: Various methods of subsurface instrumentation for monitoring slope movements are available. The methods are primarily used to locate slip planes, general slope deformation or to monitor groundwater levels and piezometric pressures. They are generally used for longer term investigations or monitoring post-construction behaviour and hence normally found in the larger scale projects. The cost, availability and accuracy of the instruments in monitoring movements or fluctuations often dictates the types of measurements made.

The instruments usually are generally placed in bore holes or shafts after they have been drilled to obtain subsurface information. Tiltmeters and slope-meters which measure ground displacement either by degree of tilt or resistance of conductive material in the hole. Open standpipes and sealed piezometers which measure ground water (or seepage) levels and piezometric pressures are in common use around New Zealand. Whatever instruments are used, the maximum value is only obtained if regular and correlated measurements are taken.

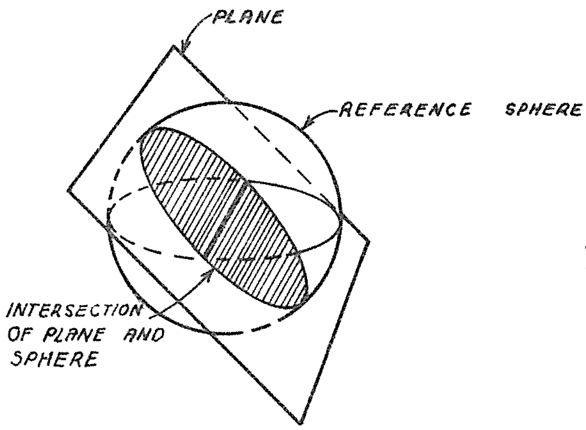
3.3.3 Maps and Sections: All the data available from surface mapping, subsurface logging, together with the location of the investigations should be presented clearly together with the location of relevant engineering features on maps and cross-sections. The cross-sections which generally give interpretations of available information should show the inferred geometries of slip planes or possible slip planes, distribution of lithologies, groundwater levels or any other information that may affect the stability of the ground in the section. The information on them should be kept to a minimum so as to give the clearest and best geological "model" for the basis of engineering analysis.

3.4 Other Engineering Geological Data: Data from other specific types of investigations are sometimes vital to slope stability studies, and are either used as a back up to normal engineering testing (e.g. mineralogy) or form the basis for engineering analysis. (e.g. stereographic projection).

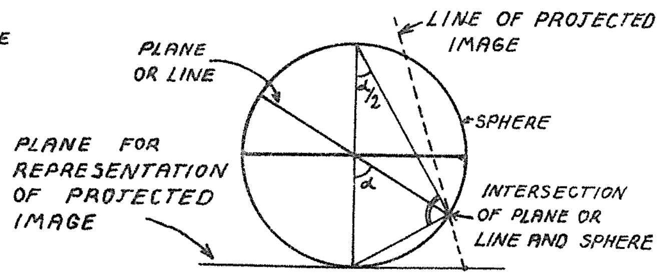
3.4.1 Mineralogy: Some rock and soil types contain minerals or mineral assemblages which can have a marked or drastic effect on the stability of slopes. X-ray and infra-red techniques analyse clay-silt size material for mineralogy (e.g. montmorillonite) and petrographic and hand specimen examinations are used for inspection of silt-sand size materials and rock fabric. (e.g. understanding the weakening of rock strength with weathering, or study of rock micro-defects).

3.4.2 Weatherability: The rate at which rock or soil weathers on exposure to air and/or water is important to the stability of a slope in addition to the degree of weathering (e.g. in Cretaceous-Tertiary age mudstones around New Zealand).

Several tests to measure weatherability have been tried in the past (Fookes et. al. 1971). Only the slake durability test, which was standardised by Franklin and Chandra (1972) will be described as it appears relevant for New Zealand conditions. In this test the breakdown of representative pieces of material to a size below 2.0mm on the application of a standard wetting and drying

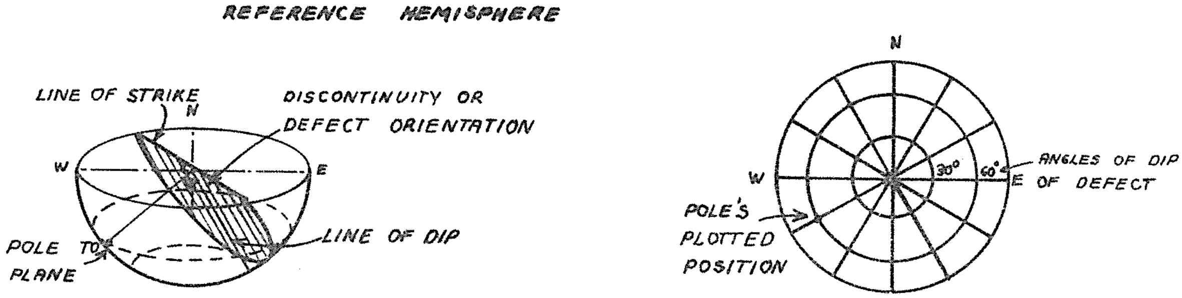


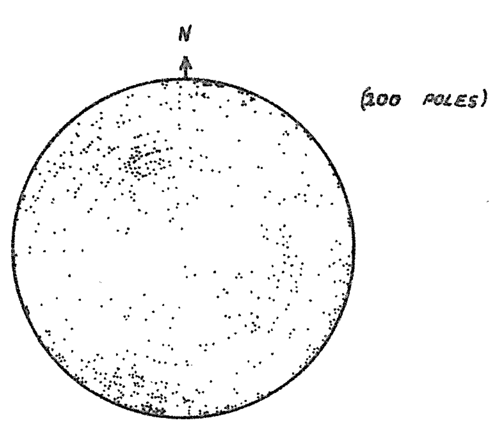
A) REPRESENTATION OF STEREOGRAPHIC PROJECTION



B) SECTION SHOWING METHOD OF REPRESENTATION OF EQUAL-AREA (SCHMIDT) PROJECTION.

c) METHOD OF REPRESENTATION OF DISCONTINUITY OR DEFECT ATTITUDE BY THE POLE OF THE PLANE ON AN EQUAL AREA PROJECTION OF THE LOWER REFERENCE HEMISPHERE





D) EXAMPLE OF PLOTTED DEFECT ATTITUDES BEFORE THE STATISTICAL STUDY PRIOR TO ANALYSIS FOR STABILITY

FIG 5: PRINCIPLES OF STEREOGRAPHIC PROJECTION SHOWING AN EXAMPLE OF PLOTTED ATTITUDES BEFORE ASSESSMENT procedure is recorded numerically as a percentage by the formula

$$\text{Slake durability} = \frac{\text{Dry weight above 2.0mm retained after slaking}}{\text{Dry weight before slaking}} \%$$

The method is only just beginning to be used in New Zealand, mainly on Cretaceous - Tertiary age mudstones.

3.4.3 Stereographic Projection: The simplest method of assessing the stability of rock or weathered rock slopes (i.e. where planar defect control the strength of the slope) is stereographic projection, a projection commonly used by geologists.

Stereographic projection is the representation of the intersection of planes and lines with a reference hemisphere (usually the lower one) when viewed from above the hemisphere (Fig 5). Hoek et. al. (1973) and John (1968) outline the concept and basis of the projection use for engineering analysis with worked examples.

The method, which analyses for kinematic stability can be used for -

1. An analysis of the effects of major individual discontinuities (e.g. bedding planes, sheared zone).
2. An analysis based on a statistical study of minor features (e.g. joints) which are too numerous and non-distincitive to be subject to study as individuals.

For an analysis the attitudes of the minor defects or major discontinuities in the rock must be recorded carefully using a compass and examining all the available geological exposure. The spacing and continuity of defects is assessed at the same time and the nature of the defect or discontinuity surfaces inspected to evaluate the angles of surface friction (e.g. is the surface clay coated or very rough ?). The angle of surface friction for a joint set or a discontinuity used for analysis is obtained by either 1. observing failures in the field, 2. shear box testing, or, 3. judgement. McMahon (1971) discusses the statistical evaluation of the initial plotted defect oritentations where a representative number of them must be measured before analysis.

In the analysis, some simplifying assumptions are made (John 1968) but has the advantage of being very quick. The method may be expanded to include external forces, water pressure in defects, and crude earthquake loading (John 1968) or variations in defect continuity (McMahon 1971).

4.0 EXTERNAL TRIGGERING MECHANISMS

While the majority of this paper has outlined the means of description of geological factors in slope stability, the geological factors themselves do not generally initiate failure during the engineering life of a slope until triggered by other conditions. This is often man himself but, in this paper, two natural conditions and associated effects will be mentioned - rainfall and earthquakes. The assessment of risk of slope failure from earthquakes and rainfall is difficult because the investigations and analysis are often done when the ground is stable and the potentially unstable conditions are very difficult to recognize and appreciate.

As well, conditions envisaged during investigations are not checked during construction when more potentially unstable conditions are sometimes uncovered. Slope failures have occurred in the geological past with the earthquakes and rain fall and to absolutely stop them is asking too much but we should reduce the risks to a minimum and avoid landslides caused by man's negligence.

4.1 Rainfall: Many landslides move after heavy rain, especially following dry spells. Drastic failure may immediately accompany this movement but also may occur at some later date (e.g. Baker slide, Peck 1967). The methods by which water provokes movement are many: (e.g. increased weight of soil due to saturation, high piezometric pressure in defects) and they vary with the geology and geological defects present. Man's intervention is one of the major causes of rainfall causing greatly accelerated occurrence of landslides (e.g. vegetation stripping of Tertiary age mudstones; undercutting of slopes) by altering the pre-existing stable or more stable conditions. When slopes are altered by man, the natural drainage and groundwater conditions existing before alteration and effects of the alterations on them must be understood for effective remedial and/or safe guarding measures to be taken. Water must be prevented as far as possible from penetrating to where it would not normally penetrate and cause instability.

4.2 Earthquakes: The occurrance of strong earthquake activity ($M \geqslant 6.0$ on Richter scale) is generally associated with renewed movement in active fault zones. The effect of an earthquake on slope stability, which is most drastic in the area near the epicentre, varies considerably with the topography, geology, magnitude and intensity of movement, and general climatic factors (e.g. dry versus wet ground). The different levels of seismic activity in New Zealand are associated with different regional tectonic regimes (G. J. Lensen pers. comm.) Regional areas which are in tension experience far more frequent earth movement (normal faulting) with lower magnitude associated earthquakes ($M < 7$) compared with regional areas which are in compression (reverse faulting) and have larger magnitude associated earthquakes ($M = 7$ or greater) at very much greater time intervals. Oborn (1974) summarizes the literature on seismic phenomena in relation to engineering geology.

The analysis of the effect of earthquakes on slope stability is very complex and one of the best guides to slope behaviour during earthquake loading is by comprehensive study of intact and failed slopes after an earthquake. The prediction of earthquakes is very difficult, but modern work is enabling the prediction that a large earthquake will occur in the future by monitoring the secular and precursory ground movements on known geologically active features. However, this does not predict exactly when it will occur, asthese movements may start up to $2 \div 3$ years or several hours before the earthquake.

Zoning maps against risk of earthquake damage should include an assessment of the risk of slope failure where possible. (e.g. Adams 1972). The zoning maps then have the same function as landslide susceptibility maps in showing high risk areas and providing a guide for land use and engineering but do not replace detailed investigations and analyses.

5.0 CONCLUSIONS

The paper has shown the main methods for the collection and presentation of geological information used by engineering geologists in providing as accurate, reliable and understandable information or geological 'model' for the basis of an engineering analysis of slope stability. However, to do the job properly there must be a clear, direct, liason between the engineering geologist and the geomechanics engineer to help bridge the bap between geology and engineering. The investigation programme must also be a continuing and flexible one as new information can become available throughout design and construction.

The reduction of the number of slope failures associated with engineering and land use to a minimum is essential and as geological factors provide a basic cause of many failures, these must be recognised and pointed out clearly before any failure occurs so that the dangers may be either avoided or designed against.

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Chairman: *Dr. M.R. Johnston*

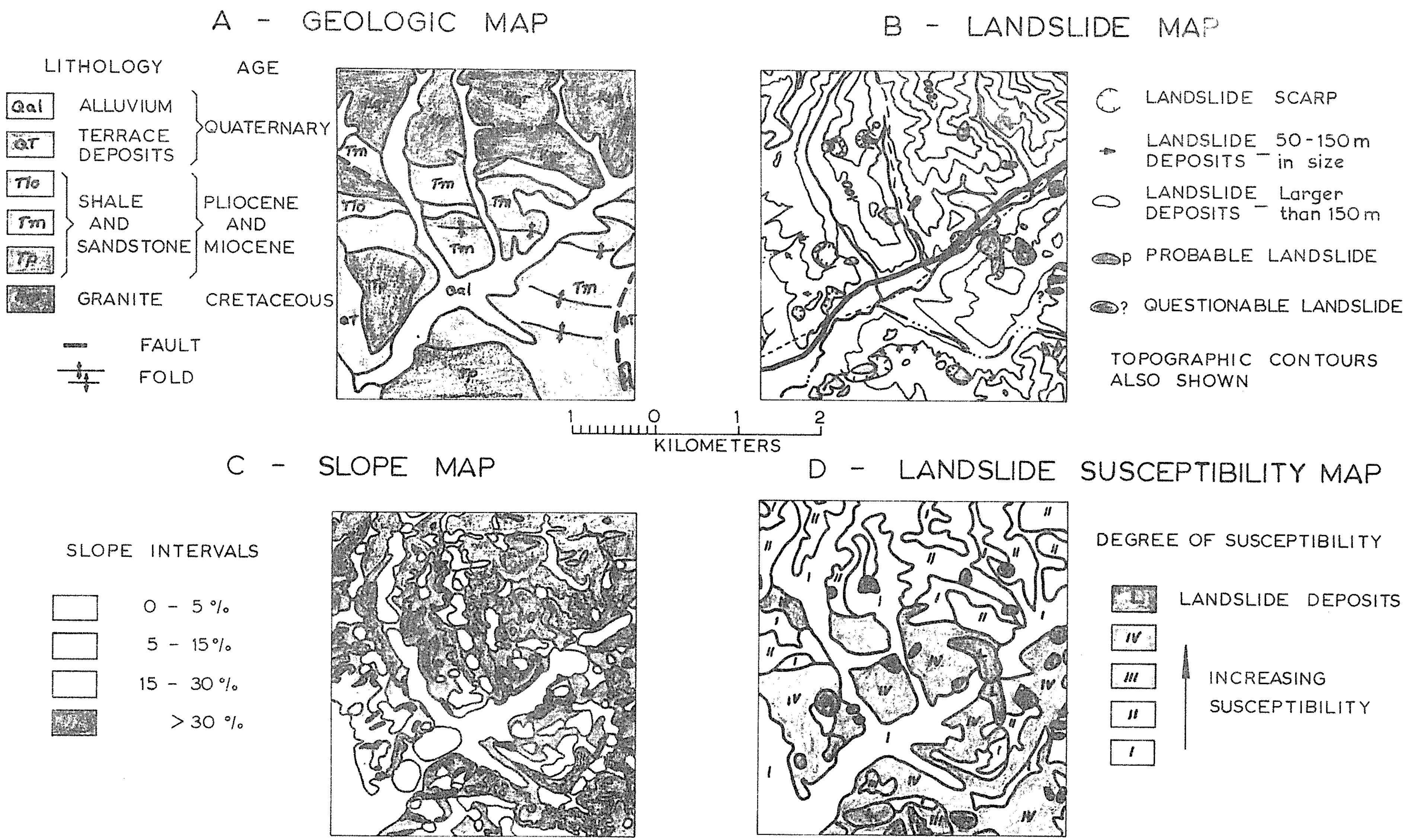
SECTION 4: GEOLOGICAL ASSESSMENT

PRESENTATION AND DISCUSSION

Mr. G.T. Hancox introduced his paper "Geological Aspects of Slope Stability" and showed some slides. The first was of the Buller Gorge area, showing the magnitude of slopes, and the landslides which occurred during 1968 during the Inangahua earthquake. There was a relationship between the angle of slope and the slope failures. He said that rock and soil varied greatly from place to place and the stability of a rock and soil slope depended on minerals, texture, structure etc. Common among the minerals which were deleterious to slopes from a stability point of view was montmorillonite which was widely found in tertiary mudstones in the centre of the North Island. The typical problems associated with the mudstones in the Taihape area were the slumps with a mud-flow or earth-flow at the bottom. In Utiku slide which affected the Main Trunk Railway and State Highway 1, the main cause of the failure was initially a rotational slide which intersected a montmorillonite clay layer. He said that the montmorillonite bearing mudstones and siltstones of N.Z. were causing a lot of problems in the Dunedin area and he showed slides depicting this area. Referring to the section on weathering he showed slides of the Wellington area illustrating the quite marked gradation of the highly weathered material at the top, grading down to largely unweathered material. Mr. Hancox said that alteration in the volcanic area was important, especially in the central part of the North Island and he gave as an example the Waihi fault scarp. He showed slides showing the relationship between this particular area and the area which failed. Further slides illustrated his remarks on the Oweka Slide in 1968; the Cromwell slide of schist (section 3.4.3); various types of faults; seismic activities; Mr. Hancox said that the mountainous country of N.Z. seemed to be prone to landslides during earthquakes and he showed a picture of the Stanley slide which occurred during the 1929 earthquake. It was about 2500 to 3000 feet in vertical extent. This had formed a lake which the river was not competent enough to cut through. Mr. Hancox said that all other points he wished to make were covered in his paper.

Mr. S.A.L. Read then presented his paper, "Engineering Geological Assessment and Slope Stability", He said it dealt broadly with engineering geology principles and particularly with aspects of slope stability. The basic first step was that the geological background and the geological framework concerning an area must be understood. This involved retrieval of all available data, which generally in N.Z. was from geological maps and industrial series maps, or in specific papers written on the areas. The data collected had to be put down in presentable and understandable form and this was difficult because of the difference between geology and engineering in language and terminology. Therefore, it was necessary to devise geotechnical language. The main points were the lithology of the material which was being considered, its strength, the defects throughout this material and the state of the weathering of the material, as set out in Fig. 1. It was desirable to get standardisation and quantitative data into these descriptions. With reference to landslides themselves, Mr. Read said the most powerful tool for recognising unstable ground was aerial photography. Stereoscopic pairs could be viewed and in some countries very detailed maps had been prepared from such photographs. The landslide would always have a morphological form which was recognisable from the air and these forms should be detailed on a map area. The typical forms were back scarps and front prows, with lakes and streams running across the area. These areas of catchment of water were very important as they served as reservoirs promoting failure through weakening or high pressure in the base of the slide. When preparing the maps a landslide should always be noted regardless of its state or its age. Then came the relative activity of the landslides, classified as fossil landslides, marginal cases and at the other end of the scale, landslides which were active at present.

After the surface areas of landslide had been delineated and a certain amount of risk assessed on the movements then one could make sub-surface investigations. The most reliable method of obtaining data for landslide purposes was just direct observation. Every piece of geological data should be shown on an engineering geological section. The bore holes and the variations in lithology up the bore hole should be shown and the slide plane drawn in. Most importantly, fact had to be sorted out from opinion. Where inferences were made this should have been very clearly stated. Basically, engineering geology



MATERIALS USED FOR A LANDSLIDE SUSCEPTIBILITY MAP
— EXAMPLE FROM NILSEN AND BRABB (1973)

data must recognise all the important factors involved in slope stability. There must be liaison with the engineer and the testing that must be done with the engineer must be discussed to help him to get the most accurate quantifiable model for analysis that is possible. Mr. Read said that the geological factors which had been outlined in Mr. Hancox's paper generally controlled landsliding but they generally needed to be triggered by something else. This was the difficult part of the assessment. He said that earthquakes produced the most spectacular types of failures and were also the most awkward to analyse. In some ways a geologist could help by providing background in that the different tectonic regimes around the country-side would provide different effect from earthquakes on the stability of the ground near the epicentre of the earthquakes. He then referred to a paper, "Landslide Susceptibility Maps" and showed slides to demonstrate a particular method of mapping used often in the San Francisco area. The investigation should be done before the area is subdivided. Once all the landslide deposits had been noted then one must statistically count up the percentage of the area of the geological unit covered by the landslide. This would give the susceptibility of that unit to landsliding. Then a slope map must be prepared after which the susceptibilities are raised or lowered according to the slope. The method is depicted in the diagram p.4.33. (The poor reproduction is regretted - Ed.)

Mr. Farley referred to the Buller Gorge area after the earthquake and asked Mr. Hancox what was the correlation between landslides and the degree of slope and what percentage of the land was actually involved in slips.

Mr. Hancox said it was not done on a percentage basis but it was quite apparent that the bulk of the landsliding affected the slopes which were steeper than 45°, or more accurately 36° in that particular time. This was in fact a steep slope in this area. He thought that probably not more than 15% of the country over 36° was affected.

Mr. Riddolls said that in his experience, most of the slope failures in cut slopes in Wellington were related in some way or another to obvious geological defects. There must surely be grounds for mandatory investigation of any areas such as those outlined by Mr. Read, prior to development; failing that at least before any properties were subdivided.

Mr. Evans said that according to Mr. Hancox' paper, montmorillonite was dangerous material to have in the ground. He asked if there was anyway of recognising this mineral without having an electron microscope.

Mr. Hancox siad that in some deposits, the clay could be recognised if it was behaving in an unusual fashion, slippery, soft and plastic and one of the sure tests was the slaking test in water. If some was put, in a relatively dry state, into a jar of water, one could see the expansion taking place. The other way was to have an x-ray or infra-red analysis. If there were only a small amount of montmorillonite, there may not be any field tests to identify it as being present. If it was causing particular problems it would show up by x-ray analysis.

Mr. D.K. Taylor said that the most interesting piece of the discussion came with the description of the landslip susceptibility maps as set out by Mr. Read and the fact that these must be done at the time when the land was zoned and basic decisions made about its use. He had been intrigued by the statement that some of the landslides were 'fossil' landslides and therefore would not move again. He felt that perhaps that conclusion could not be drawn and would like to know by what standard one could decide that a very old landslide would not move again. This raised a pertinent question about the road going up the Waihi slip at the bottom end of Lake Taupo. What was all this crying about engineers taking total responsibility, if society was prepared to put a road in that position with people travelling up by car every day. Then on the other hand people ask for an engineer's certificate that a particular piece of land is stable. He felt the scale was something one had to try and reconcile in one's own mind.

Mr. Read said that a fossil landslide was stable under the conditions which were existing at the time. Engineering development could reactivate it, but under the present conditions it should not move again. As to the other part, it was engineering judgement to find out whether it was stable or unstable. Not just straight engineering, but engineering geology.

Mr. Hancox in referring to the Waihi slide, said there was a State Highway passing through the centre part of the affected area and a number of motels, expensive houses etc. in the path of the future failures. These people had been warned of the existing dangers. Some people had expressed interest to find out what the likelihood of further failure was. It was hard to be quantitative about the frequency of this sort of thing and he did not know what could be done. Perhaps there should be some sort of legislation to say it was a dangerous area and one could not build there or remove what was there.

Mr. Millar commented on Mr. Read's suggested use of the point load tester for assessing strength. An evaluation of the tester at Central Laboratories had shown that the machine was poorly designed and results obtained did not agree well with the theory presented by the developers (1). The anomaly appears to arise from an apparent desire of the developers to obtain a strength index with the dimensions of stress. The point load strength index, Is, was defined as:-

$$Is = \frac{KP}{d^2}$$

K = constant

P = load at failure

d = distance between platens (diameter of core in diametric test).

This equation had some support from the ideal elastic stress analysis but did not take into account the influence of other factors such as the increased probability of a defect with increasing d. Corrections such as relative brittleness indices, shape factors and crack propagation factors had also been suggested (2).

The point load index is not independent of d and a correction is required when comparing results obtained on cores of varying diameters. An analysis of results presented by the designers (1) showed that for an index independent of d, the equation would have to be modified to the

$$Ksmod = \frac{KP}{d^x} \quad \text{where} \quad x < 2.0 \quad \text{(generally between 1.0 and 1.7)}$$

thus, a value of x must be determined for each material whenever results on cores of different diameters are compared. An alternative might be to investigate the possibility of relating the term x to a visual or otherwise-established homogeneity index.

Some modifications to the design of the point load tester were desirable. The gauge should be relocated to prevent damage during brittle rock failures and a more effective wiper seal should be used to protect the ram seal from dust and rock fragments. The seal on the Central Laboratories tester was damaged and required replacement after only two months service, (250 tests). Damage to the platens, base and distance measuring system were also apparent.

<u>References:</u> supplied by Mr. Millar

1. BROCH, E., and FRANKLIN, J.A. The Point-Load Strength Test Int. J. Rock Mech. Min. Sci, Vol. 9. p. 669-697, 1972.

2. REICHMUTH, D.R. Point Load Testing of Brittle Materials to Determine Tensile Strength and Relative Brittleness. Status of Practical Rock Mechanics. 9th Symposium on Rock Mechanics, Colorado, 1967.

A participant outlined his experience at Turangi where it was found there was fairly good agreement between the axial compression strength and the point load test results. This related to the strength of the intact rock and did not have any bearing at all on the defects.

Mr. Depledge asked Mr. Read to elaborate on the use of stereographic projection in analysis of slopes; he felt that this was one of the major contributions engineering geology could make to slope stability analysis.

Mr. Read replied by saying that stereographic projection was a representation of planes and lines in space. He described the method and illustrated his remarks with slides.

Mr. Depledge asked whether Mr. Read considered one should analyse the slope in terms of all the joints present, or just use a subjective analysis of sets of joints.

Mr. Read replied there were two ways this could be done: one was to take a purely statistical point of view and contour the plots and what was then considered statistically the most relevant joint feature would be analysed; The other was to simply assess that a particular joint was the important one and analyse that. An experienced person can normally pick up the more important joints.

Mr. Riley asked if this were being used in rock which might have 100 joints per cubic foot, was it necessary to plot a pole for every joint or first of all decide that there were preferred orientations?

Mr. Read said that with such broken rock, the question was whether it should be analysed by soil mechanics or rock mechanics. Some judgement was required.

The Chairman summed up by saying that the two speakers had made a very good attempt to present an insight into how a geologist goes about appraising slope stability, and thanked them for their contribution toward the Symposium.

A CONTRACTOR'S VIEWPOINT AND RECOMMENDATIONS

N. S. SMITH

1.00 INTRODUCTION

Speaking as a contractor specialising in landslide prevention and remedial work, some 50% of the soil movement problems on which we are called upon to carry out remedial works, need never have happened, had the average man in the street sought out and taken advice from people more experienced in soil engineering, or in the building of retaining walls.

1.01

For every major slip necessitating relocation of a highway or railway, or severe damage to one or a number of properties, there are some 30 to 40 minor slips or dropouts which require immmediate clearing and/or remedial treatment. We class a major slip as one which necessitates major reconstruction work to a road or railway, or where property valued in excess of $100,000 is affected. During an average year we would be involved, in one way or another, in slip remedial work on properties having a total capital value of well over $1,000,000 excluding public roads and railways.

1.02

One cannot always predict potential land movement in natural or cut slopes, but we are concerned at the growing number of sites we come across, where soil movement had been observed before erection of a structure on or adjacent to an unstable slope. Whilst the majority of architects, engineers, and surveyors will take steps to investigate and remedy the situation before erecting structures, there are, unfortunately, a number of people who ignore the situation either because of lack of experience with slope stability problems, or because they can "pass the buck". To give you an example of the latter, in late 1970 in Auckland, a major slip occurred which damaged two houses and endangered another four valuable residential properties. A soils investigation and corrective remedial work were subsequently carried out, and the Earthquake and War Damage Commission was involved in at least one substantial claim resulting from this slip. Today, just four years later, one can walk along public property at the toe of this slip, look up and observe a very large swimming pool which has been built on the head of the slump, immediately in front of one of the damaged houses. From enquiries we have made, we understand that the Local Authority concerned issued a permit for the construction of this pool, and to the writer this is a shocking state of affairs.

1.03

Whilst this example is extreme, a similar situation exists in other areas where new houses are today being built on ground many of us know to be potentially unstable. Revetments and retaining walls are being built on these properties, by amateurs, to hold up massive natural slopes above or below their homes, and we frequently encounter situations where walls have collapsed as a result of underdesign, or merely poor construction. It is horrifying to think of the very costly damage that is likely to occur on some of our larger areas of hillside homes, as a result of unstable slopes and badly constructed walls after a severe storm or in the event of an earthquake.

1.04

The terms, unstable slopes and badly constructed walls are emphasised, because the majority of our correction work results from:

(a) Lack of attention to existing visible signs of instability in soil slopes.

(b) Adoption and construction of standard batter slopes and retaining structures without proper regard to soil type and its proneness to movement.

(c) Poor selection of materials and/or design a particular structure.

(d) Incorrect siting of services and structures relative to existing natural features.

(e) Poor constructional work, usually attributable to the nature of the site, inexperienced personnel, or lack or proper equipment.

1.05

Whilst much has been written on the various methods of land slide prevention and correction, sound practical experience is also required so that one can correctly analyse and assess the basic problem, and utilise the prevention or remedial technique most suitable to that particular site. It is appropriate to review our own particular experiences with regard to failures of cut slopes and retaining structures, and to review some of the less well understood techniques and types of materials used to improve their stability.

2.00 COMMON TYPES OF SLOPE FAILURES:

The most common type of slope failure we encounter is that of a too-steeply-battered bank simply collapsing because the supporting toe and/or protective plant cover has been removed. Another common slope failure we encounter results from a cut slope inadequately retained with a veneer of simply mortared stone, a lightweight cribwall, or concrete block wall often built without a footing or reinforcement. Obviously, the fault is not in the materials used, but in the choice of materials and design for the particular situation.

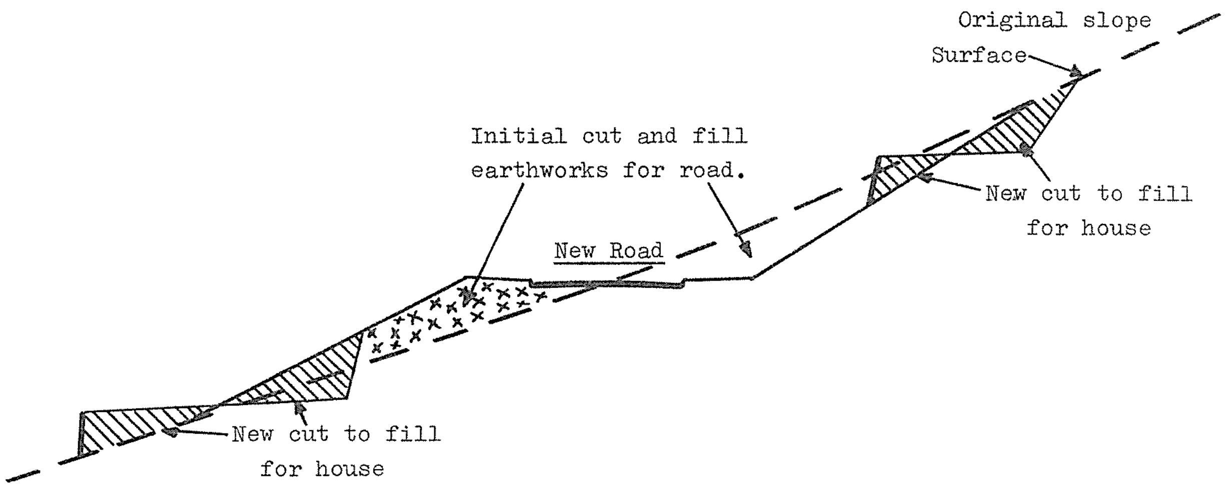
2.10 Failure of Steeply Battered Slopes

Failure of steeply battered slopes is most common on major road and railway cuttings usually where such batters are necessarily steep to keep within legal boundaries. They also occur on hilly subdivisions where the natural slopes have been cut and steeply battered to meet the newly formed road, and road reserve levels.

2.11

On residential and industrial properties, the failures usually occur where the owner or developer has cleared the site and cut into the slope to form a driveway or level building platform without any regard to stability of the resulting batter - so long as the job looked alright! Unfortunately, these people have often been ignorant of the fact that they have cut into a slope already steepened during original subdivisional earthworks (see sketch 1). The situation is more common than realised and very dangerous because several houses can be situated on the same slope in the vicinity of the failure.

SKETCH No. 1.



2.20 Failure of Slopes Due to Inadequate Retaining Walls

These situations are generally associated with residential properties where the owner realised that a bank required retaining, but failed to obtain proper advice before construction. Most of these walls have been built by the owner himself, or he has engaged a local general contractor with limited knowledge of soil behaviour. Between them, and sometimes with the assistance of the "resident cook", a visually attractive wall has been designed to suit the funds available, but not to withstand the thrust of the earth behind it. It is perhaps relevant to note that most people responsible for this type of structure, seem to have been aware that drainage was necessary between the earth and the back of the wall.

2.21

Failure involving a concrete crib wall is usually due to poor design and/or construction. Crib walls are easily constructed, they are seen revetting slopes all over New Zealand, and it is perhaps natural that the average Kiwi homeowner and local general contracter so frequently choose this type of retaining wall. The crib wall failures we encounter usually occur in known unstable areas, and to a very disturbing degree have often been designed or recommended by an architect, surveyor or engineer who has either failed to recognise, or accept, the fact that he is dealing with an unstable soil mass.

2.30 Failure Due to Undercutting by Water

A less common failure is that where a retaining structure has been undermined by river or wave erosion. In nearly all river situations the failing structure has either been founded above the bed level, or the effect of scour at the base of the structure has been grossly underestimated.

2.31

From our experience in remedial works, the common most factor applicable to nearly all failures we have encountered, is that a revetment type structure has been built in an unstable area where the situation called for a retaining type structure.

3.00 THE NEED FOR LEGISLATION:

Precautionary and remedial works are difficult and sometimes impossible on private properties, because individual owners seldom have sufficient funds to pay for the full and proper measures necessary. Also, when neighbours are involved they seldom agree on the extent of actual remedial work required, nor to the precautionary work often necessary on the flanks of a slip, and seldom want to contribute to the costs of the work in any case.

3.01

So long as our present system allows us to pass the buck as quoted earlier, and whilst we continue to allow inexperienced engineers, surveyors, contractors, accountants, butchers and milkmen to tear our natural slopes apart, the situation will get worse. As a specialist contractor, the writer is aware of numerous examples of ridiculouly inadequate walls revetting massive slopes known to be unstable where houses have been built, and are continuing to be built. He is aware of one situation where a portion of a new blockwork house was built over a tension crack in spite of the owner being advised not to, prior to construction commencing. In another situation where a deep slip scarp exists within four feet of an existing house, the owner is awaiting the time when the next soil movement will damage the house so that he can claim on the Earthquake & War Damage Commission.

3.02

If engineers regularly involved in slip remedial works will seriously review their own past experiences, they will probably recall similar situations to those above quoted. Since the Earthquake and War Damage Commission is now bound to cover damage to property by landslide, we are all going to have to pay towards the costs which, the writer suggests, will increase substantially in the years to come.

3.03

The writer has given the situation much thought and would suggest, somewhat reluctantly, that the best possible precaution would be for Local Authorities to restrict uncontrolled earth works in certain areas. Most Local Authorities are aware of particular unstable areas in their own localities and at the time of proposed subdivision, they should insist that all earthworks in such areas should be designed and supervised only by engineers with facilities and proven ability to deal with soil mechanics problems. Proven ability could be easily defined as an engineer engaged primarily in soils work for say, 75% of his working time. This is not necessarily putting an unreasonable responsibility on to the soils engineer, who is already working in his chosen field, is experienced, and probably already aware of potentially unstable areas in his district, and who would have the advantage of being able to assess the situation after his own thorough investigation and analysis.

3.04

In addition, Local Authorities should bring in a By-law preventing "machine excavation" in such areas unless the natural toe support when removed, is replaced immediately with a properly designed and built retaining structure, not merely a pretty wall or revetment. By using the term "machine excavation", Local Authorities would automatically have a simple checking and restrictive device. If a contractor knows that he will violate a By-law and possibly be liable for any damage resulting from his operations, he is going to ensure that the owner of the property concerned has first obtained Local Authority approval and/or a permit to carry out the work. The system envisaged would be similar to that already existing, whereby Local Authority drains cannot be tampered with, until a registered drainlayer has taken out a permit. In our case, the owner of the property would have to satisfy the Authority that the earthworks and the retaining structure proposed have been designed and will be supervised by an experienced soils engineer. However, we cannot and must not expect miracles from the soils engineer, but at least the Earthquake and War Damage Commission, Local Authority, Subdivider, Architect, Builder, the future owner of the property and his neighbours would know that all possible precautions are being taken to prevent landslide, subsequent property damage, and expensive remedial works in known unstable areas.

4.00 METHODS OF STABILISING SLOPES:

It is well established that water is the most common causes of soil movement, and it should not be necessary to say that any precautionary measures which assist rapid removal of water from the surface and subsoil is a positive and essential step to improve stability.

4.10 Cut-Off Drains

The aim should be to lower the natural ground water level on sloping soils and to reduce piezometric pressure. This can best be achieved by construction of a deep and positive cut-off drain, sited at the top of the slope well behind calculated slip circles. Too often, cut-off drains are constructed too close to the top, or on the side of a cutting and within a potential slip circle. This is very bad practice.

4.11

It must be borne in mind that although a smooth bore perforated PVC pipe will rapidly remove surplus ground water, the bottom of the trench is not level and in fact allows water lying below the invert of the pipe to constantly flow along, and permeate into the underlying subsoil. In weak soil zones, this continual seepage will reduce shear strength of the soil in a plane and, in the writer's opinion, has been the cause of many a slip. Cut-off drains, to be properly effective must be 10' to 20' deep with the invert level well below the natural ground water level, and the trench must always be sealed off at the top to prevent ingress of surface run-off water. Surface water should be led away along a bituminous sealed invert, concrete half pipes, or similar surface drains.

4.12

For reasons above stated and for ease of construction, all service trenches should be sited well away from potentially unstable areas, and where necessary to traverse a suspect slope, should run straight down the slope rather than across it. To prevent washing out during and after construction, steep service trenches, and indeed any trench running down a steep slope, should be locked off every25' or so (depending on steepness of slope) with non-rotting loosely packed bags of sand or scoria. Remember that all service trenches in hill country are potential ground water and surface water traps, and a low cost PVC collector pipe installed during the construction period is an excellent insurance against slipping, in the long term.

4.20 Horizontal Drains

On very high cuts and slopes where seepage problems are encountered, unstable areas can be economically treated with horizontal drilled-in drains. These consist of perforated pipes, usually, of PVC, which are drilled into the slopes at a slight angle above the horizontal, and facilitate rapid removal of groundwater where a positive cut-off drain is impracticable or insuffieciently deep to be effective. Horizontal drains can easily be installed up to 100ft, in length and under favourable soil conditions, up to 200ft.

4.21

TYPICAL HORIZONTAL DRAINAGE INSTALLATION

Discharges in gallons per day

DATE	PIPE No.1	No.2	No.3	No.4	No.5	No.6
June	1440	111	254	Nil	480	617
August	1270	108	223	Nil	450	982
October	771	70	150	Nil	327	654
February	1121	122	129	Nil	227	576
April	1350	154	170	Trace	296	720
Average Daily Flow	1190	113	185	-	356	710
Average Annual Flow	434350	41254	67525	-	129940	259159

Average daily flow of 6 pipes	=	2554 gallons
Total Perforated Pipe installed	=	793 ft
Total drains in installation	=	6
Average length of pipe	=	132 ft
Average discharge all pipes	=	932, 219 gallons per annum
Average discharge per pipe	=	155, 370 gallons per annum
Maximum discharge one pipe	=	434, 350 gallons per annum

4.22

Diameter of drains need seldom exceed 2in, and only on two occasions in the writer's experience with horizontal drains has he seen any real necessity to go above 2in, in diameter. In both of these latter cases groundwater flows in excess of 50,000 gallons per day were encountered behind major earth movements. In the majority of cases, 1in nominal diameter drains will adequately cope with normal ground water flows and are easier and more economical to install.

4.23

The equipment necessary for installation of horizontal drains consists basically of a light, protable horizontal drilling machine. The locally made Geodrills are about 7ft 6in in length and, weighing between 150 lbs and 360 lbs depending on its size, can be very easily manhandled into areas where operation of other equipment is impossible, e.g. on a soft soil slump. Whilst an 8ft wide bench is ideal on which to operate, these small drills can operate on the side of hills and banks simply by raising one end on a light H frame. Current cost of 1in and 2in diameter drains in-place vary between \$1.50 and \$2.50 per lineal foot, depending on soil type, access and locality.

4.24

The first horizontal drainage in New Zealand, to the writer's knowledge, was that carried out for the Ministry of Works on an unstable slope on S.H.1 on the Orewa Hill about 1955. To date, this batter is still standing as it was cut, and is probably one of the best examples of precautionary horizontal drainage schemes existing in New Zealand.

4.25

In addition to horizontal drains, it should be noted that drilled-in drains can be installed on any inclined angle required, and so reduce piezometric pressure well below the point of installation. A notable example of this type was carried out at Pakuranga in 1970 and effectively reduced movement of a large landslide affecting a number of residential properties.

4.26

In a similar manner inclined drains can be installed without difficulty to reduce seepage and spalling of near vertical cliff faces. On high slopes, particularly where access is difficult for heavy mechanical plant, it is sometimes economical to drill large diameter vertical drains well behind the top of the cliff or slope, and to drill in horizontal drains to intercept the vertical drains. This type of installation is usually more efficient than horizontal drains alone, and if closely spaced are often more effective than a cut-off drain along the top of the slope because of the greater depth, and lowering of the ground water level. Skilled operators are required for this work, however, and generally depths in both directions should not exceed 70 to 80ft because of the accuracy required.

4.27

In all precautionary and remedial work, the aim must be to reduce the piezometric pressure and lower the ground water level to increase shear strength of the soil and reduce its weight, so that the whole of the stabilised mass acts as a strong buttress to support the deeper and more inaccessible portions of the slope.

4.30 Plant Cover

A continuous cover of vegetation is necessary on all slopes to prevent surface soil erosion and to provide a pleasing environment. On existing slopes where underlying soil may be unstable, stability may be improved by good plant husbandry. The aim should be to gradually improve the quality, and height of grass and shrubs on unstable slopes to provide a better and more suitable root system. Tall, ranck, broad leaved weeds and kikuyu grass should be progressively grubbed out or killed off with selective weed killers, taking care not to destroy the soil binding effect of the existing rooting system which would allow erosion to begin.

4.31

Fortunately hydraulic seeding and mulching has now become a well established method of surface slope protection in New Zealand. In most hydraulic seeding projects, the optimum time for application is between March and November. Application during winter months is not harmful because the seed does not germinate during cold weather, preferring to wait until ground and air temperatures are warm enough to bring on germination and support growth.

4.32

One of the most important points in all seeding, is to ensure that sufficient humus and nutrients exist or are added to the soil to support growth after germination. This aspect is frequently overlooked, particularly on deep newly formed cuts where nutrients just do not exist, are locked up in very plastic soils , or have leached out of permable soils such as sand and pumice. In these areas lacking top soil and humus, it is necessary to apply balanced doses of nutrients at time of seeding, and in such a way that the nutrients are released gradually to support plant life until such time as the natural humus making cycle is established.

4.33

In cut and bared slopes consisting of fine grained soil, a paper or straw based mulch consisting of an adhesive material, innoculants, nutrients and seed is usually the most satisfactory method of establishing growth. The type of seed used must be compatible with the pH values of the cut soil and may necessitate a variety of seed species, rather than only one type. The pH value of the soil can vary considerably in a distance of only 300' and should always be taken into consideration. On steep cuttings, and on slopes where topsoil has been respread over the surface, the natural tendency to use large proportions of clovers should be avoided. Experience has shown that after heavy rain, the weight of the water and hamper growth can be too much for the shallow root structure and surface sheet slipping results.

4.34

In granular soils such as sand and pumice, a mulch is not quite so necessary because the aqueous solution will pemeate into the outer surface of the soil taking with it the nutrients necessary to support growth. The adhesive, which can be latex or bitumen based, will secure the nutrients and seeds to the soil particles until germination and a rooting system are well established.

4.35

The problem of sheet slipping and surface runnelling can be overcome by the utilisation of loosley woven polypropylene meshes, such as Geomesh Leno, which can be laid over the slope area and penetrated by future growth.

4.36

A precautionary measure that has been largely overlooked in New Zealand to date, is that of planting small, selected shrubs and plants on large slope areas to help prevent erosion and assist stabilisation. It is perhaps not well known that in Melbourne and Los Angeles, it is now necessary to submit a plant programme with a schedule of estimated costs when depositing plans of proposed subdivisions and earthworks to the approving Local Authority. This precautionary measure of planting would certainly help prevent erosion of cuttings and slopes, particularly on new subdivisions, and at the same time provide a very pleasing landscape.

4.40 Lime Stabilisation

This is not a common method of permanent stabilisation, but is a very useful temporary method in a situation where a slip of highly plastic soil has to be held during the winter months, until accessibility improves. A typical situation would be a rotational slip above a road or a house. Vertical holes are bored in rows across the slipped material commencing from a point about 10' below the scarp. The boreholes are then filled with hydrated lime, the action of which flocculates the finer soil particles into coarser particles, so increasing friction in the slip plane. It is necessary to keep the vertical holes topped up with water and lime in a slurry form for 3 or 4 weeks so that the lime will seep through the slip plane. Costs of lime stabilisation vary considerably according to depth and diameter of holes and accessibility, but an indicative cost for 6 to 8in diameter holes is $3 to $4 per lineal foot of borehole.

4.50 Grouting

Cementitious grouting of slips in natural ground is not common in New Zealand, probably because very little written information is available, and also because cost factors cannot be readily obtained. Nevertheless, cementitious gouting has a place in stabilising slips and certainly as far as railways are concerned, grouting is considered to be one of the most satisfactory and economical methods of slip stabilisation available.

4.51

In carrying out cementitious grouting, points are first driven through the slip debris to a point below the slip plane, at about 10 to 12ft centres on a diamond pattern. Grout is then pumped in at fairly high pressure initially, and this reduces substantially once the initial resistance is overcome. The grout penetrates the slip plane and weak soil zones in the vicinity of the injection points as they are slowly withdrawn.

4.52

Whilst the volume of grout can vary considerably from 20 to 80 cu.ft per point, a closely indicative estimate of total grout required for a particular situation can be taken as approximately 1/100th of the volume of the material to be stabilised. Current all in cost of grouting on large slips varies from approximately $1.50 per cu.ft for a 4:1 sand/cement mix, to approximately $2.50 per cu.ft. for neat cement. On small slips involving only 4 or 5 days work, the above costs could be doubled.

4.60 Crib Walls

These are most satisfactory when retaining earth which has to be tipped, and for retaining materials of a pervious nature, (Civil Engineering Code of Practice No.2 Earth Retaining Structures). Before constructing a crib wall it is most important that the bed be evenly graded and closely within tolerances allowed. Any undulation or variation in the bed will follow up in the structure itself and, in extreme cases, put undue stress on inter-connecting members. Back filling should proceed as the crib is erected, and be well compacted.

4.61

Crib walls should be used with considerable caution when placed at the toe of slopes where soil conditions are unstable and liable to slip. Similarly, in river and coastal situations cribs should not be used where the toe resistance, or structure itself, may be subject to scour and washout unless protected by a flexible apron such as gabions. Current cost of construction of crib walls per sq.ft. of face area is approximately $3.00 for 2ft., $3.75 for 3ft., and $4.50 for 4ft. thick crib walls.

4.70 Gabions

Gabions are one of the most suitable materials for revetting and retaining slopes because of their weight, free drainage properties and ability to flex and still remain structurally sound.

4.71

On slopes less than 45°, revetments are usually formed with 9in or 20in thick mattress units, and on steeper slopes these are reinforced with stakes, or with counterfort gabions at approximately 12ft centres. One metre high gabions can also be used on steep slopes by stacking them one on top of the other with a step-back of 12 to 20in for each gabion course. Gabion retaining walls are designed as normal gravity type retaining structures and the lower course should, where possible, consist of headers and be at least 2/3 of the wall height in width.

4.72

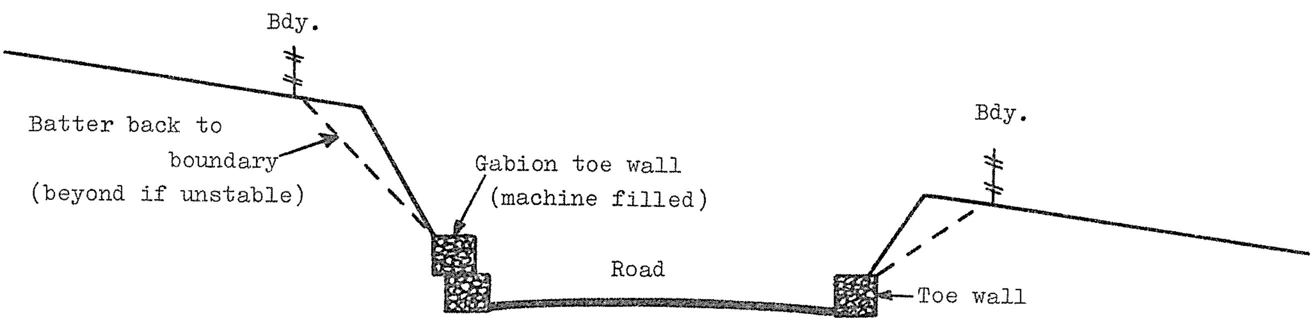
Where undercutting by water may occur, the wall can be founded on a gabion apron, the end of which will flex downwards as scour occurs and so protect the wall proper. The protective apron length in front of the wall should be 1½ to 2 times the expected depth of scour. Prior to construction of gabions, only a roughly levelled bed is required because the individual courses can be evened up as construction proceeds.

4.73

For retaining walls, toe resistance is required as with any other gabion wall, and the units must be laced tightly together. In most gabion work a much stronger and tidier job can be achieved by stretching the gabions prior to filling, and also by tying the front face to the back face of the units in each compartment.

4.74

Skilled labour is not required, the gabion units can be quickly filled by hand or machine, and are an ideal material for fast and permanent slip remedial works on land, or in river or coastal situations. Current in-place costs when hand or machine filled are approximately $23.00 per cubic yard for galvanised gabion units and $28.00 per cubic yard for PVC coated gabion units. Walls cost from $3.00 and upwards per sq.ft of face area depending on height and thickness.



SKETCH 2

4.80 Gobimat

Gobimat is satisfactory for lightweight revetment work on land, in rivers and also coastal stabilisation work, particularly where access by pedestrians or vehicles is desirable, e.g. along a foreshore.

4.81

Prior to construction the bed should be graded closely to the final tolerances allowed because final levels will be determined by the underlying bed level. Accordingly, tree stumps, projecting rocks, and similar obstructions should be removed prior to construction and to ensure that the filter fabric is kept in close contact with the ground to prevent water from running unimpeded beneath the mat. It is good construction practice to bury the perimeter rows of blocks where severe erosion conditions are expected, and to ensure that the edge blocks on the individual filter fabric bases butt closely together at all joints. Large areas of Gobimat can be laid quickly with backhoe or crane, and in-place costs in Auckland are approximately $8.00 per sq.yd. In areas outside of Auckland, rail cartage costs add approximately $1.00 per sq.yd. per 50 miles.

4.90 Tie-Backs

Retaining walls either tied back to a deadman, or with ground anchors, are both tie-back walls. Fortunately the use of several strands of No.8 gauge wire has been superceded by high tensile strands and rods, but as rememdial contractors we still come across an occasional wall where the No.8 wire is popping off.

4.91

The principal of a tie-back wall is excellent, and very well known. However, in New Zealand there have been difficulties experienced in the use of ground anchors and their use in new permanent works is not generally recommended. Where an existing wall has to be strengthened and the anchors are relatively short, say 30 to 40 ft in length, then ground anchors can be an economical solution. Only experienced operators should be employed however, because cleaning out of the bores, installation of anchor tendons and grouting operations are all critical factors.

4.92

For new and permanent works the most satisfactory tie-back wall uses large diameter in-situ RC.piles, or R.C. deadmen. The tie-back tendons can be pulled through horizontally drilled-in ducts, and grouted after stressing operations have been completed. The load on the wall can be anchored and spread using galvanised steel plates, a steel channel section, or simply pre-cast R.C. blocks.

5.00 GEOLOGICAL CONSIDERATIONS

On most of our highways and railways, steep batters are often necessary, and certainly inevitable. It is very easy to criticize design of slopes and batters after a failure has occurred, however at this point we should express gratitude and respect for the engineers and other personnel responsible for the maintenance of our essential communication systems. Their personal time and considerable effort is often put in to clear dozens of slips in steep country, during bad weather, so that traffic flow is maintained.

5.01

Obviously we cannot revet or retain all the slopes and batters on our roads and railways, as in steep country the cost of even one retaining structure can be astronomical. Basically the design engineer must decide on the most suitable grade for a slope or batter, or the type of structure required to suit a particular problem. However, regardless of design considerations, most new cut slopes will slip or fritter away within a few years in an effort to regain their natural angle of repose. This process can be observed by looking closely at a number of recent, stable-looking batters in almost any area of New Zealand. The debris from the most recent dropout usually lies in the ditch alongside the road or track, and looking up one sees numerous scars where saucer dropouts, sheet slides, and tops of batters have dropped off by the forces of gravity and the natural elements. Conversely older batters have already reached a reasonable state of equilibrium,having a good hamper of natural vegetation, and a clean surface drain.

5.02

One can learn a great deal about the soil and stability of slopes merely by examining existing natural slopes and vegetation about us. We can benefit immensely by heeding what Mother Nature has already done and what She will do to a batter that is cut too steeply.

5.03

In very steep and mountainous areas of New Zealand, steep and sometimes vertical batters cannot be avoided. The design engineer's role is certainly not one to envy. His basic task is to ensure that slope failures which will inevitably occur will avoid injury or loss of life, and cause the least possible property damage. In such areas the design engineer should call for geological as well as soil mechanics advice at the earliest possible opportunity. In this way the design and construction engineers can take advantage of natural features and sometimes avoid very costly construction and remedial work. The cost of annual maintenance and possible remedial work can be substantially reduced and even eliminated by reducing a batter slope, providing a bench, or by constructing a simple toe wall at the time of construction. (see Sketch 2)

5.04

In all cases where major new or reconstruction works are proposed, the writer is firmly of the opinion that design based on geological information as well as engineering criteria could save thousands of dollars on construction costs alone.

6.00 CONTRACTUAL PROBLEMS ASSOCIATED WITH REMEDIAL WORKS:

In carrying out remedial works, the writer has seen numerous examples where new highways or improvements have cut into, and reactivated old, massive landslides. Similarly, deep cuts have been made through hillsides where the bedding was such that when the toe was cut, massive landslides have poured across or into the new cutting and continued to do so even after constructional works have long been completed. Some of these cuts and subsequent failures have caused thousands of dollars worth of additional earthworks, and remedial works where often, the wet debris has had to be removed with draglines whilst other crawler and rubber tyred equipment sit idle. Remedial materials and equipment has had to be manhandled or winched into position to doctor the situation. The cost to all concerned is enormous, and from a contractor's point of view disastrous because his plant cannot operate efficiently, and what should have been a profitable operation turns into a hefty loss. Men engaged on remedial work have to traverse and work in deep mud, often in the rain, they quickly become despondent, demand a high standard of temporary accommodation and higher wages than normal, and still will not endure the work for more than very short periods of time. Remedial contractors have to allow for these factors in their pricing of work, and sometimes are criticized for submitting what appear to be high rates. In designing remedial works, engineers should aim to employ a method which utilizes plant to best advantage and which reduces labour content to the minimum possible.

6.01

Another problem which causes embarrassment to a contractor fairly regularly, is that he is asked to take out an Insurance Policy covering "All Risks". In the case of landslides involving structures, Insurance Companies will not accept this risk which is likened to that of trying to insure a ship when it is known to be sinking. Providing the contractor has satisfactory experience and adequate Public Liability cover, the writer believes that the Principal should accept liability for all other risks, and we generally make this a condition of contract on all landslide corrective work.

7.00 CONCLUSION AND SUMMARY

From a contractors veiwpoint, it is clear that many slips and costly remedial works can be avoided if more care is taken when planning to cut into natural slopes. Wherever possible, preventative measures should be planned and incorporated into the works to avoid expensive corrective measures after a failure has occurrred. There is considerable merit in the simple precautionary measure of unloading the top of a slope and supporting the toe.

7.01

In so far as development of steep subdivisional properties is concerned too many inexperienced disciplines and laymen are allowed to interfere with the stability of our natural slopes, often without regard for neighbours, and this is a deplorable and shortsighted situation indeed.

5.16

7.02

In steeply contoured land where soil conditions are known to be potentially unstable, it is imperative that we place more reliance on experienced soils engineers to reduce the number of slips and future large claims on the Earthquake and War Damage Commission. To achieve this, the writer is firmly of the opinion that legislative precautionary measures as well as physical measures are necessary.

INCLINED PLANE SLOPE FAILURES IN THE AUCKLAND WAITEMATA SOILS

THREE CASES WITH DIFFERENT REMEDIAL MEASURES

G.R.W. EAST

1. INTRODUCTION

A large section of the Auckland Isthmus has an undulating topography consisting of the residual soils overlying the interbedded sandstone/siltstone of the Waitemata Group with little or no Pliestocene cover. Typical natural slopes would range up to 20° in angle and 25m in height.

The slopes are generally stable in their natural state but are susceptible to slipping if the topography is altered by relatively extensive but conventional earthworks such as the formation of motorways, school playing fields and subdivisions.

These slips are generally not the classic slip circle type but a composite 'block' failure sliding on an inclined plane. They are generally slope failures. The typical conditions and properties of these failures are given in this paper, and three case histories are outlined each with different remedial measures. In two of the cases the measures involved a local reduction in ground water level and in the third a rock toe was utilised. The first two were successful but the latter failed almost immediately on reconstruction.

These successful measures are generally applicable to all soil slope failures.

2. SOIL HORIZONS AND THE GEOMETRY OF THE SLIP PLANE

The process of weathering in the Waitemata Group results in a soil horizon generally following the topographic surface. The depth of weathering is typically 6 to 9 metres before the interface of the relatively unweathered interbedded sandstone/siltstone is encountered. With this cover of residual soil the inclination of the surface of sandstone/siltstone can range up to 20° in the undulating or valley controlled areas. The residual soils grade from the surface in the sequence of firm to stiff clays and clayey silts to sandy silts and silty sand at the interface of the sandstone/siltstone.

The silts and sands still retain some of the interbedded structure of the parent material and because the interface does not generally follow the bedding the interface soils are very hetrogeneous. For this reason the clay content and cementing of the interface soil is variable, ranging from high clay content and cementation to near cohesionless soils.

3. CONDITIONS OF FAILURE

The winter ground level is generally within 1m of the ground water surface. <u>It is considered that this high water level in conjunction with the near cohesionless silty sand and sandy silts immediately above the inclined sandstone/siltstone interface generates the stability problem</u>.

The formation of the slope removes a large proportion of the residual soils from the batter. This results in a reduction of normal stress (p) on the silts and sands on hence effectively reduces the shear strength. $S = (p-u) \tan \phi'$, $(C'=0)$

The construction also removes all or a large proportion, of the toe passive force that resists sliding.

4. VISUAL PROPERTIES OF THE SLIPS

(a) The timing of the major failure in the formed slopes is generally 18 months after construction and, as with most slips, in the wetter months of July and August.

(b) The rate is moderately slow with the initial movement taking place over a day or two and further progressive movement after periods of heavy rain.

(c) Typically the top of the slip has a slump portion between two vertical tension cracks. (grabens). The width of the slump depends on the period of time after the initial failure (see Selwyn College slip).

(d) The toe of the slip is dependent on the geometry of the sandstone interface and the cut batter. If the hard interface is exposed in the excavated batter an overlapping sheer is observed.

(e) Water is visible in the tension cracks and the water percolates from the toe in local areas. Some piping of the silts and sands can be observed in the eroded water channels beyond the toe.

5. INVESTIGATION AND SUBSURFACE PROPERTIES

5.1 A detailed survey is undertaken on a number of crossections and pegs are installed for movement checks.

5.2 Depending on conditions, rig bores, posthole bores and Dutch Cone Penetrometer are used in the investigation to determine the soil horizons and properties. The usual problems of placing heavy plant on an inclined slope are encountered. Because of its portability the light 2 Tonne penetrometer is found very useful in determining the inclination of the sandstone/siltstone.

5.3 Ground water levels are observed by perforated pipe standpipes placed in the bores with full realisation that more sophisticated piezometers could be used if time was available.

Typical winter ground water levels range from 0 to 1m from the ground surface even after the slope excavation exposes the sandstone/siltstone interface. Water levels higher than ground level have been observed in lower portions of some slopes. A 2 to 3 metre drop in water level can be expected in summer.

5.4 Low cost slip plane indicators are used in the bore and penetrometer holes. These consist of "3/4 inch" flexible alkathene pipe fed down the hole with a 10mmØ, 150mm long mandrel lowered to the base of the pipe with nylon cord. After further slip movement (50 - 100mm) the cord is raised, and the mandrel catches where the slip plane has distorted the alkathene. The relative difference between the length of cord withdrawn determines the depth of the slip plane.

When utilising these indicators on the failures discussed here they have determined the slip plane is on or immediately above the sandstone/siltstone interface within the silts and sands.

5.5 Conventional soil testing is carried out with emphasis on the silts and sands emcompassing the slip plane. Typical properties of the clayey silts and silty sands

$c' = 0 \quad \phi = 28^\circ - 32^\circ$ cone resistance $20 - 40 kg/cm^2$

$Cu = 25 - 35$ kPa

$\gamma = 17 Kn/m^3$, $W\% = 30\% - 40\%$

$PI = 20\%$ $LL = 45\%$ $PL = 25\%$ $< 2\mu = 20\%$ $< 60\mu = 80\%$

$$K\ lab = 10^{-4}\ mm/sec$$

The relatively low permeability of the silts and sands is indicative of the high ground water levels even when the soils are exposed in the batter.

6. PROBABLE MECHANISM OF FAILURE

Before any remedial measures can be attempted the likely mechanism of failure must be postulated.

In the type of failure considered the failure surface can be confined to a relatively narrow depth zone. The failures are considered "long term" and hence effective stress strength parameters (c', ϕ') are applicable on the sand/silt shear plane. Although not free draining the samd/silt layer is considered the aquifer of the slope and hence trasmits the hydrostatic or piezometric head from the surrounding catchment.

The probable mechanism of the slips is as follows :

(a) 1st summer: Excavation with the partial removal of the normal stress (p) on the sands and silts and the complete or partial removal of the toe soil pressure. Undetectable movement occurs with the development of tension cracks.

(b) 1st winter: High ground water levels and hence high pore pressures occur in the sand/silts. The sand/silt shear strength is effectively reduced but probably retains some internal negative pore pressures from the unloading. Some undetectable creep probably occurs.

(c) 2nd summer: The tension tracks open further with soil shrinkage accompanied by a drop in ground water level.

(d) 2nd winter: Major failure generated by high ground water level and the high water force of a water filled tension crack. Further movement occurs during periods of high rainfall.

7. REMEDIAL MEASURES

7.1 Any remedial measures mut alleviate the mechanism and detrimental properties of the failure. As with all slips the cause is likely to be more complex than postulated even after a very extensive investigation. For this reason any remedial measures undertaken should allow for the possibility that the correct cause or mechanism has not been ascertained and hence the most economical measures should be undertaken initially. There have been numerous occasions where very expensive reconstruction methods have not resulted in stability largely because the cause of the failure was not acted upon or detected.

7.2 As far as these inclined plane failures are concerned the remedial measures undertaken have been primarily a large reduction in the ground water level. Without going into details a reduction in pore pressure must increase the effective shear strength of the slip plane $S = (p-u) \times \tan \phi'r$. In conjunction with the reduction in water level it is essential that the tension cracks should be adequately drained to avoid any build up of high water forces, ($\frac{1}{2} \gamma w H c^2$), from surface run off. This reduction in water head in the slope and tension crack has a major advantage in that even if the conditions of the failure plane are incorrect the safety factor of the slope must increase.

7.3 Little can be gained from reducing the batter angle in this type of slip as any reduction in weight force is compensated by a reduction in normal stress (p) and hence a corresponding reduction in shear strength.

7.4 Loading the toe is generally not practical although toe 'shear keys' of high strength soils can be used in conjunction with drainage. If granular fill is used in the "key" an angle of friction greater than the failure plane must be guaranteed ($\phi' > 30^{o}$).

7.5 The increase in stability from these measures can be shown by an effective stress analysis. It should be noted that although the slope had a safety factor of unity on the initial failure the slip plane has decreased in strength with mobilisation such that a reinstated batter without remedial measures could have a safety factor considerably less than unity (see penetrometer tests of Massey School slip). The increased safety factor can only be analysed if the residual effective stress angle of friction (ϕ'r) of the mobilised slip plane is known. The ϕ'r values used to date have been obtained from the Tan ϕ'r/PI relationship given by Voigt (Ref. 2) or the Tan ϕ'r/$< 2\mu$ relationship given by Skempton (Ref. 1). A ϕ'_r of 25^{o} seems applicable for the soils considered.

8. METHODS UTILISED FOR THE REDUCTION OF WATER LEVEL IN THE SLOPE AND TENSION CRACK

8.1 Counterfort (Groyne) Rubble Drains as Used in Massey School Slip:

These are rock filled trench drains constructed on, and in the direction of, the slip. The drains intercept the major tension cracks and are taken 15 metre beyond the slip to drain undetectable cracks. On major slips they can only be practicably installed in summer.

In general they are 1 metre in width and spaced at 15 metre intervals. The spacing has no theoretical justification other than it has been found adequate in practice. The depth of the drains are governed by the plant available and the surface of the slip but 5 to 6 metres has been found practical. The drains are backfilled with free draining well graded granular fill (Max. size 0.3m) as the excavation proceeds. Ideally a filter coarse should be utilised but clean "run of pit" has been used for economic reason. A veneer of topsoil is placed on the surface of the rock for grassing.

In all cases to date the drains have been sufficiently deep to intercept the sandy silt and silty sand immediately above the siltstone/sandstone. The cost of Counterfort drainage is in the order of \$4.75 per cubic metre in place (1972 price using M.O.W. plant rates).

If the silt/sand aquifer is beyond the practical depth of the available plant some means of ensuring that the piezometric level is at the invert level of the drains may be required. Although not utilised to date vertical drains bored in the silt/sand at the base of the counterfort have been considered.

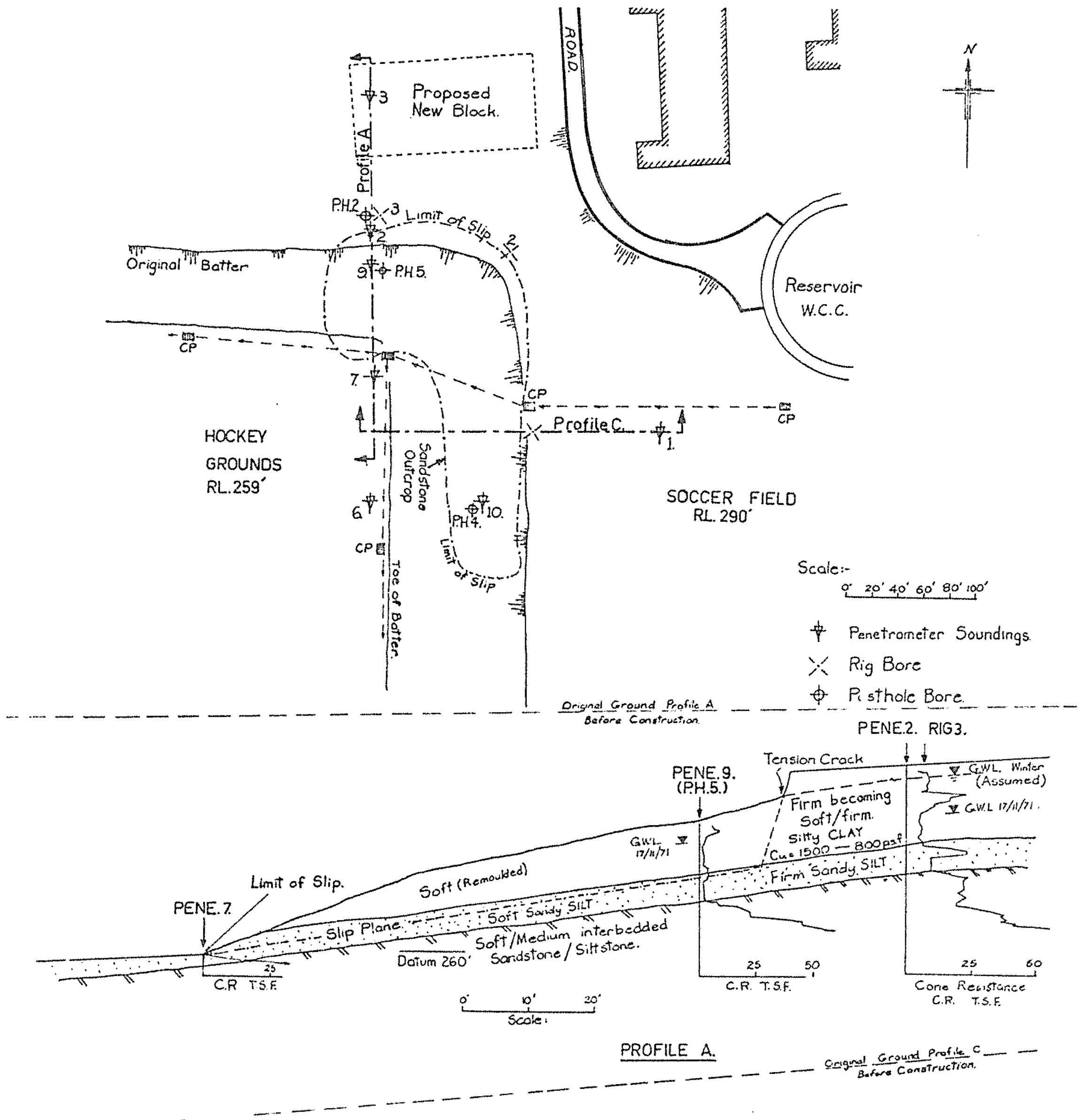
8.2 Bored Horizontal Drains as Used in the Henderson Substation Slip:

These are generally slotted P.V.C. pipe, 50mm in diameter placed in a near horizontal prebored hole at the toe of the slip or slope. The optimum length of the drains is in the order of 20 to 30 metres. They can be placed in rows in the order of 3 metre centres, or 'fanned' from a central point(s).

With the type of slip discussed it is considered essential that the silt/sand acquifer be intercepted by the drains for the major portion of its length. This criteria is not difficult as the drill bit readily follows the interface of the sandstone/siltstone and hence is in close contact with the silts and sands.

Unfortunately there is no guarantee that a badly slumped tension crack can be drained permanently by this method even though the drains extend beyond the crack. This can be overcome by installing a row of drains higher up the slope such that the tension crack is intercepted ... the firmer intact clays.

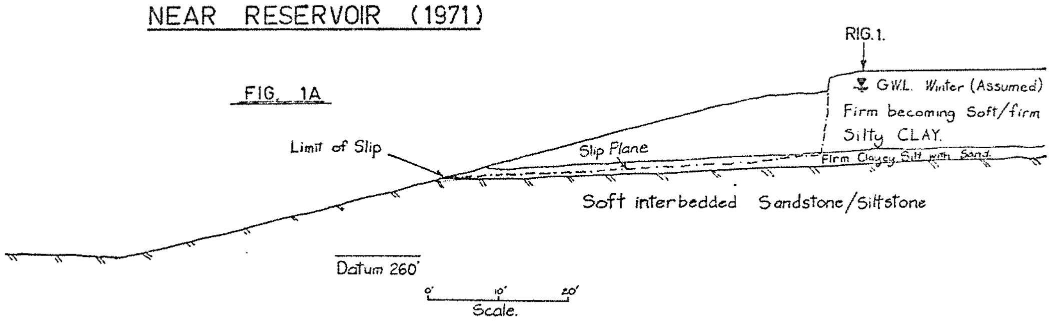
The current cost of installation is in the order of \$4.50 per metre in place.



MASSEY HIGH SCHOOL SLIP

NEAR RESERVOIR (1971)

FIG. 1A



8.3 Comparison of the two methods: The overall costs of the two methods are in the same order and hence some consideration must be given to the appropriate system for the slip in question. Other than the obvious differences in speed, accessibility and construction aesthetics of the two systems, the principle requirements of drainage must be guaranteed.

9. STABILITY ANALYSIS

Two methods of analysis have been utilised for this type of failure. Both methods are a mixed cu/c',Ø' analysis using the effective stress properties (C', Ø') for the silt/sand sliding plane and the undrained shear strength (Cu) in the upper clay soils. A vertical tension crack of depth $\frac{2Cu}{\gamma F}$ is included which is water filled if thought applicable. (Ref. 3)

9.1 Wedge Method : An approximate method using the statics of a sliding "block" and Rankine pressures resolved to the inclination of the slope. This method has the advantage in the analyses of slopes prior to excavation in that the critical length of sliding plane can be determined using quick analytical means. This method of analysis is shown in the appendix.

9.2 Janbu Method : (Ref.4) The more rigorous, conventional method of analysing non-circular slips using slice forces resolved to the horizontal. A paper by 'L.R. Pimenta' showing how the Janbu method (and circular methods) can be analysed using the small programmable calculators currently on the market is given in the appendix. It is considered essential that the force of a water filled tension crack is used in the analysis. (Ref. 5).

10. CASE HISTORIES

Three inclined plane failure slips are outline, each with different remedial measures. In two of the slips the measures have been successful but the third failed almost immediately after reconstruction.

1. Massey High School Reservoir Slip - Counterfort Drains - Successful.
2. Henderson Substation Slip - Bored Horizontal Drains - Successful.
3. Selwyn College Slip - Rock Toe without Drainage - Unsuccessful.

It should be noted that no investigations had been carried out on these slopes prior to the instability.

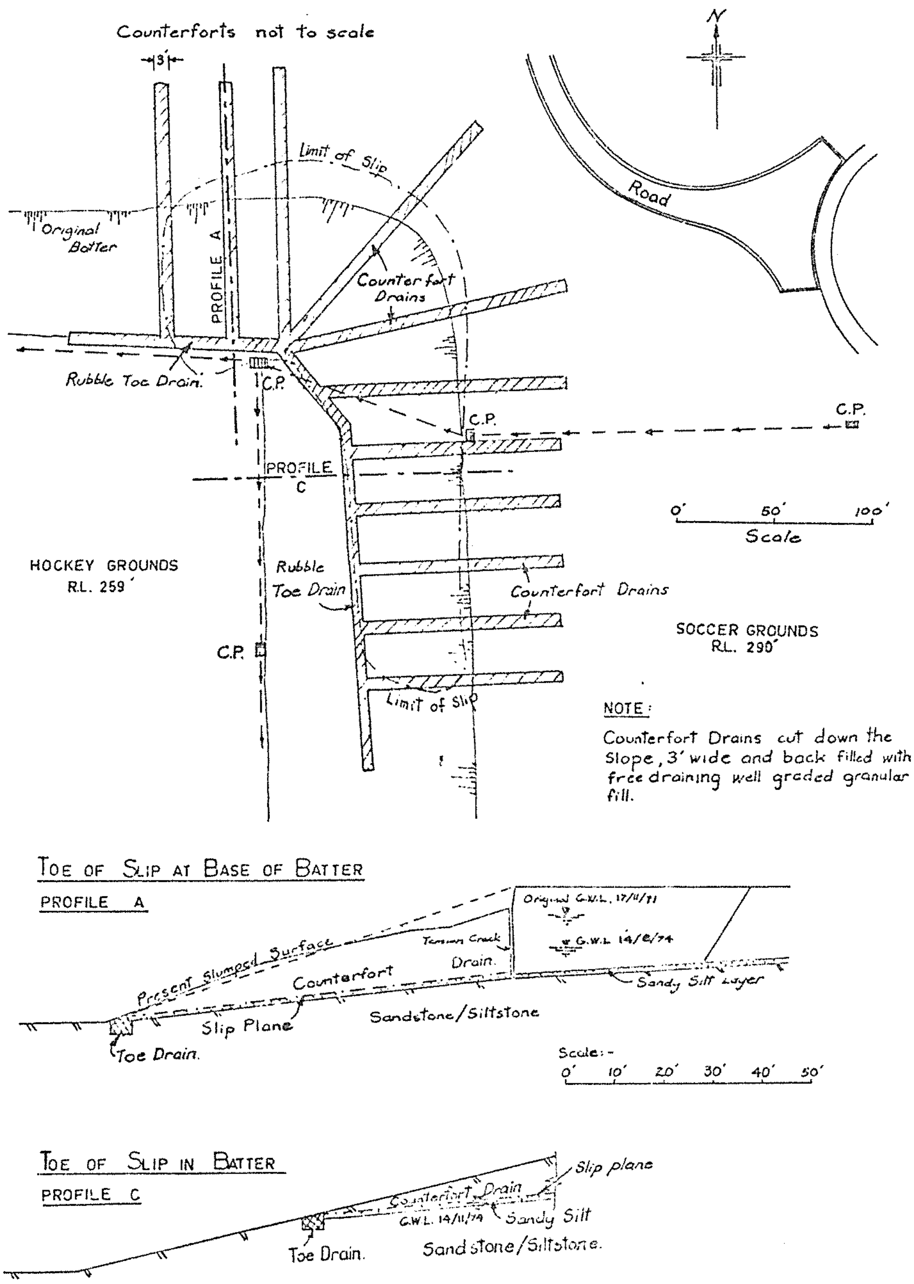
11. MASSEY HIGH SCHOOL RESERVOIR SLIP

(figures 1a and 1b)

11.1 History: The batters were excavated in 1970 to form the school playing fields. Failure occurred in the winter of 1971 at a position that risked an adjacent proposed teaching block site and to a lesser extent, a W.C.C. reservoir. Remedial measures were considered of moderate priority and counterfort drains were constructed in the summer of 1971/72.

The original topography was a 6:1 slope in the general direction of the slip and 14 metre of overburden was removed from the toe level to form a 9 metre high, 3:1 slope. Sandstone/siltstone was exposed at, or above, the toe of the batter.

The slip topography consisted of a badly slumped double tension crack and slip surface with an overlapping soil toe especially where the sandstone/siltstone was exposed in the batter. The subsoil link drains between the upper and lower playing field cess-pits were of the field tile type. After



MASSEY HIGH SCHOOL SLIP NEAR RESERVOIR, REMEDIAL MEASURES.

FIG. 1B

movement these were exposed to the tension crack which must have been completely filled during heavy rain.

An investigation was carried out in November 1971 (fig. 1a). This indicated that the inclination of the sandstone/siltstone ranged up to 10^{o} in the direction of the slip and a thin (1m) layer of sandy silt was present on the interface in most bores. This soil is overlain by up to 4m of clays. The slip plane was determined to be above the sandstone/siltstone interface by indicators and geometry. At the time of investigation the ground water level was between 1 and 2.5 metre below the surface although a surface winter level was assumed.

11.2 Remedial Measures and Results: (See fig. 1b) Counterfort drains were constructed to the depth of the sandstone/siltstone and at a 10 metre spacing. These were intercepted at the toe by a shallow rubble filled toe drain fed into the existing cesspits. All field tile link drains between the fields were replaced by continuous P.V.C. pipe. The minimum of earthworks was undertaken to reform the batters.

No further movement has taken place in the ensuing 2½ years. A recent check on the winter ground water level midway between the drains (August 1974) indicated a drop in level to between zero and 1 metre above the sandstone/siltstone.

Ground Water Levels

Bore	November 71	Winter 71	Winter 74	Drain Depth
2/2A	2.0m	0m (assumed)	3+ m	5 m
3/3A	2.5m	"	4.5 m	5 m
4/4A		"	2 m	2 m
5/5A	1.0m	"	3.5 m	4 m
Rainfall to August		1000mm	700mm	

11.3 Stability Analysis : An analysis was carried out on Profile A using the approximate wedge method.

Original Slope: (winter) ($\phi' = 30^{o}$)		Drained Slip : ($\phi'r = 25^{o}$ assumed)	
Without water filled tension crack	With tension Crack water filled		
1.4	0.8	2.6	Safety factor

12. HENDERSON SUBSTATION SLIP

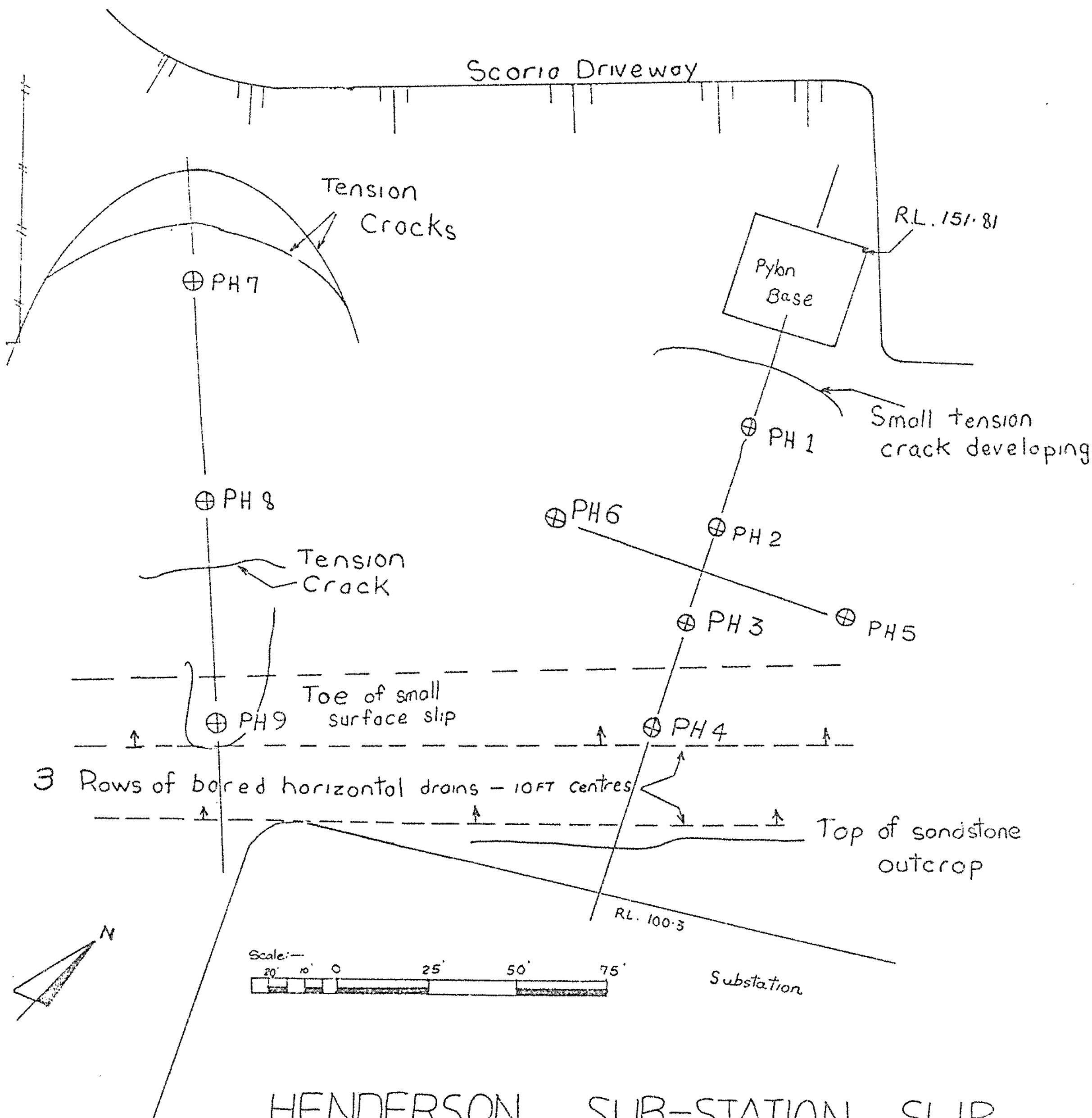
(see figures 2a & 2b)

12.1 History: The batter was excavated in 1966 to form the substation building platform. Failure occured in the winter of 1972 in the form of two separate but adjacent slips. One slip was in an advanced stage and the second, which was immediately below a high tension power pylon, had just formed a tension crack. Remedial measures had a high priority as the pylon was the main feeder to the Marsden Point Power Station. A system or bored horizontal drains were drilled in July 1972.

The original topography was a 6:1 slope in the direction of the slip and 11 metres of overburden was removed from the toe level to form a 3:1 batter some 15 metres in height. The sandstone/siltstone was exposed at or above the toe of the slope.

The slip topography consisted of several tension cracks with little slumping and a relatively intact but saturated surface. Some slight heave of extremely soft soil and free water was observed in the toe of the more advanced slip but no toe was visible in the pylon slip.

The limited investigation indicated that the inclination of the sandstone/siltstone was in the order



HENDERSON SUB-STATION SLIP

SITE PLAN (7/72)

FIG 2A

NOTE : Pylon contains main feeder to Marsden Point of 10^{o} in the direction of the slip and soft/firm sandy silts and silty sands were present on the interface at a depth of 3 to 4 metres. Bore depth soundings and geometry indicated that the slip plane was in the silts and sands. The ground water level was observed to be at ground level in the pylon slip and 0.7 metres in the more advanced slip.

12.2 Remedial Measures and Results: (see Figure 2b) Three rows of bored borizontal drains with 50mm Ø slotted P.V.C. pipe were installed by an Auckland based company. A 3 metre spacing was utilised with a row of spacing of approximately 6 metres. 55 drains of nominal length 30 metres were used. Because of the critical position of the slip speed was essential. The limited, but adequate, investigation and drain installation was completed in July within 10 days. No earthworks were undertaken.

No further movement has taken place within the two years of drain installation. Within 5 days of drain installation the ground water level had dropped between 1 and 2.5 metres although some fluctuation was observed during periods of heavy rain (+.5m). A recent check on the winter ground water levels (August 74) indicated that the lower drained level still exists.

Bore No.	July 1972	August 1972 (after drainage)	August 1974
PH 1	.5m	1.6m	2m
PH 2	.3m	2.2m	2.7m
PH 3	surface	1 m	.7m
PH 8	1 m	2 m	2.6m
Rainfall to August		800 mm	700 mm

The flow characteristics of the horizontal drains may be of interest. Recent measurements on a fine winter day indicate the flow rate of individual drains varies from 1.00 litre/min to 0.02 litre/min (13 gal/hr to .25 gal/hr) with only 1 drain not passing water. The total flow from all drains was summed to 9.3 litre/min (120 gal/hr).

Flow rates in litre/min (gal/hr) Lower row 3.0 (40) Middle row 2.6 (35), Upper row 3.7 (45) Total - 9.3 (120).

These relatively small flows seem sufficient to maintain the low ground water level.

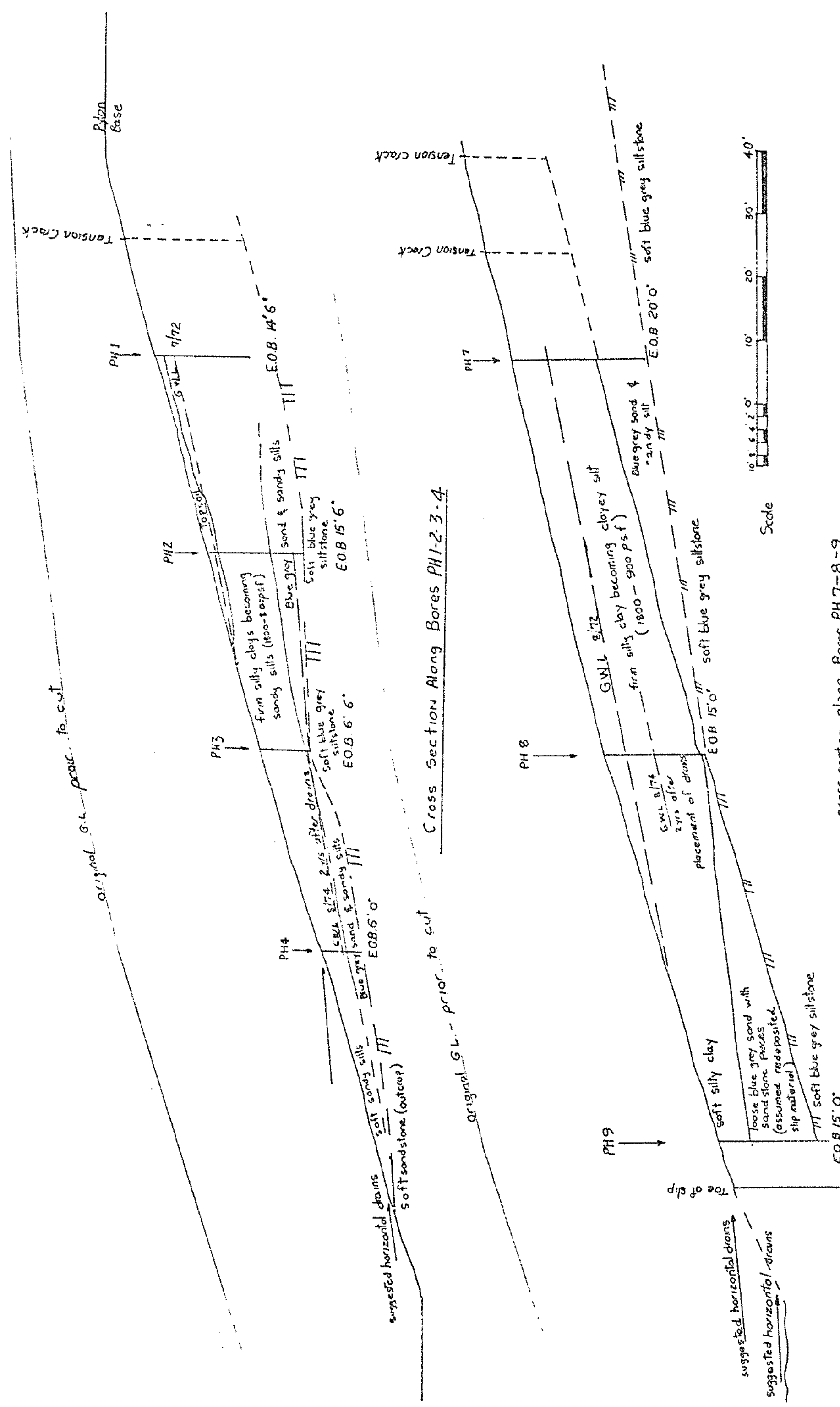
12.3 Stability Analysis (Approximate method)

	Original slip (winter $\phi' = 30^{o}$)		Drained slip ($\phi'r = 25^{o}$ assumed)
Conditions	without water filled tension crack	with water filled tension crack	
Safety Factors	1.5	1.1	2.9

13. SELWYN COLLEGE SLIP
(see Fig. 3)

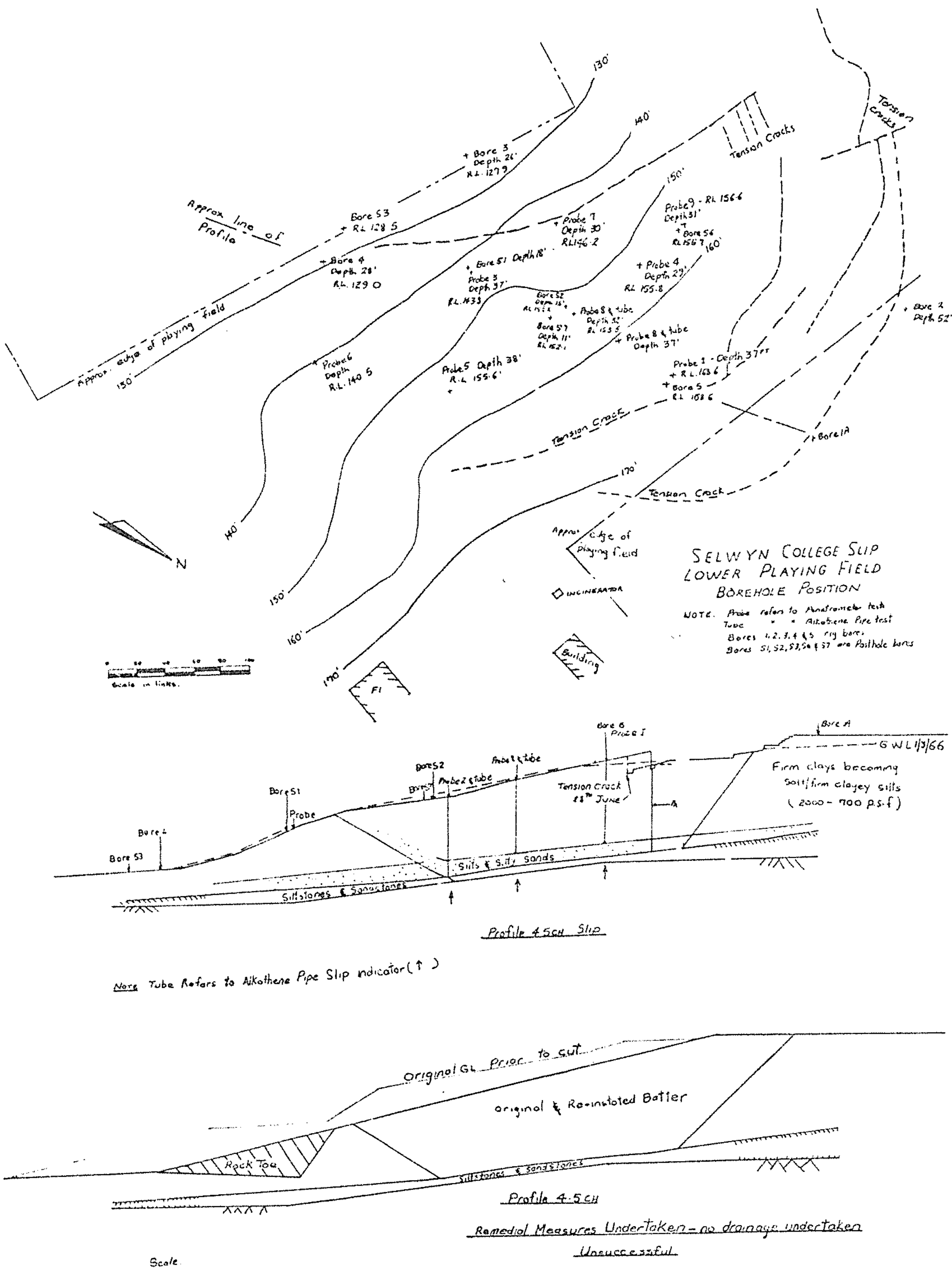
13.1 History : The slope was excavated in 1965 to form additional playing fields. The existing high level playing field had been formed from cut in 1955. The failure occurred in the winter of 1965 and movement continued through the summer of 65/66, with the loss of a large proportion of the upper rugby field. The priority was considered moderate and a buttress rock toe without drainage was constructed in the summer of 66/67.

The original topography was valley controlled with a 1:5 slope in the direction of the slip. 5 metre of overburden was removed from the toe level to form a 3:1 batter some 13 metres in height.



HENDERSON SUB-STATION SLIP (7/72)

FIG. 2B



SELWYN COLLEGE SLIP (1966)

FIG 3

Sandstone/siltstone was not exposed in the batter. A suspicious local gradient change can be observed on the pre-construction contours midway up the slope.

The slip topography included a massive tension crack slump some 15 metres in width and 1.5 metres in depth with two distinct deep tension crack boundaries. The surface of the slip was relatively intact and the toe had the unusual condition of a large bulge some 15 metres above the toe of the batter. This toe consisted of very soft and disturbed silt soils with some free water and piped silts and sands flowing in erosion channels. Considerable movement (3m) took place in the summer months during periods of rain. Subsoil field tile drains were exposed in the tension cracks.

An investigation was carried out in February 1966 (Fig. 3) which indicated that the apparent inclination of the sandstone/siltstone was in the order of 10^{O} and at a depth of 9 metres. On the interface of the sandstone/siltstone a 2m layer of silty sands and soft silts was present. The overlying soils were firm clays and clayey silts with numerous shear planes. These were observed in bores even outside the slip zone.

From the shape of the toe this slip had the appearance of having a circular slip plane but the slip plane indicators proved that failure was occuring along the interface of sandstone/siltstone. This was also justified by borehole depth probes and survey pegs.

The ground water level during dry periods was immediately below the surface but even after short periods of rain water actually flowed out of the boreholes on the slip. A stand pipe near to the toe of the slip indicated a piezometric level in excess of 1 metre above the ground.

The geometry and conditions of this failure are such that it is considered highly likely that it is the remnant of a much larger slip that existed prior to any construction on the site (pre 1955).

12.2 Remedial Measures and Results: Although the investigation indicated that a lowering of the water table was required for which a 'wishbone' drainage system was recommended, the on-site decision was to use a rubble rock toe without drainage. Unfortunately the rock was placed immediately in front of the slip toe and did not replace the slip toe soils. Earthworks were undertaken to reinstate the batter.

The measures were unsuccessful. The slip moved again almost immediately after the completion of earthworks and formed a similar geometry to that investigated.

12.3 Stability Analysis:

	Original Slope		Original Slope if Considered a Remnant of a larger slip
Conditions	(slip plane $\phi' = 30^{O}$, G.W.L. 1 m above G.L., Tension crack $Hc = \frac{2c}{\gamma_F} = 5m$, Cu of remaining slope 50 kPa)		(slip plane $\phi'r = 25^{O}$ assumed, G.W.L. + 1 m above GL Hc = 5m, Cu toe = 5 kPa).
Safety SF Approx method		1.5	0.95
Factors SF Janbu method		1.55	1.0

	Reinstated slip with Drainage		Reinstated slip with correctly placed Rock toe and No drainage
Conditions	($\phi'r = 25^{O}$, G.W.L. = 4.5m) (drained)		($\phi'r = 25^{O}$, ϕ' rock = 30^{O}, G.W.L. surface, Hc = 5m)
Safety SF (approx)		1.65	-
Factors SF(Janbu)		1.75	1.25

These factors indicate that it was highly likely that the slip was a remnant of a larger slip. They also compare the relative usefulness of a rock toe over an adequately drained slope.

Note : As the tension crack depth $\frac{2c}{\gamma F}$ was less than the depth to the slip plane Rankine active forces were included in the approximate analysis.

14. CONCLUSIONS

(1) Where major earthworks are to be undertaken in residual soils similar to the Waitemata Group, the site investigation should determine the contours of the relatively unweathered rock and the properties of the soil lying immediately above the interface.

(2) If the excavation of batters over an inclined interface of unweathered rock cannot be avoided and "block" failure is suspect, a local ground water reduction is likely to be the most satisfactory and economical remedial measure.

(3) Under critical conditions failure can occur within the interface inclinations of less than 10^{o}.

(4) Both counterfort and bored horizontal drains have been successful in alleviating stability problems providing due attention is given to tension crack drainage. Any subsoil drainage through batters should be carried out using continuous pipes.

(5) The analyses and remedial measures utilised in the slips discussed are in general applicable to any layered soil system with a hard interface. Within cohesive soils the reduction in piezometric head and the corresponding increase in safety factor would required some period of time.

(6) The investigations of stability should include the testing of the residual or ultimate angle of friction of the failure soil ($\phi'r$). If past movement is suspect this parameter should be used in the analysis.

ACKNOWLEDGEMENT:

The author wishes to thank the Commissioner of Works for permission to publish this paper.

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APPENDIX

EXAMPLE OF THE SAFETY FACTOR ANALYSES

APPROXIMATE WEDGE METHOD

The method employs a block sliding on the inclined sand layer. A Rankine passive wedge at the toe acts against a slide and a water filled tension crack at the top of the block increases disturbing force. All forces are resolved parallel to the slope of the inclined sand layer. (See figure A1).

Safey Factor $F = \dfrac{\sum \text{Restraining Forces}}{\sum \text{Disturbing Forces}}$

Restraining Forces

(1) Sliding Block Shear Resistand Force in the sand layer resolve down the slope

$= L\,(p-u)\ \text{Tan}\ \phi'$

$= (W\ \text{Cos}\,\alpha - U)\ \text{Tan}\ \phi'$

$= \left(W - \dfrac{U}{\text{Cos}\,\alpha}\right)\ \text{Tan}\ \phi'\ \text{Cos}\,\alpha$

(2) Passive Wedge Shear Force Resolved parallel to slope

$= \dfrac{2CuHp}{F^*}\ \text{Cos}\ \alpha$

L = Length of sliding surface

p = average normal soil stress on sliding plane

u = average pore water pressure normal to plane = Depth x Density of water above plane. γ_ω

W = Weight of soil in block.

U = Weight of water in block.

ϕ' = Angle of friction of thin sand layer = 30^o

α = Angle of inclination of slope.

Hp = Height of passive wedge.

$\dfrac{Cu}{F}$ = Mobilised undrained cohesion of passive wedge.

F* = Trial Safety Factors.

Disturbing Forces

(1) Weight force of sliding block resolved down slope

$= W\ \text{Sin}\ \alpha$

(2) Weight force of passive wedge resolved down slope

$= -\ \tfrac{1}{2}\ \gamma_s\ Hp^2\ \text{Cos}\ \alpha$

(3) Weight force of water filled tension crack resolved (where applicable).

$= \tfrac{1}{2}\ \gamma_w\ H_c^{\,2}\ \text{Cos}\ \alpha$

W = Weight of soil in "block"

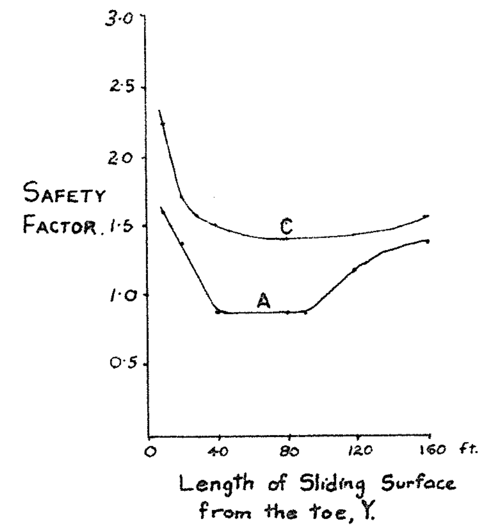
α = Angle of inclination of slope.

γ_s = Density of Soil

Hp = Height of passive wedge

γ_w = Density of Water

H_c = Height of tension crack $= \dfrac{2Cu}{\gamma F^*}$



A Proposed Cut with no remedial measures.

C Proposed Cut with Counterfort Drains.

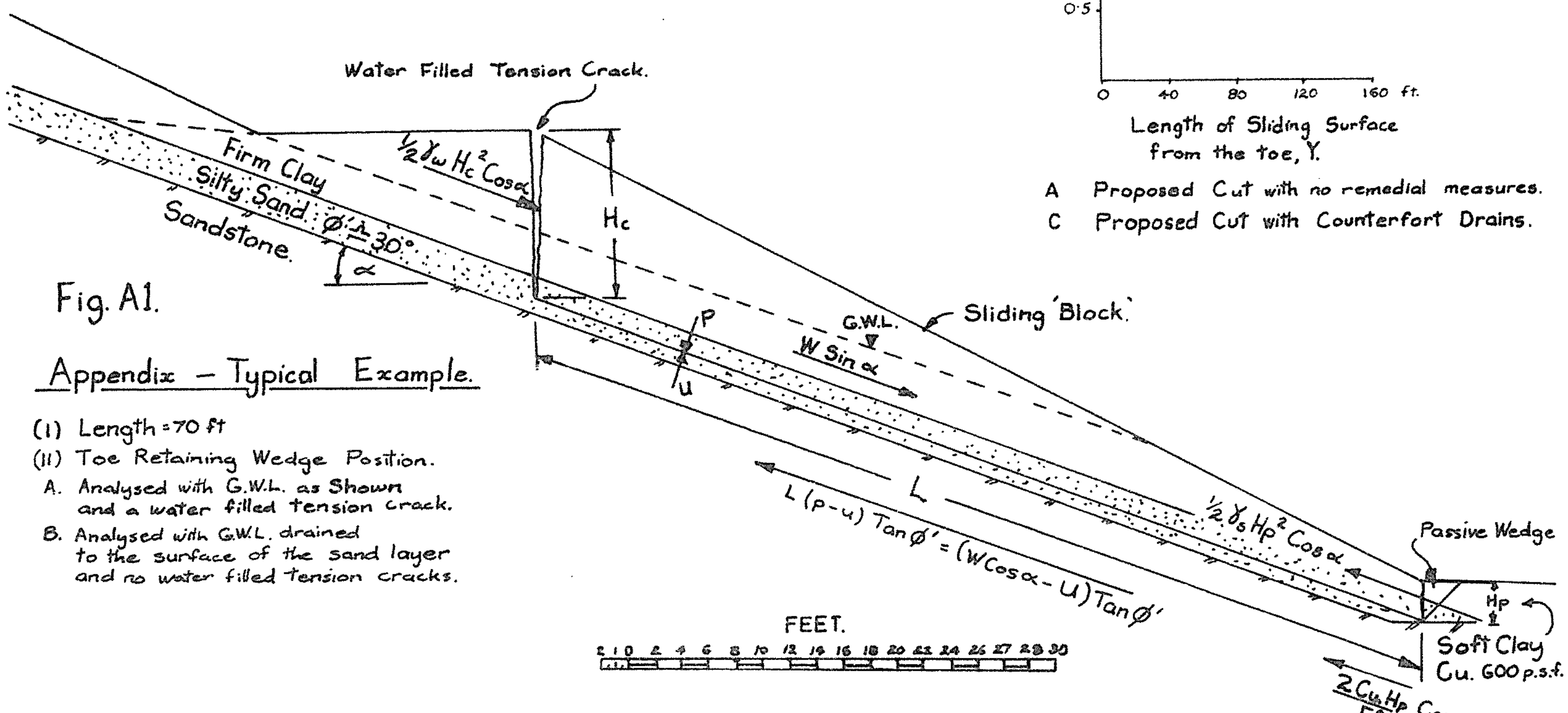


Fig. A1.

Appendix – Typical Example.

(I) Length = 70 ft

(II) Toe Retaining Wedge Position.

A. Analysed with G.W.L. as Shown and a water filled tension crack.

B. Analysed with G.W.L. drained to the surface of the sand layer and no water filled tension cracks.

Safety Factor $F = \frac{\Sigma \text{ Restraining Force}}{\Sigma \text{ Disturbing Force}}$

$$F = \frac{(W - \frac{U}{\cos\alpha}) \tan \phi' \cos\alpha + \frac{2CuHp \cos\alpha}{F^*}}{W \sin\alpha - \frac{1}{2} \gamma s\, Hp^2 \cos\alpha + \frac{1}{2} \gamma_w H_c^2 \cos}$$

$$F = \frac{(W - \frac{U}{\cos\alpha}) \tan \phi' + \frac{2cuH_c}{F^*}}{W \tan\alpha - \frac{1}{2} \gamma\, sHp^2 + \frac{1}{2}\gamma\, wH_c^2}$$

Note: Trial values of F^* are made until $F \triangleq F^*$

Example: L = 70' See Figure A1

(A) Slope without drainage

Restraining Forces

(i) $(W - \frac{U}{\cos}) \tan \phi' \cos\alpha$
$= 20.2$ kip

(ii) $2 \frac{cuHp\cos\alpha}{F^*}$
$= 2.3$ kip

L = 70'
W = 61.7 kip
U = 24.1 kip
$\phi' = 30^\circ$
$\alpha = 19^\circ$
Cu = 600 psf (soft clay toe)
F* = 1.0 (Trial)
Hp = 2 ft

Disturbing Forces

(i) $W \sin\alpha$
$= 20.0$ kip

(ii) $-\frac{1}{2}\gamma s\, Hp^2 \cos\alpha$
$= -0.2$ kip

(iii) Water Force
$\frac{1}{2}\ w\ H_c^2 \cos\alpha$
$= 4.2$ kip

γs = 110 p.s.f.
Hp = 2 ft.

γw = 62.4 p.c.f.
Hc = 12 ft.

Safety Factor

$F = \frac{\Sigma \text{ Restraining Forces}}{\Sigma \text{ Disturbing Forces}} = \frac{20.2 + 2.3}{20.0 - 0.2 + 4.2} = \frac{22.5}{24.0}$

$F = 0.94$

(B) Slope with drainage such that the ground water level is at the top of the sand layer.

Forces are the same as (A) except that :

(a) the restraining force of the "block" shear resistance is increased owing to the lowering of the G.W.L. and

(b) the disturbing force is decreased because water could not be contained in a tension crack in an adequately drained slope and hence Water Force $\frac{1}{2}\gamma_w H_c^2 \cos\alpha = 0$

Restraining Forces

$L = 70$ ft

$W = 61.7$ kip

$U = 10.7$ kip

(i) $\left(W - \frac{U}{\cos\alpha}\right) \tan\phi' \cos$

$= 27.5$ kip

(ii) $\frac{2c}{F^*} H_p \cos\alpha = 1.5$

$F^* = 1.5$ (Trial)

Disturbing Forces

(i) 20.0 kip

(ii) - 0.2 kip

(iii) 0.0 kip

Safety Factor

$$F = \frac{\Sigma \text{ Restraining Force}}{\Sigma \text{ Disturbing Force}} = \frac{27.5 + 1.5}{20.0 - 0.2} = \frac{29.0}{19.8}$$

$$F = 1.47$$

(C) Notes

If the tension crack depth $H_c = \frac{2c}{\gamma F^*}$ is less than the depth to the slip plane, Ha Rankine active forces will have to be included in the analysis.

Weight Force of Active wedge resolved down the slope

$= \frac{1}{2}\gamma_s (H_a^2 - H_c^2) \cos\alpha$

Active wedge Shear Force resolved

$= \frac{2C_a}{F^*} (H_a - H_c) \cos\alpha$

$$F = \frac{\left(W - \frac{U}{\cos\alpha}\right) \tan\phi' + \frac{2c_p H_p}{F^*} + \frac{2c_a (H_a - H_c)}{F^*}}{W \tan\alpha - \frac{1}{2}\gamma_s H_p^2 + \frac{1}{2}\gamma_s (H_a^2 - H_c^2) + \frac{1}{2}\gamma_w H_c^2}$$

ASSESSMENT OF SLOPE STABILITY AT PORO-O-TARAO TUNNEL SOUTH PORTAL

I.M. PARTON

1. INTRODUCTION

Poro-o-tarao tunnel is located on the North Island Main Trunk Railway 32 km south of Te Kuiti. The tunnel penetrates a ridge which forms the watershed between the headwaters of the Mokau and Wanganui Rivers.

The tunnel, which was completed in 1891, is constructed through grey cyclic mudstone of the Mahoenui Formation. The railway south of Poro-o-tarao tunnel is situated on the west side of the valley occupied by the Ohinemoa Stream. The valley is formed mostly in mudstone and thin interbedded sandstone of the Mahoenui Formation, while sandstone occurs more abundantly in beds up to 4m thick in the upper slopes.

Since 1910 there has been a continuing need for maintenance of the tunnel lining. In parts, especially near the south portal, this has resulted in complete relining of sections of the tunnel. Because insufficient clearance exists within the tunnel, due partly to deformation of the tunnel lining and partly to an increased moving structure gauge, rather than attempt to reline the whole tunnel it has been decided to discard the existing tunnel and build a new tunnel.

Investigations outside the present portals have revealed the mudstones and siltstones to be disturbed to a considerable depth by rotational slumps, planar slides and earth flows. Thus, when the location of the new portals was being fixed, one of the prime concerns was to establish them in areas where damage to the track from earth movement was less likely.

The northern portal is able to be established in a ridge immediately adjacent to the existing line where it is considered that rotational slides and slumps have not penetrated to tunnel grade.

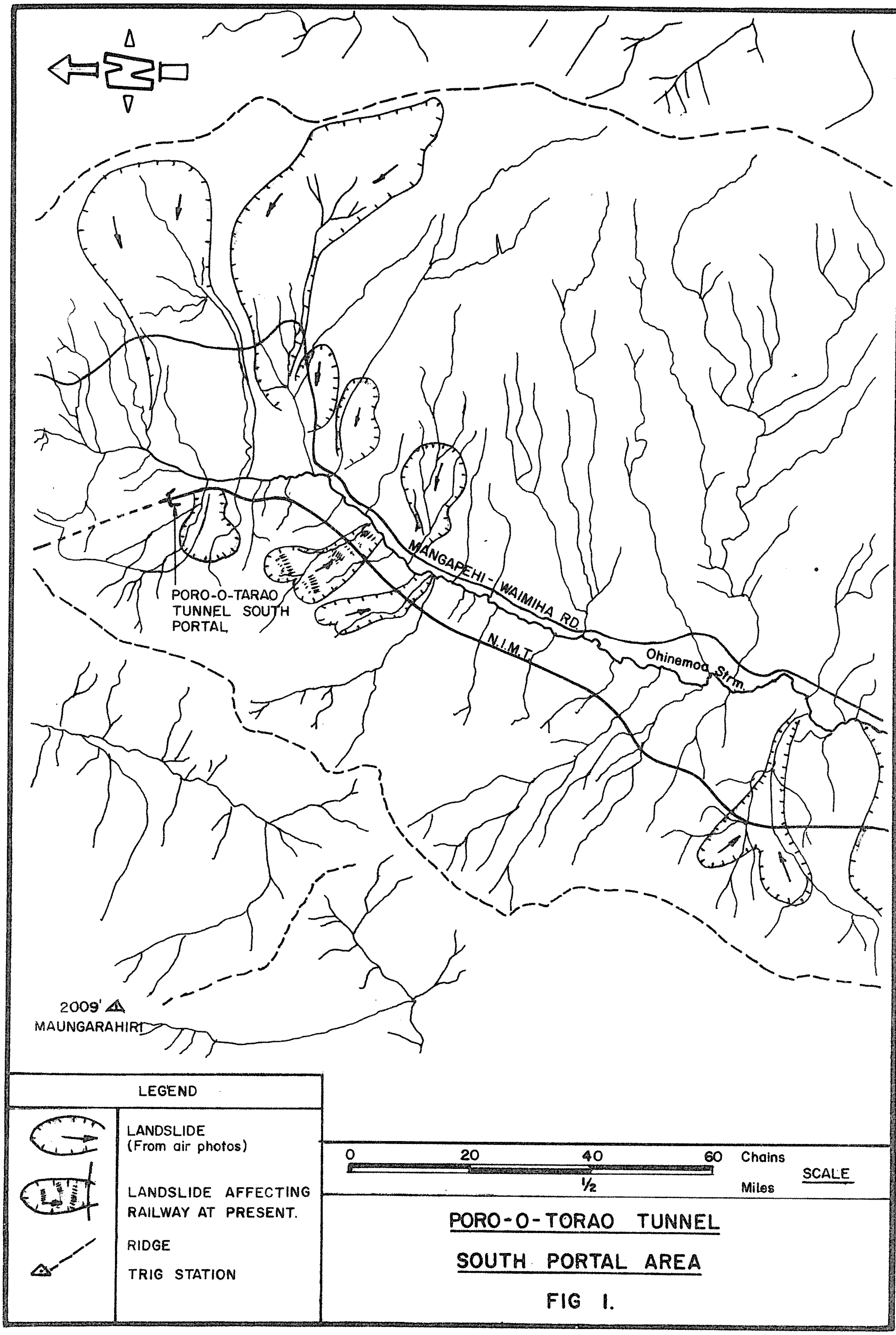
The southern portal location has been fixed only after considerable investigation along several lines. The whole southern portal area is one characteristic of rotational slumping and earth flows. Hillocky and hummocky ground, scarps of rotational slumps and exposures of large blocks of mudstone are visible. The upper surfaces of the slumps are often deeply cracked with dendritic drainage meeting at a focus. Aerial photos show the series of slumps on the hillside above the southern portal to cover an area of about 200 acres.
the paper).

A history of continued movement over the last 40 years has been obtained through discussion with local residents. A plan of the south portal area is presented in Figure 1. Landslides inferred from aerial photographs are detailed.

2. INVESTIGATIONS

2.1 Drillholes

Investigations for a replacement tunnel have been in progress for some years. However, only recently has a route been finalised. The investigations have covered an area outside both existing



PORO-O-TORAO TUNNEL

SOUTH PORTAL AREA

FIG I.

portals. Interpretation of the logged core has led to speculation as to the history of land movements in the area. A large number of drill holes have been put down, many exceeding 40m in depth.

Because the mudstones are so hard, complete core recovery is difficult. Thin lenses of softer or weathered material sandwiched between hard blocks of mudstone tend to be washed out. Conversely, hard fragments of mudstone floating in the surface colluvium tend to become lodged in the shoe of the core barrel and be pushed ahead through the colluvium giving poor core recovery.

2.2 North Portal Investigation Drive

An investigation drive approximately 150m long was excavated at the northern site 20m west of the proposed tunnel line. The drive revealed hard dry cyclic mudstones and siltstones. Two thin zones of crushed and sheared mudstone were encountered along the test drive length, which are thought to be indicative of slumping or movement of large blocks.

2.3 Exploratory Shafts

In January 1974 a 1m diameter exploratory shaft was sunk immediately outside the proposed southern portal position. The purpose of the exploratory shaft was, firstly, to provide visual examination of the colluvium known to overlie hard mudstone and, secondly, to aid in interpretation of the recovered small diameter drill hole core.

The excavated colluvium (slide debris) was found to be stiff dark grey silty clay in which was set hard and soft particles of mudstone and siltstone. Occasional large slabs of hard sandstone ("floaters") were encountered which could not be drilled by the Calweld bucket and had to be removed manually.

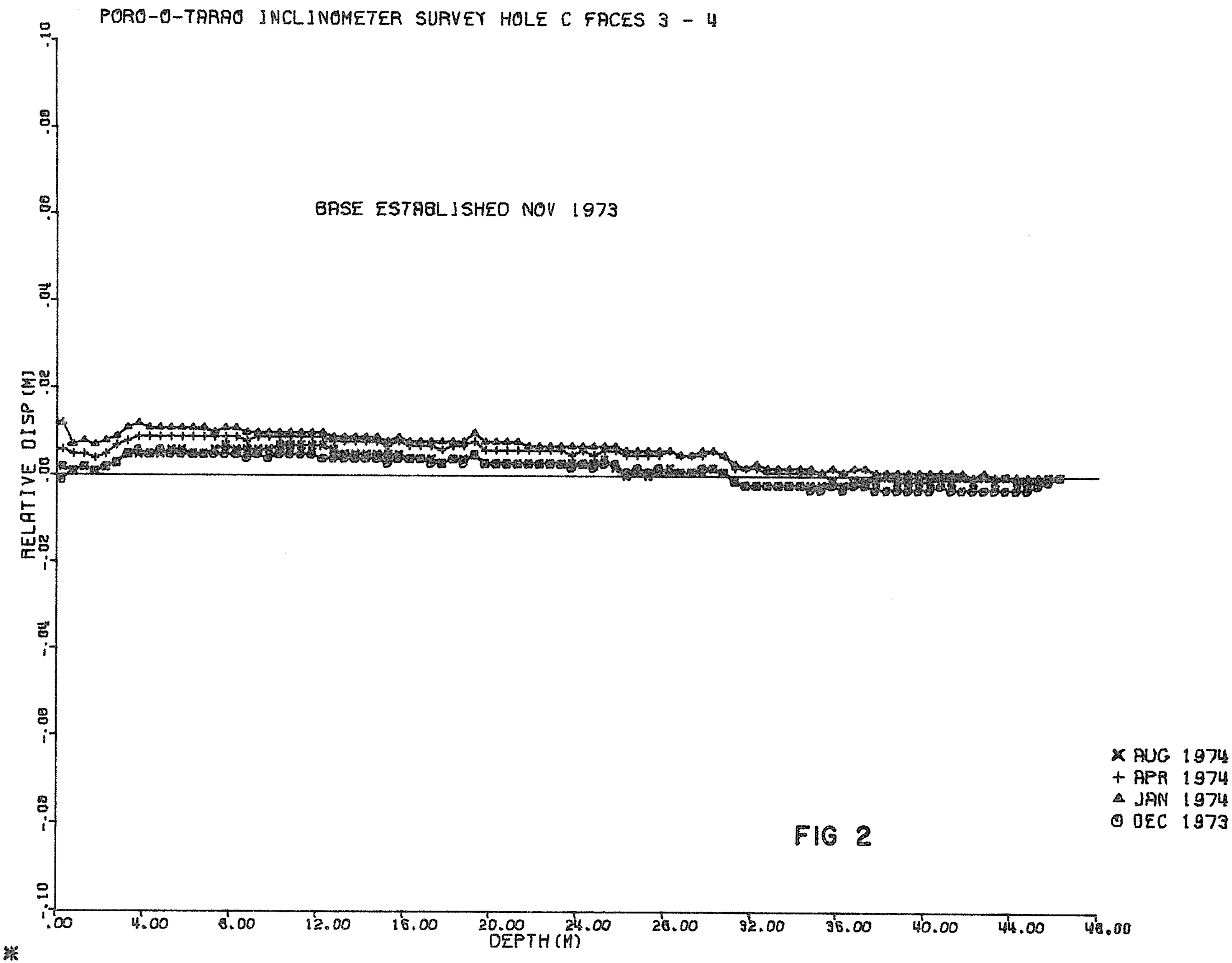
At 11.5m depth very hard dark grey mudstone was encountered. Visual examination of the shaft walls revealed a thin seam of green tinted highly plastic clay overlying the mudstone surface. Considerable water inflow was encountered on this interface although the colluvium above was relatively dry. (A colour slide will be screened showing the clay seam insitu).

After drilling for another metre the hole was cased off. In all probability the mudstone encountered was only a large block and not insitu mudstone. Small diameter drill holes in the area have often shown the presence of soft seams of materials at some depth (30m) indicating large blocks of mudstone underlie the site.

Six more 1m diameter shafts were sunk in the south portal area to define the hard mudstone surface. Generally, where past movement was evident, colluvium was found to overlie hard mudstone with considerable localised water inflows. In two holes, hard broken and crushed mudstone was encountered before reaching an intact block. This is thought to be broken slide debris.

2.4 Borehole Inclinometers

Five inclinometer access tubes have been installed in the area surrounding the south portal. The holes in which the tubes were grouted were advanced and logged until it was considered they would terminate in a large, stable, (hopefully insitu) block of mudstone. This usually involved a minimum of 40m of drilling.



PORO-O-TARAO INCLINOMETER SURVEY HOLE C FACES 3 - 4
BASE ESTABLISHED NOV 1973
RELATIVE DISP (M)
.10
.08
.06
.04
.02
.00
-.02
-.04
-.06
-.08
-.10
.00
4.00
8.00
12.00
16.00
20.00
24.00
28.00
32.00
36.00
40.00
44.00
48.00
DEPTH (M)
✕ AUG 1974
+ APR 1974
▲ JAN 1974
⊙ DEC 1973

FIG 2

The access tubes have been monitored periodically to detect and record subsurface movement. A typical plot of recorded displacements is shown in Figure 2.

As yet no significant movements have been recorded. It is not known whether the trend shown in Figure 2 is in fact significant; further observations over a long period will be required before an interpretation can be made.

The inclinometer survey will also provide a warning of accelerated movement caused by construction activity and define the depth and extent of movement. Precise surveying of surface markers is being undertaken concurrently.

2.5 Piezometers

Electrical (vibrating wire) piezometers were installed in five of the shafts at the level of the colluvium-mudstone interface. Following standard practice the piezometers were placed in sand lenses and sealed by bentonite-cement cutoffs on either side. A malfunction in the frequency counter has prevented a series of recordings being obtained, although individual readings have been used in stability calculations.

2.6 Rainfall Measurements

Rainfall measurements have been kept at the site. Average monthly rainfall over the 1973 winter approached 200mm with a peak rainfall of 80.5mm over a 24 hour period. Unfortunately no records of borehole or standpipe piezometer water levels have been kept and correlated with rainfall measurements.

3. LABORATORY TESTING

Laboratory testing has been carried out on a range of samples obtained from the site with a view to obtaining representative strength parameters for use in stability analyses.
Tests have included:

(1) Triaxial testing of undistrubed samples of mudstone and siltstone, recovered from small diameter drill holes, to determine insitu strength parameters.

(2) Shear box tests under high confining pressures (up to 6 MPa) on samples trimmed from blocks to determine insitu strength parameters.

(3) Triaxial tests on samples of colluvium, trimmed from larger block samples, carved from the walls of exploratory shafts.

(4) Shear box tests on remoulded samples of mudstone and siltstone to determine residual strength parameters.

Representative effective stress strength parameters from the tests listed above are included in Table 1.

The associated range of classification tests, and a series of consolidation tests on selected samples, were also performed. Test results have been reported by Parton (1974).

Mineral analyses revealed the colluvium to contain traces of illite, chlorite, halloysite and montmorillonite clay minerals.

TABLE 1

	C'(kPa)	ϕ'	Mass Density
Undisturbed siltstone and mudstone (triaxial and shear box)	40 ? 300	57.5 60^{o}	2.2 t/m^3
Colluvium	15	33^{o}	1.65 t/m^3
Remoulded siltstone and mudstone (residual strength parameters)	0	16^{o}	

4. LANDSLIDE DETAILS

Details of landslides in the Ohinemoa valley are most prominent in aerial photographs. Surface inspection of the lower slopes shows the hummocky ground described in the introduction with evidence of local slumping.

Depths to hard mudstone have been established on the slope outside the southern portal. The thickness of colluvium (slide debris) overlying harder rock is related to the landslide origin. Smaller slides today probably only affect the superficial cover of colluvium while larger, older, slides involved the mudstone bedrock.

Large debris slides probably formed as a result of oversteepening of slopes during an earlier period of rapid downcutting along the valley of the Ohinemoa Stream. Massive failures of the valley sides probably began several thousand years ago, the present landslides being only a remnant of much larger unstable masses (Riddolls, 1972).

The surface of slides outside the south portal are inclined overall at about 9.5^{o}. Stable slopes above the slides are more steeply inclined, up to approximately 60^{o}.

While landslide movement has probably occurred intermittently over several thousand years, it is likely that stability has been further reduced by disruption of natural drainage with the removal of native bush and the railway construction.

On the northern side of the ridge (above the north portal) the existing county road ascends across old slumps which have left bare scarps in the country above the road. Below the road, slump debris has flowed inwards toward the railway track. Kermode (1972) reports that several planes of old movement were revealed when the earthflow was trimmed back in 1971.

The south edge of the ridge contains scarps resulting from large scale rotational slumping. Kermode has postulated a series of rotational slumps and earthflows which can be correlated with damage to the existing tunnel lining. His interpretation implies that, of the 1071m of tunnel, approximately 300m at the southern portal and 200m at the northern portal has been damaged directly by slumping or earth pressure resulting from slumping. His schematic interpretation of the sequence and extent of slumping at the southern portal is presented in Figure 3.

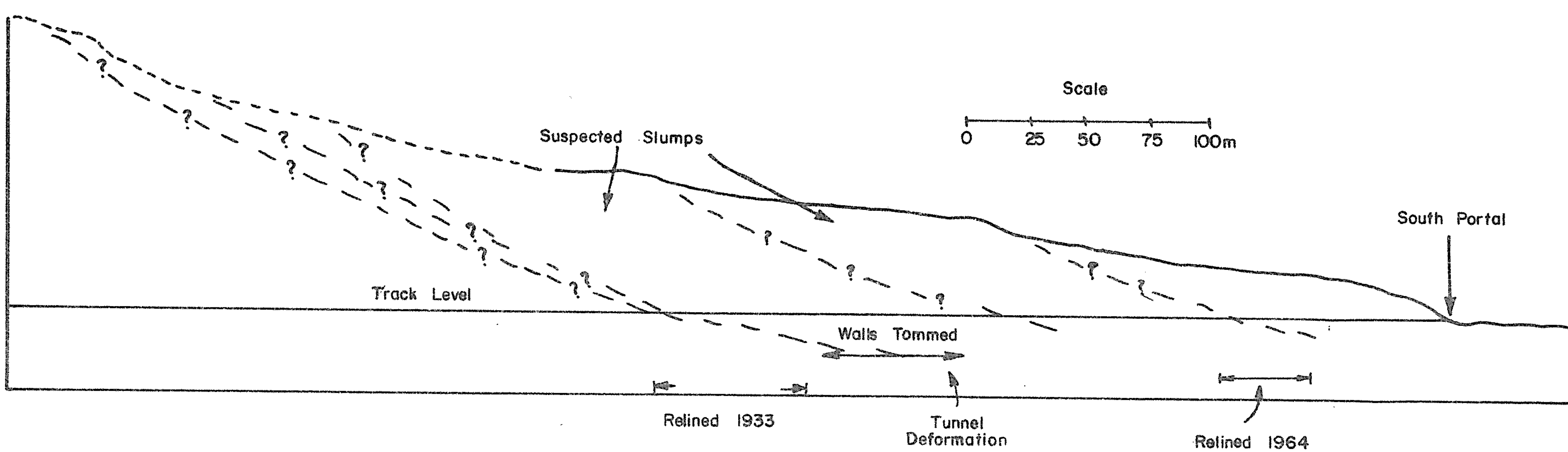


FIG 3

(After Kermode -(1972))

The investigations described earlier in this report have been focussed on the lower slopes adjacent to the railway formation. It is the stability of these slopes that is considered critical in redevelopment of the railway alignment.

Between Poro-o-tarao and Waimiha (approximately 10 km), Riddolls (1972) has investigated a series of similar landslides. The pattern again is typical of multiple retrogressive landsliding. Examination of the scarps, by the author, revealed large cracks against the sole of the uppermost slip down which surface runoff was flowing. Active landsliding is shown in the distortion of fence lines. Inspection shafts at the foot of the slopes revealed mudstone overlain by colluvium as at Poro-o-tarao.

5. STABILITY ANALYSIS

5.1 Introduction

An excellent state-of-art review of the stability of natural slopes has been presented by Skempton and Hutchinson (1969). Skempton and Hutchinson discuss the types of multiple and complex landslides that have been recognised and classified. They define a multiple retrogressive landslide as one which ".... develops from a single failure by the occurrence of further retrogressive failure which interact to form a common basal slip surface".

Multiple retrogressive landslides may be predominantly rotational or predominantly translational in nature, becoming more translational as the number of component rotational slips increases (c.f. Kermode's interpretation, Figure 3).

Skempton and Hutchinson state that, given such a situation (where the active mode of failure becomes translational) the shear parameters remain constant at their residual values, and the influence of varying groundwater pressures has a pronounced effect on the incidence of the slides.

Skempton and Hutchinson describe slump earthflows, which occupy a position between rotational slides and earthflows, and slides in colluvium, which are slides in colluvial debris which has accumulated at the foot of degrading cliffs. It is believed that evidence of all these types of slides is visible at Poro-o-tarao.

5.2 Mechanics of Stability Analysis

Advances in slope stability studies in recent years have been confined largely to the rotational, compound and translational type of slide. Falls, flows and mass movement have seldom been investigated quantitatively.

The most commonly used method of slope stability analysis is the limit equilibrium method where a condition of incipient failure is postulated along a given slip surface of known or assumed shape. A quantitative estimate of the factor of safety with respect to shear strength is then obtained by considering the equilibrium of the soil mass above the given slip surface. This method will be used in calculations summarised in this paper.

Generally, methods of stability analysis are subdivided according to the postulated shape of the failure surface. Briefly, these may be (in order of increasing complexity):

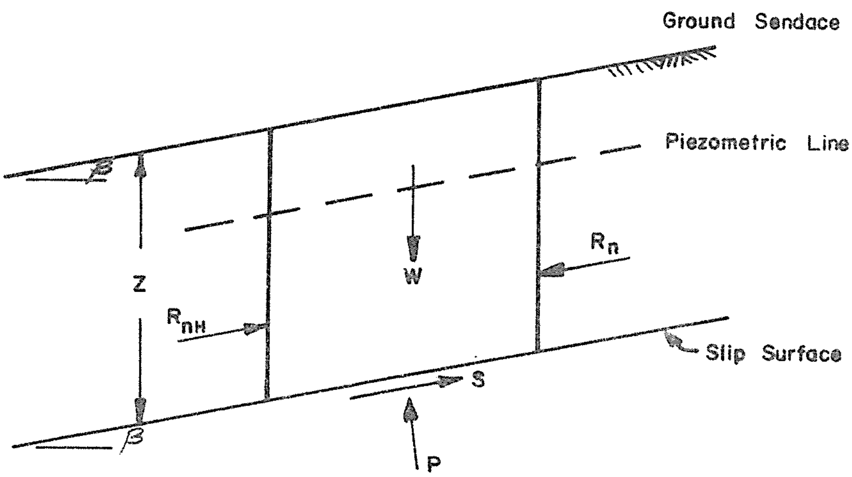


FIG 4a
Infinite Slope Analysis

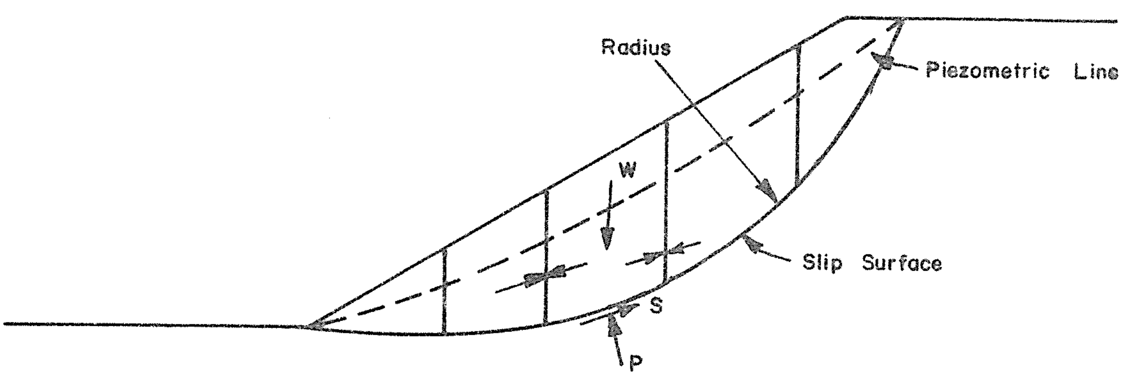


FIG 4b
Circular Shear Surface

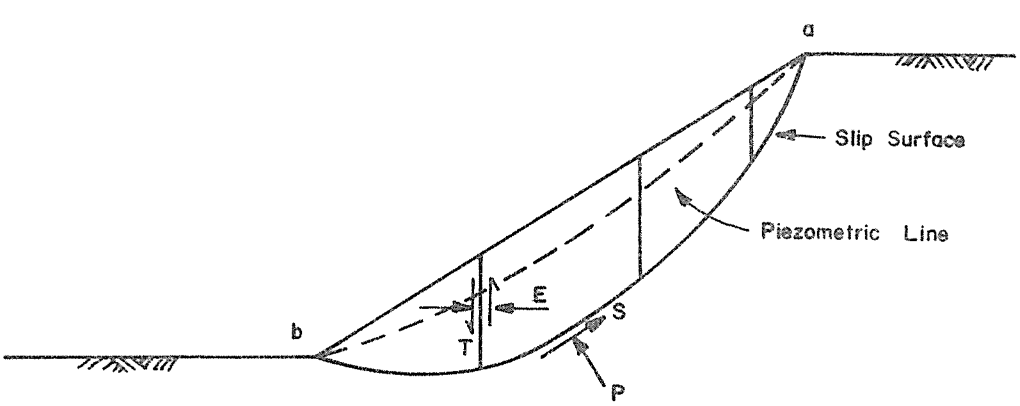


FIG 4c
Noncircular Shear Surface

(1) Planar slides in infinite slopes (Figure 4a).

(2) Circular slip surfaces (Figure 4b).

(3) Non-circular slip surfaces (Figure 4c).

It is not intended to make a critical examination or even appraisal of the analysis methods listed above. Instead it will be left to the reader to investigate the mechanics of the different approaches where it is considered necessary.

5.3 Subsoil Profiles at the South Portal

The pattern of drill holes and inspection shafts has made it possible to infer subsoil profiles along several lines. Stability analyses have been performed along selected lines using each of the three methods listed above.

A key plan and soil profiles are presented in figures 5 through 7. A consistent pattern of colluvium overlying hard mudstone is revealed. While the mudstone may only be the upper surface of large slumped blocks, it is considered that the colluvial slopes are the result of comparatively recent activity and their stability is most critical.

Drilling has revealed crushed and shattered zones at depth, indicating that the underlying mudstone is composed of large blocks. However, the extent or size of the blocks is indeterminate and it has been assumed they are now stable. Without this assumption the slope stability problem becomes insoluble.

The assumed piezometric lines marked on the profiles are the results of borehole observations correlated with piezometer readings.

5.4 Infinite Slope Analysis

The factor of safety of a slide on a planar surface of failure in an infinite slope may be shown to be expressed by:

$$\frac{c' + (\rho g\ Z\cos^2\beta - u)\ \tan\ \phi'}{\rho g\ Z\sin\ \beta\ \cos\ \beta} \qquad (1)$$

where c', ϕ' = effective stress strength parameters

ρ = mass density

Z = thickness of slide

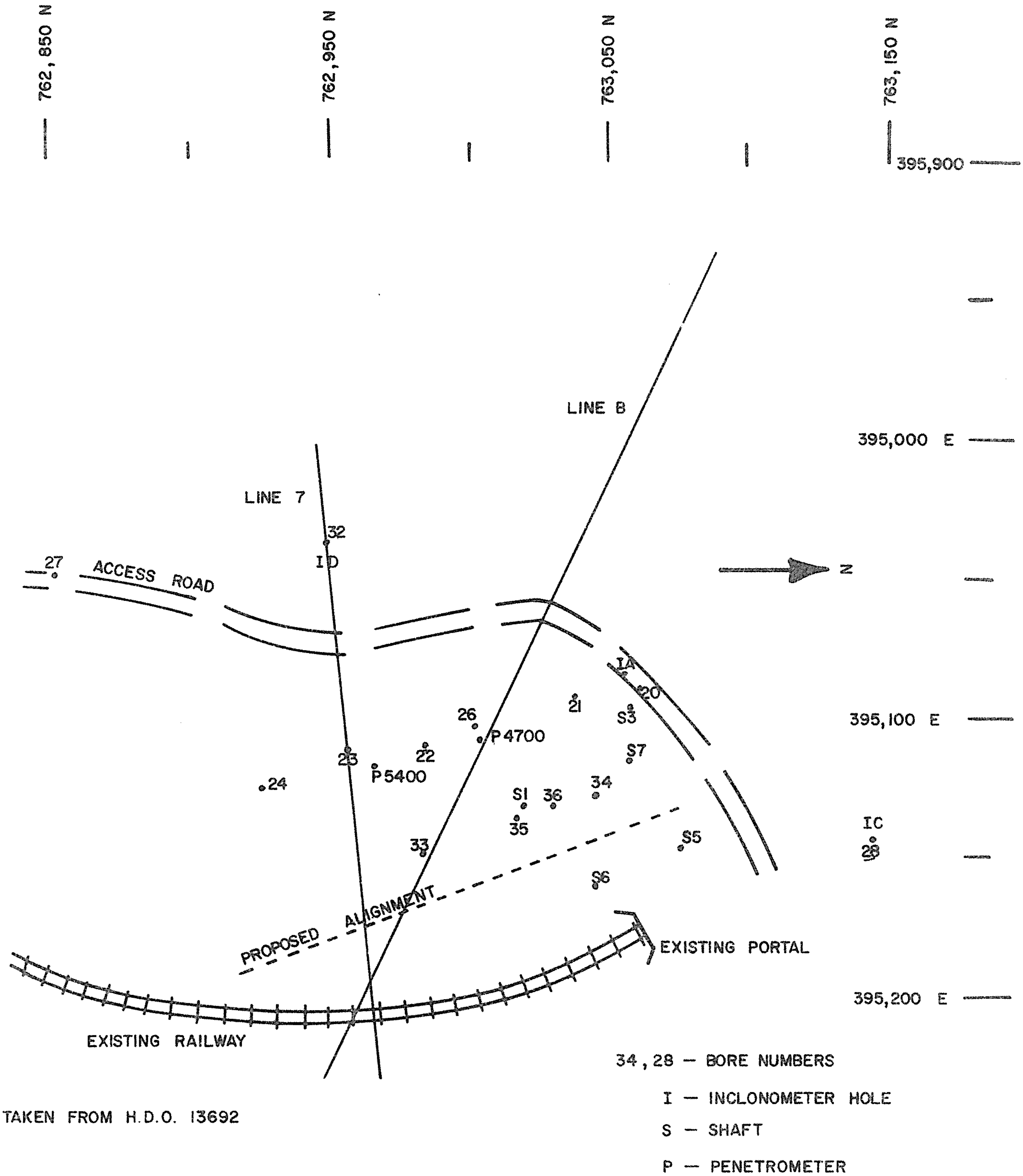
u = pore water pressure on failure surface

β = slope angle

Observations of ground water levels in boreholes has enabled the water table depth to be established. These observations can be correlated with piezometer measurements.

Figure 6 shows the subsoil profile along line 'B'. Water level measurements in borehole 26 stabilised at the time of drilling (November 1973) at a depth of 5m. Piezometer readings in shaft S2 indicated a piezometric head of 44.1 kPa (May 1974) at the colluvium - mudstone interface.

Assuming the soil profile in the vicinity of shaft S2 and borehole 26 to represent "average" conditions for an infinite slide type of analysis and using the laboratory-determined soil parameters presented in Table 1, the following factors of safety were obtained. The depth of slide was taken as 12m.



KEY PLAN SHOWING
INVESTIGATIONS AND SECTION LINES

FIG. 5

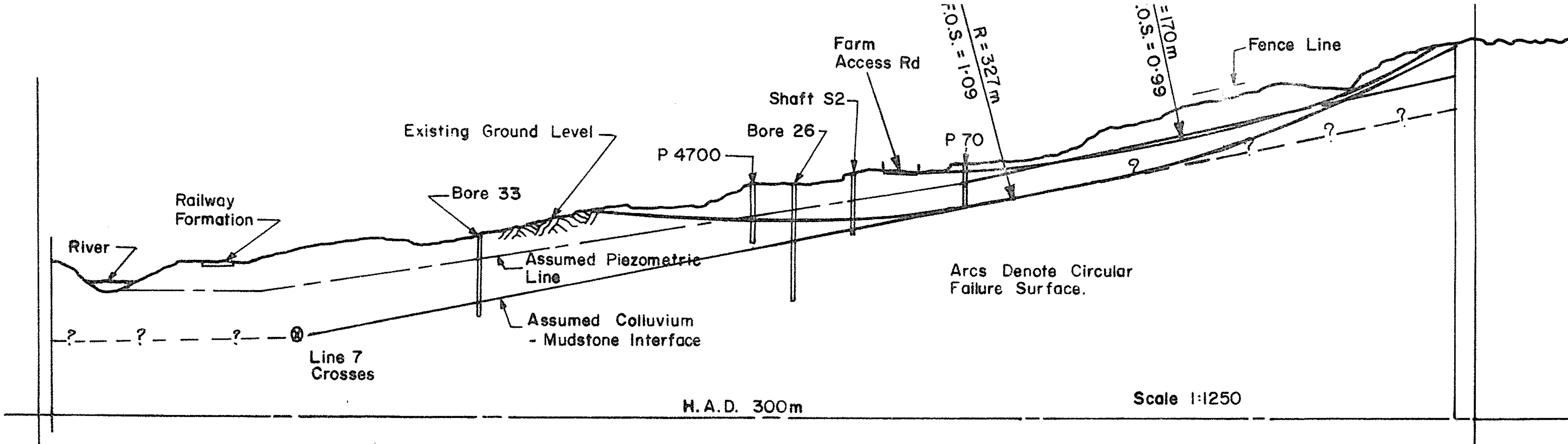


FIG 6

SECTION ON LINE 'B'

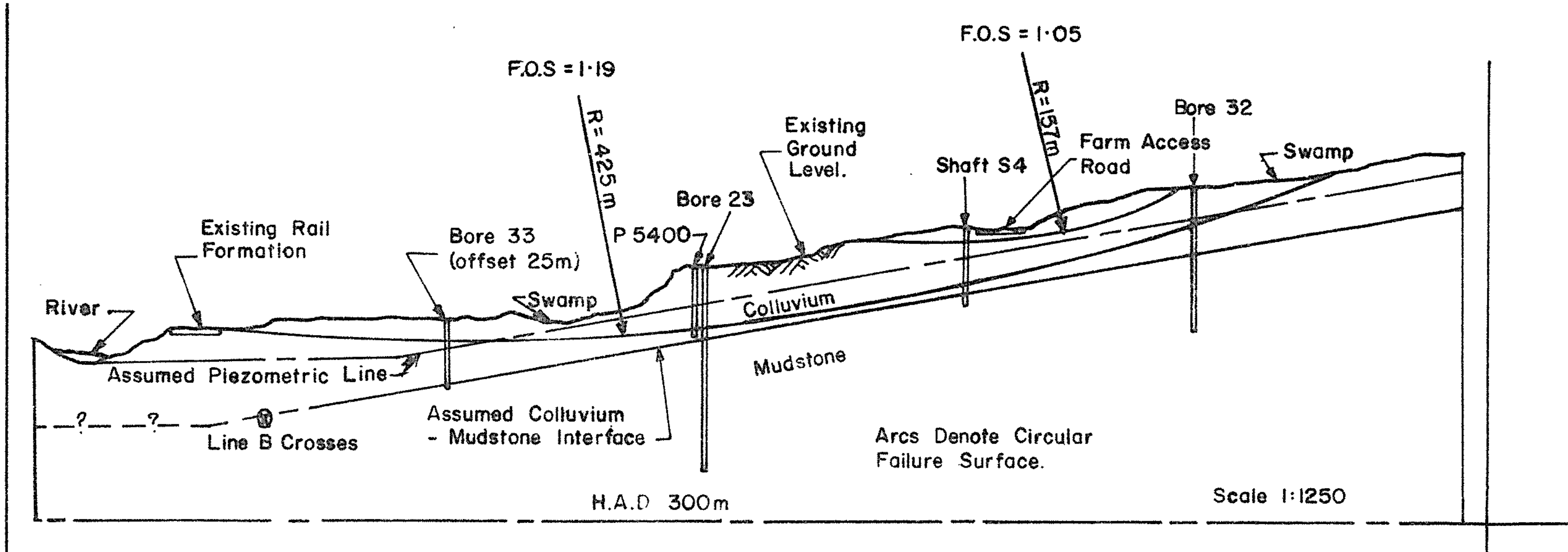


FIG 7

SECTION ON LINE '7'

TABLE 2

Pore Water Pressure Conditions	68.67 kPa (7m head)	44.1 kPa Piez. reading	85.34 kPa (8.7m head)
Factor of Safety	1.11	1.32	0.96

The last column in Table 2 has been included to demonstrate the effect of varying pore water pressure conditions. In winter months the water table has been noted at a depth of 3.3m (8.7m head) in certain boreholes. The implication of this is readily evident!

Similar results have been obtained along line '7' (Figure 7) and are presented in Table 3. The "average" conditions along the slope have been taken between borehole 23 and shaft S4. The pore-water pressures have been obtained from observations in borehole 23 (November 1972) and from electric piezometer readings in shaft S4 (May 1974).

TABLE 3

Pore Water Pressure Conditions	68.67 kPa (7m head)	68.2 kPa (Piezometer reading)
Factor of Safety	1.11	1.11

5.5 Slip Circle Analysis

Stability calculations using the classical slip circle analysis were performed on an IBM 370 computer using the ICES-LEASE program. The program has the ability to handle any given soil profile and porewater pressure conditions. Factors of safety according to the Bishop "simplified" method and the "Swedish" method are computed.

The two basic limitations of stability calculations using ICES-LEASE are:

(1) Only failure surfaces that are arcs of circles can be analysed.

(2) The simplified methods used do not satisfy statics.

While (2) above is generally accepted, because it is thought that the errors incurred are small or there is usually a judicious cancellation of errors, the condition imposed by (1) above, may not be insignificant in this case where a planar failure surface is thought to exist.

Stability analyses were performed on each of the profiles shown in Figures 6 and 7. Initially a coarse grid was specified above the profile and the program left to compute the stability over the mesh of trial centres. The radius having a minimum factor of safety at each centre was located. A search routine was then initiated, at centres having low factors of safety, to search out the local minimum value.

Analysis on Line 'B'

Shown on Figure 6 are the subsoil profiles and piezometric line as determined by investigation and discussed in section 5.3. These data have been used to assess slope stability.

Computed factors of safety for two slides are shown on Figure 6. Because of the length and depth of the slide, critical slip circles are confined to the upper portion of the slope where there is a slight steepening of the land form.

The critical slip circle (FOS = 0.99) is the true factor of safety for the slope. However, the second and larger slip circle has been included because it involves a larger slide and has only a slightly increased factor of safety (FOS = 1.09). Many other slip circles with higher factors of safety were computed.

Analysis on Line '7'

Again the data used in the stability analysis are marked on Figure 7. Computed factors of safety are marked for two slides. The critical slide is high up the slope with a factor of safety of 1.05. Of more concern is a larger slide involving the whole slope with a factor of safety of 1.19. Again numerous other slides were computed with higher factors of safety.
Results are summarized in Table 4.

TABLE 4

SECTION	FACTOR OF SAFETY	RADIUS (m)
Line 'B'	0.99 1.09	170 327
Line '7'	1.05 1.19	157 425

5.6 Analysis by Generalised Procedure of Slices

The basic principles of this method were laid down for the first time at the Conference on Stability of Earth Slopes, held in Stockholm in 1954, by Janbu. Since then many other authors have dealt with the problem of stability analysis for shear surfaces of any shape. Papers by Morgenstern and Price (1965) and Nonveiler (1965) are well known. The method used in this paper to analyse slope stability using a generalised shear surface was described by Janbu (1973).

Figure 4c shows a cross-section of a slope. The soil mass between the ground surface and the assumed (or known) shear surface is divided into a number of slices by vertical lines. For stability analyses the shear strength parameters must be known and their values may be different from slice to slice. Horizontal and vertical loadings within the soil mass or at ground level can be taken into account.

The output from the calculation (which is an iterative procedure) includes average factor of safety, normal and shear stresses along the shear surface and the horizontal and vertical interslice forces.

A computer program has been developed by the author (in conjunction with the Systems Laboratory, M.W.D. Head Office) to analyze slope stability by the generalized procedure of slices. Apart from eliminating the need for time consuming iterative hand calculations (which may take 3-4 hours for each slide analyzed) the use of computer facilities makes it possible to analyze quickly and effortlessly a large number of slides.

Figures 8 and 9 show the slides analyzed with the failure surfaces marked by dashed lines. Of prime concern was the question of stability of the whole slope - a case that could not be analyzed accurately by the slip circle method due to the length and depth of the slide. Several failure surfaces have been postulated and the effect of tension cracks at the upper end of the slide investigated, and the effect of the crack filling with water.

Results of stability analyses along Line 'B' are presented in Table 5. Groundwater conditions and soil strength parameters are identical to those used in the foregoing analyses, described in sections 5.3 through 5.5.

TABLE 5

TRIAL NO.	FACTOR OF SAFETY	BOUNDARY CONDITIONS
FS1	1.62 1.55	Tension Crack Tension Crack plus water
FS2	1.47 1.42	Tension Crack Tension Crack plus water
FS3	1.46 1.36	Tension Crack Tension Crack plus water
FS4	1.14 1.08 1.03	No Crack, c.f. LEASE example, Table 4, 1.09 Tension Crack Tension Crack plus water
FS5	1.74	Tension Crack plus water

Results of stability analyses along Line '7' are presented in Table 6. Again, identical soil parameters and ground water conditions have been used.

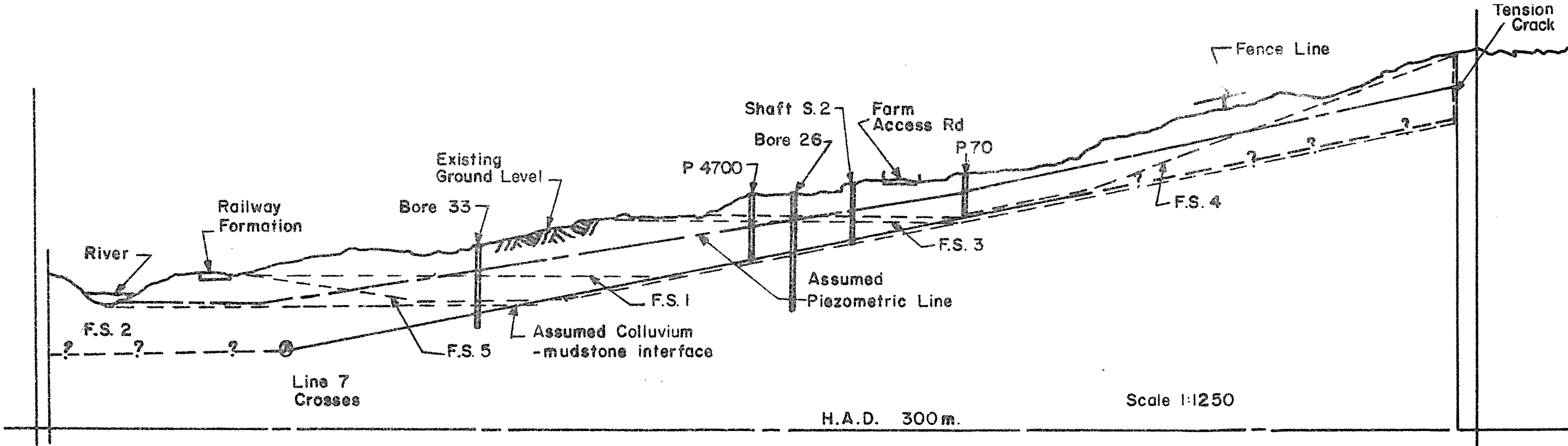


FIG 8

SECTION ON LINE 'B'

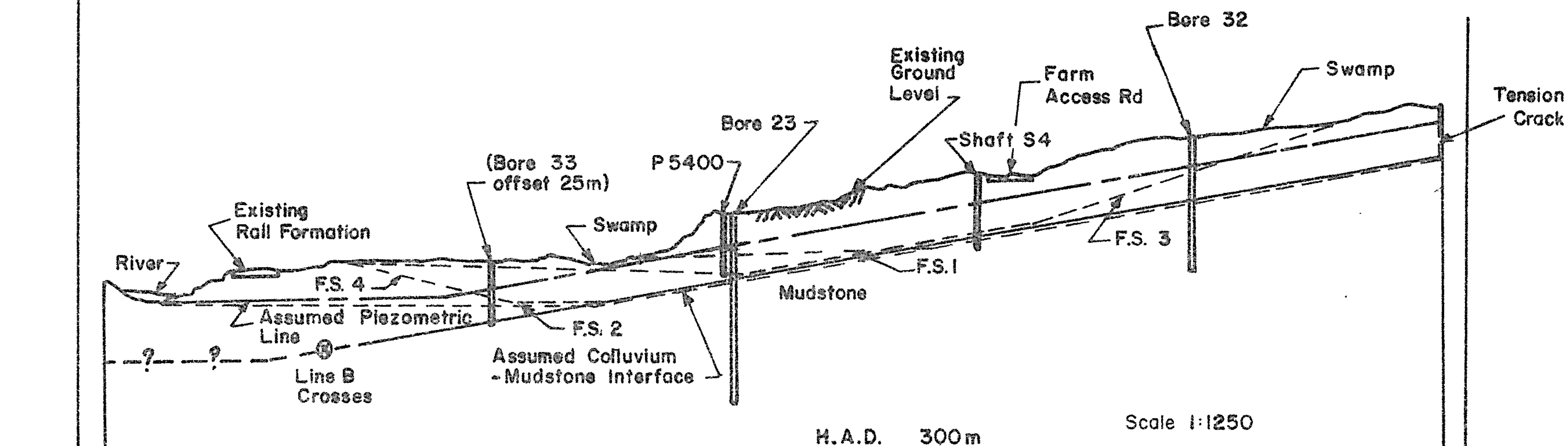


FIG 9

SECTION ON LINE '7'

TABLE 6

TRIAL NO.	FACTOR OF SAFETY	BOUNDARY CONDITIONS
FS1	1.45 1.54 1.69	Tension Crack Tension Crack plus water No Crack (Tensile force)
FS2	1.66	Tension Crack plus water
FS3	1.19 1.17 1.12	No Crack, c.f. LEASE example, Table 4 (1.19) Tension Crack Tension Crack plus water
FS4	1.41	Tension Crack plus water

6. DISCUSSION AND APPLICATION OF RESULTS

6.1 Comparison of Presented Results

Factors of safety determined from the infinite slope and the slip circle analyses are in close agreement. Factors of safety of several slides analyzed by the generalized procedure of slices have been presented, those for slides of similar configuration to the circular slides being in close agreement. It is important to note that the factors of safety presented from the slip circle analyses are the minimum values selected from the hundreds (literally) of slip circles analyzed.

Factors of safety determined by the slices method may be higher than those computed by infinite slope or slip circle analyses. This difference is a result of the method of treatment of interslice (side) forces by the latter methods and will be discussed briefly in Section 6.2.

In Tables 5 and 6 the effect of tension cracks is demonstrated briefly. It is assumed in the analysis that a tension crack forms at the head of the slide. The crack may become filled with water, the resulting fluid pressure exerting a force on the uppermost slice and decreasing the factor of safety. The effect of a tensile force existing within the soil is also demonstrated in Table 6, trial FS1.

Clearly the effect of a tension crack forming - and filling with water - should be considered in a stability analysis. Crack formation can be simulated using slip circle analysis by an adjustment of the soil cohesion, but the effect of the crack filling with water is more difficult to analyse.

Finally, a comparison between the infinite slope and procedure of slices has been made. Using a single slice and neglecting side forces, a factor of safety of 1.09 was obtained which compares favourably with the value presented in Table 3.

6.2 Comparison of Methods of Analysis

The three methods used above to analyze the natural slope at Poro-o-tarao employed the limit equilibrium method. The value of the shear strength, s, at any point on a potential shear surface is dependent upon the normal effective stress σ' at that point, as shown by

$$s = c' + \sigma' \tan \phi'$$

in which c' and ϕ' are the Mohr-Coulomb effective stress shear strength parameters.

The problem of determining the distribution of normal stress on the shear surface involves more unknowns than there are equations of equilibrium. Thus, in order to solve the problem it is necessary to increase the number of equations or reduce the number of unknowns.

All limit equilibrium methods of slope stability analysis employ assumptions to reduce the number of unknowns to be equal to the number of equations of equilibrium. Some methods, like Janbu's generalized procedure of slices and the analysis of Morgenstern and Price, satisfy all equations of equilibrium. That is, two equations of force equilibrium and one equation of moment equilibrium for each slice, giving 3N equations in total (where N is the number of slices).

Other methods, like Bishop's Simplified Method and the Swedish Circle Method (Ordinary Method), do not satisfy all conditions of equilibrium. Bishop's method satisfies vertical equilibrium for each slice and overall moment equilibrium, giving N + 1 equations, while the Swedish method satisfies only overall moment equilibrium giving one equation.

Comparative studies of the equilibrium methods by Whitman and Bailey (1967) have led to the following conclusions:

(1) The values of the factor of safety using Janbu's generalized procedure of slices or the method of Morgenstern and Price are very nearly the same. Any method which satisfied all conditions of equilibrium was found to give virtually the same value of factor of safety for any reasonable set of assumptions employed.

(2) Values of factor of safety calculated by Bishop's simplified method are generally comparable to the rigorous solutions described above. Bishop's simplified method may give rise to discrepancies of up to 7% (i.e. under-estimate the factor of safety).

Finally, Whitman and Bailey make the following comments on the use of the accurate methods of slices for complex problems where non-circular slip surfaces are postulated:

> "....such problems are, of course, strongly indeterminate and an engineer's intuition as to the form of a reasonable solution becomes severely taxed. Hence..... the method should .... be used primarily for the following purposes:
>
> (1) In practical work, to study failure surfaces other than circles and those composed of two or three straight lines.
>
> (2) In practical work, to check the reasonableness of solutions obtained by simpler methods".

6.3 Application of Computed Results and Improvement of Slope Stability

Initial estimates of the factor of safety against sliding, along two lines, have revealed that the slope is marginally stable. This conclusion has been reached despite the fact that there is considerable evidence of movement having taken place in recent years.

A more rigorous method of analysis was applied to the portions of the slope showing critical factors of safety by simplified methods of analysis. In accordance with the comments of Whitman and Bailey (quoted above) the generalized procedure of slices has been used to check the initial calculations and to investigate other failure surfaces. For reasons of simplicity these failure surfaces have only been considered as straight line segments. Further study of the stability of these slopes may involve more complicated failure surfaces. In accordance with the findings of other authors, the more rigorous solutions may produce higher factors of safety along identical failure surfaces.

It may well be that stability analyses have shown the slopes to have a factor of safety only through a fortuitous combination of circumstances. While every confidence is held with regard to the accuracy of the laboratory determined soil strength parameters, a number of imposed boundary conditions markedly affect the result of slope stability calculations. The principal of these is groundwater level.

The piezometric data used in all the foregoing calculations are based on test results and observations made over the summer months and during what have been considered to be relatively dry winters. With an exceptionally wet period it is quite possible that groundwater levels could rise at least 25%, to give piezometric levels as high as that postulated in Table 2. In practice this corresponds to an average water table depth of 3.3m as observed in boreholes 20 and 23 after the winter months.

Recalculation of factors of safety, using the generalized procedure of slices, of the critical failure surfaces using the increased piezometric data was carried out. Results are summarized in Table 7.

The implication of the data presented in Table 7 is readily apparent. Assuming that the analyst has confidence in his data and analysis, the simplext and easiest method to increase the stability of a slope is to lower the groundwater level. This may be accomplished in two ways:

(1) The surface area of the slide is generally uneven, hummocky and traversed by fissures. Elimination of swampy depressions and proper collection and diversion of runoff in lined drainage ditches will aid substantially in drying out and controlling the slide.

(2) Subsurface drainage by horizontal drains or drainage galleries designed to intercept aquifers or drain water from the sliding surface will increase the factor of safety.

TABLE 7

FAILURE SURFACE	ORIGINAL FACTOR OF SAFETY	RECOMPUTED FACTOR OF SAFETY
Line 'B' FS4	1.14	1.02
Line '7' FS3	1.19	1.08
Infinite Slope (from Table 2)	1.11	0.96

Terzaghi (1960) has made the following comments on stabilization of natural slopes:

> "The most economical way of stopping such slides (in detritus slopes) consists in the construction of drainage conduits which tap the most important water veins and water pockets .... the instances are rare in which a detritus slide cannot be stopped by drainage alone".

A method commonly proposed to assist with the stabilization of landslides is that of reafforestation. The quantitative effects of this method are not known exactly to the author, but it has been suggested that transpiration and collection of water may reduce runoff by up to 30%. It is generally accepted that forest growth has two functions; drying out of the surface soil and consolidation by a network of roots.

7. SUMMARY AND CONCLUSIONS

Field investigations and laboratory testing of materials have provided sufficient data to enable slope stability calculations to be performed.

Drill holes and exploratory shafts have revealed the presence of what is thought to be planes along which movement has occurred. Stability analyses show that the slopes do have a margin of safety, but the actual value is strongly dependent upon the groundwater level. More rigorous methods than the Bishop simplified method have been used to calculate factors of safety.

The effect of tension cracks and their filling with water has been demonstrated, and applied to the particular problem. Clearly tension crack formation should be considered in any analysis.

Because of the low factor of safety computed, and the fact that boundary conditions strongly influence the result, other factors have been considered which might assist in providing a key to the accuracy of reliability of the calculations. In section 2.4 the installation of borehole inclinometers and surface markers was described briefly. It is considered that interpretation of the data obtained from the periodic monitoring of these devices can assist in assessing the reliability of the calculations.

To date no significant movements have been recorded on any inclinometer tubes (c.f. Figure 2). Additionally no significant movements have been recorded from the precise surveying of the surface markers. It is considered that the position of markers can be surveyed to within 0.003m using a computerised least squares process to reduce the effect of random errors or inconsistencies.

Two possible alternatives existed before the movement survey information was introduced:

(1) The slip is the result of some previous configuration of the topography and the slope, providing the present conditions are maintained, is now stable.

(2) That one or all of the measurements of soil properties, piezometric data or subsoil profiles, or the calculation methods, are in error and nearby known active slips should be analyzed to test the calculation methods and parameters.

On the basis of the substantiating evidence obtained from the movement surveys, it is concluded that the slopes are stable, provided drainage or groundwater conditions are not allowed to deteriorate. Obviously, every effort must be made to improve surface drainage and collect runoff into lined channels for diversion away from the area of the slip. Areas which are conducive to the formation of swampy depressions should be levelled out. The provision of horizontal drains or drainage galleries warrants further investigation.

Currently the drainage measures outlined above are being implemented. It is planned to reafforest the area when construction is complete to improve long term stability.

ACKNOWLEDGEMENTS

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TEMPORARY SUPPORT OF VERTICAL EXCAVATION IN WEATHERED SEDIMENTARY DEPOSITS

K.H. GILLESPIE

1.0 INTRODUCTION

During a stage of construction for the Boulcott Street Underpass in the Shell Gully section of the Wellington Urban Motorway, the Contractor, Wilkins and Davies Construction Company Limited, was required and instructed to make a temporary open excavation into a steep hillside slope to provide working space for construction of the western abutment. The site boundary, outside which the Contractor could not operate, was located only 12 feet beyond the back face of the abutment pile cap location. Ground surface level on the boundary ranged up to 50 feet above the proposed base of pile cap level. Thus the flattest open batter slope which could be created was on the order of ¼ horizontal to 1 vertical.

The Contractor asked the writer's firm to comment on the stability of such an open cut and, following detailed inspection, was given the opinion that it would be unstable; alternative solutions considered feasible from a geomechanics viewpoint were:-

(a) acquire more land to enable flattening of the batter to a slope of about 50° to the horizontal, or

(b) retain the slope by construction of a fully effective wall held back by rock anchors.

Following discussions and meetings with officers of Ministry of Works, the Contractor was instructed to have detailed proposals prepared for the tied back wall system. The Department obtained a special way-leave for the anchors to extend through into the neighbouring land.

This paper briefly describes the design, construction and ultimate removal of that wall, including an indication of construction costs.

2.0 SITE CONDITIONS

2.1 Topography: The site for the abutment construction was located on the steep slopes which form the western side of Shell Gully and extend up to Kelburn Park.

Proposed bottom of pile cap level was elevation 144 feet on Wellington City Corporation datum while actual ground surface level over the net pile cap area, before any work, ranged between elevations 154 and 174 feet on the east and west sides, respectively. East and north of the pile cap area the ground was level at about elevation 154 feet but on the west side it continued rising for a considerable height at an average slope of 1½ horizontal to 1 vertical.

2.2 Geology: From a geological reconnaissance of the proposed wall site, it was concluded that most would be overlain by a 3 to 5 feet thick mantle comprising generally natural soil deposits but in some places man-dumped rubbish. This was estimated to directly overlie insitu highly to moderately weathered greywacke (sandstone) and argillite(siltstone). From an examination of on-site and nearby cuts, it was concluded that the base of the required cut would be highly weathered greywacke, the centre would be interbedded highly weathered greywacke and moderately weathered argillite while the top (excluding the soil mantle) would be highly weathered greywacke again.

The southern part of the site was found to comprise a small valley in the rock surface which had been naturally infilled with fine angular gravel in a silty clay matrix interbedded with clayey silt strata. One of the silt layers, near the central portion of the valley, was found to be some 6 feet thick. The headward extent (westward) of the infilled valley could not be determined by surface observation.

Bedding of the rock was found to strike approximately north-south (almost parallel to the proposed excavation face) and to dip steeply to the west (into the face). Several well defined fault traces were observed in existing cut batters just north of the site; these were aligned parallel to the bedding strike and appeared to be "strike slip" faults. They dipped westward at 45° and 75°.

Jointing of the rock units was observed to be both numerous and dense, as is typical in the basement rock of Wellington. The interbedded bands of greywacke and argillite contained well developed joints aligned both parallel and at right angles to the bedding strike. Dominant jointing in the greywacke dipped at 75° to the south with a density averaging 5 joints per foot; a second set dipped at 50° to the west and a third set, dipping eastward at 20° to 30° had a density of 1 to 2 per foot. This last set was assessed as the only one with definite adverse alignment in respect to the proposed cut face position.

Serious consideration was given to the drilling of exploratory test holes; it was finally concluded that the very high cost of establishing the rig on the steep slope could not be justified.

3.0 DESIGN

3.1 Alignment: From discussion with the Contractor, it was decided that the temporary wall should be vertical and placed such as to leave 18 inches of clear working space for construction of the pile cap. Topographic details supplied by Ministry of Works indicated that, in this position, the top of the wall would reach a maximum elevation of about 176 feet, making the wall height on the order of 32 feet.

3.2 Concept: Based on previous experience with this type of rock in Wellington, it was concluded that the wall would have to be built from the top down. The maximum vertical cut to be made for each horizontal row of wall was set at 8 feet, also by previous experience.

The steep slope of the site severely restricted access to all but small crawler mounted equipment. Thus it was not feasible to contemplate the use of any form of continuous driven or drilled-and-cast-in-place soldier pile.

A review of available structural steel showed that 4" by 2" by 7 pound/foot and 6" by 3" by 12 pound/foot universal channels could be obtained. It was therefore decided that each eight foot high increment of cut would be supported by eight foot long vertical soldiers, each comprising pairs of rigidly spaced universal channels. The soldiers were to be held back by either one or two rock anchors and were to support timber lagging.

Again based on experience but also with a knowledge of readily available plant and material, it was decided that the rock anchors should comprise 1" and 1 1/8" diameter mild steel deformed rods grouted into pre-drilled holes of 4-inch diameter. Short threaded plain rods of larger diameter were to be butt welded on to the outer end of each anchor rod to allow for tightening a nut on to the soldier piles through a tapered washer. Guides shaped from ¼" diameter rod were to be welded on to each anchor rod at regular intervals to ensure it sat centrally in the drilled hole. The anchors were inclined at 15° to horizontal.

3.3 Design Parameters

In addition to the topographical features already described, the following design parameters were assumed:-

1.	Cohesion	500 PSF	lb./sq.ft.
2.	Angle of internal friction	30°	
3.	Bulk density	135 PCF	lb./c.ft.
4.	Working bond stress of grout to country	25 PSI	lb./sq.in.
5.	Ground water free draining through lagging		
6.	General shear failure likely		

The stability of the cut face was analysed by conventional wedge theory but with a great range of cases being explored by use of a Hewlett Packard Model 20 desk-top computer. This gave a maximum static load on the total wall height of 12.5 kips per foot length and also showed the inclination of the plane behind which anchors would have to obtain their bond.

A suitable horizontal spacing for the soldier piles was found to be 6 feet.

The design found that at the point where the wall was highest, seven rows of anchors would be needed. Required anchor capacities ranged from 3.5 to 21 kips and the maximum length of anchor (top row at highest point) was 29.5 feet. It was decided that, apart from a low-height wing at one end of the wall, all anchors should have a length of at least 15 feet.

4.0 CONSTRUCTION

Initial access for a bulldozer to the top of the cut line was gained by heaping up imported soil. The first vertical increment was then cut down, the anchors installed, the soldiers placed and tied and lagging put in position. Three successive and similar sequences followed down to design excavation level. During construction two selected anchors were load-tested to yield point of the anchor rod and one in every ten of all others was load-tested to twice design load.

The whole wall, comprising an overall length of some 90 feet, was completed in just over four weeks.

The gross cost per square foot, excluding recoverable items, was less than $10. This figure includes the expense of removing part of an old but heavy concrete wall which infringed on the site. Also, it does not allow for normal establishment costs as it was an extra item on an existing contract.

5.0 PERFORMANCE

The installation of the wall allowed the Contractor to work unrestricted and safely on the piles and pile caps. The Project Manager reports that he was entirely satisfied with the design concepts and procedures and would use them again in every detail on a similar problem.

It is of interest to note that the wall recently became redundant and the lagging plus soldiers were removed. Much of the face collapsed immediately.

PANEL ROW NUMBER	SOLDIER PILE NUMBER	ANCHOR NUMBER	ANCHOR LENGTH (ft)	ANCHOR ELEVATION (ft)	GROUT LENGTH (ft)	ANCHOR FORCE PSF	SOLDIER PILE LENGTH (ft)	SOLDIER PILE STEEL U.C.
	A	(a)	23.0	169.0	6.5	6400	PLATE	2/2x4"x7lb
	B	(a)	24.0	170.0	6.5	3500	3.5	"
	C	(a)	24.5	170.5	6.5	3500	4.0	"
	D	(a)	24.5	170.5	6.5	3500	4.5	"
	E	(a)	25.0	171.0	6.5	3500	5.0	"
	F	(a)	28.0	172.5	6.5	3900	6.0	"
	F	(b)	24.5	170.0	6.5	5500		"
	G	(a)	29.5	174.0	6.5	3900	7.0	"
	G	(b)	26.0	170.0	6.5	5500		"
	H	(a)	29.5	174.5	6.5	3900	7.5	"
1	H	(b)	26.0	170.0	6.5	5500		"
	I	(a)	25.0	173.5	6.5	3900	7.0	"
	I	(b)	2[illegible].0	170.0	6.5	5500		"
	J	(a)	23.0	171.0	6.5	5700	5.0	"
	K	(a)	2[illegible].0	170.5	6.5	3500	4.0	"
	M	(a)	18.5	167.0	6.5	4500	4.5	2/3x6x12lb
	N	(a)	12.5	164.5	6.5	6400	6.0	"
	O	(a)	10.0	164.0	6.5	7100	6.5	"
	P	(a)	8.0	162.0	6.5	3000	6.5	"
	P	(b)	7.3	158.0	6.5	4000		"
	A	(a)	18.5	163.0	6.5		PLATE	2/2x4x7lb
	B	(a)	18.5	166.0	6.5	6500	8.0	"
	B	(b)	16.3	161.0	6.5	8000		"
	C	(a)	18.5	166.0	6.5	6500	8.0	"
	C	(b)	16.5	161.0	6.5	8600		"
	D	(a)	19.0	166.0	6.5	6500	8.0	"
	D	(b)	17.0	161.0	6.5	8600		"
	E	(a)	19.5	166.0	6.5	6500	8.0	"
	E	(b)	18.0	161.0	6.5	8600		"
	F	(a)	20.0	166.0	6.5	8000	8.0	"
	F	(b)	18.0	161.0	6.5	9900		"
	G	(a)	21.5	166.0	6.5	8000	8.0	"
2	G	(b)	19.5	161.0	6.5	9800		"
	H	(a)	21.5	166.0	6.5	8000	8.0	"
	H	(b)	19.5	161.0	6.5	9900		"
	I	(a)	18.0	166.0	6.5	8000	8.0	"
	I	(b)	16.0	161.0	6.5	9900		"
	J	(a)	17.0	166.0	6.5	6600	8.0	"
	J	(b)	15.0	161.0	6.5	9100		"
	K	(a)	15.5	166.0	6.5	6300	8.0	"
	K	(b)	14.5	161.0	6.5	8600		"
	L	(a)	14.5	166.0	6.5	6100	8.0	"
	L	(b)	13.0	161.0	6.5	7200		
	M	(a)	10.5	~~[illegible]~~ 165.5	6.5	4100	8.0	2/3x6x12lb
	M	(b)	10.0	160.0	6.5	5000		"
	N	(a)	11.0	158.0	8.5	12000	6.5	"
	O	(a)	9.0	157.0	6.5	7500	4.0	"

PANEL ROW NUMBER	SOLDIER PILE NUMBER	ANCHOR NUMBER	ANCHOR LENGTH (ft)	ANCHOR ELEVATION (ft)	GROUT LENGTH (ft)	ANCHOR FORCE PSF	SOLDIER PILE LENGTH (ft)	SOLDIER PILE STEEL U.C.
	A	(a)	11.0	156.5	8.5	12000	PLATE	2/3x6x12lb
	B	(a)	15.0	157.0	7.5	11000	8	
	B	(b)	12.5	152.0	8.5	13000		"
	C	(a)	15.5	157.0	7.5	11000	8	
	C	(b)	13.0	152.0	8.5	13000		
	D	(a)	16.0	157.0	7.5	11000	8	
	D	(b)	13.5	152.0	8.5	13000		
	E	(a)	16.0	157.0	7.5	11000	8	
	E	(b)	13.5	152.0	8.5	13000		
	F	(a)	17.5	157.0	8.5	12000	8	
	F	(b)	14.0	152.0	8.5	14100		
	G	(a)	18.0	157.0	8.5	12000	8	
	G	(b)	15.0	152.0	8.5	14100		
3	H	(a)	18.5	157.0	8.5	12000	8	
	H	(b)	15.0	152.0	8.5	14100		
	I	(a)	16.0	157.0	8.5	12000	8	
	I	(b)	12.5	152.0	8.5	21000		
	J	(a)	15.0	157.0	8.5	11200	8	
	J	(b)	14.0	152.0	10.5	20200		
	K	(a)	12.0	157.0	6.5	10800	8	
	K	(b)	11.0	152.0	8.5	19100		
	L	(a)	11.0	157.0	6.5	9500	8	
	L	(b)	10.0	152.0	8.5	14300		
	M	(a)	8.5	157.0	6.5	8300	8	
	M	(b)	8.0	152.0	8.5	14300		
	B	(a)	14.0	148.0	12.5	15500	5.5	2/3x6x12lb
	C	(a)	14.0	147.0	12.5	15500	5.5	
	D	(a)	14.0	147.0	12.5	15500	5.5	
4	E	(a)	15.0	147.0	12.5	15500	5.5	
	F	(a)	15.0	147.0	12.5	16400	5.5	
	G	(a)	15.0	147.0	12.5	16400	5.5	
	H	(a)	15.0	147.0	12.5	16400	5.5	

CONSTRUCTION SEQUENCE:

1. Excavate to level I (El. 171.5 ft) and spread over natural surface to form a flat working platform.
2. Drill and grout rock anchors at the specified elevations and lengths.
3. Hand trim a vertical slot in natural ground for soldier pile.
4. Excavate in bulk to level II (El. 165.5 ft).
5. Drill and grout rock anchors at the specified elevations and lengths.
6. Fix vertical pre-fabricated soldier beams to grouted anchors for panel Row 1.
7. Place horizontal lagging for full height of panel Row 1.
8. Repeat the above sequence for the remaining panel rows.
9. Completely lag panel Row 2. For panel Rows 3 and 4 lagging may be only placed in areas of visual instability.
10. Allow for one anchor per ten grouted to be tested to 1½ times its working load. If permanent deflections of test anchors are recorded, additional anchors should be drilled and grouted.

NOTES:

1. The rock anchoring of the existing retaining wall at the northern end of the site should be carried out before footing is demolished.
2. Mild steel rock bolts should comply to N.Z.S. 1693.
3. If high tensile stressing anchors are used to provide a tied back system, anchors should be stressed to ½ their working load and released to ¼ their working load.
4. Grout should comprise the following components:
   - (a) Cement 112 lbs
   - (b) Water 4½ gallons
   - (c) Additive Intraplast "A" 1 lb

   Three test blocks of grout used in the anchorage system will be taken for each batch. Compression tests will be made at 7, 14 and 28 days.
5. Brickell, Moss, Rankine & Hill, Consulting Engineers, will be engaged by the Contractor to supervise bulk excavation, anchor testing and to determine the extent of lagging required.

GCA

CONTRACTORS MUST VERIFY ALL DIMENSIONS AT THE JOB BEFORE COMMENCING ANY WORK OR MAKING ANY SHOP DRAWINGS WHICH MUST BE SUBMITTED AND APPROVED BEFORE MANUFACTURE

G F E D C B A REVISION BY DATE

BOULCOTT STREET WESTERN RETAINING WALL EXCAVATION

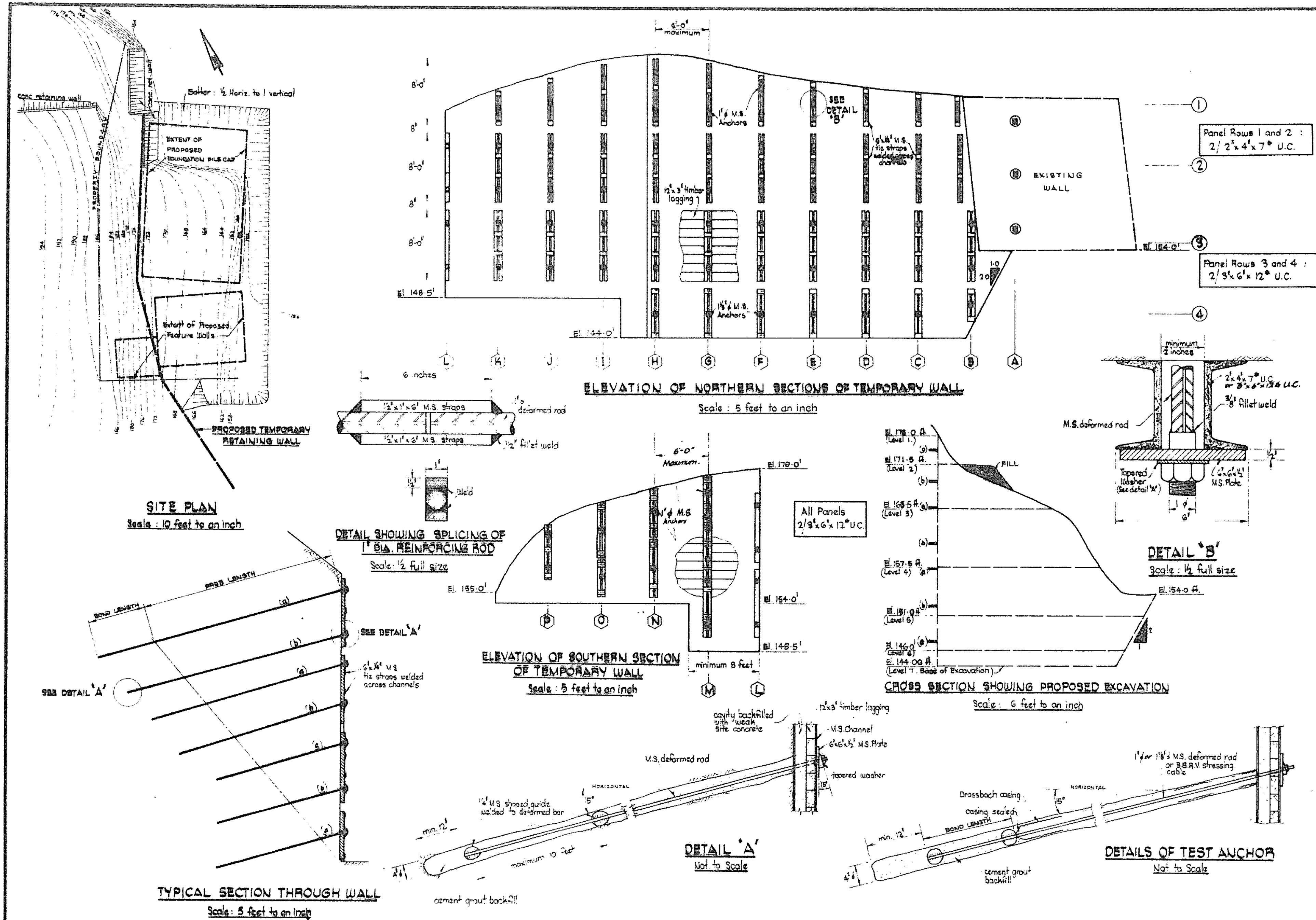
TEMPORARY RETAINING STRUCTURE

BRICKELL, MOSS, RANKINE & HILL
CONSULTING ENGINEERS — CIVIL · STRUCTURAL · SURVEYING · SOIL MECHANICS · FOUNDATIONS
WELLINGTON LOWER HUTT AUCKLAND DUNEDIN

SCHEDULE OF QUANTITIES

WILKINS & DAVIES CONSTRUCTION CO. LTD

SCALE N.T.S. | DATE 13.10.72
DRAWN GCA | TRACED K.A. | DWG No 5952
CHECKED K.N.G. | APP'D K.N.G.



CONTRACTORS MUST VERIFY ALL DIMENSIONS AT THE JOB BEFORE COMMENCING ANY WORK OR MAKING ANY SHOP DRAWINGS WHICH MUST BE SUBMITTED AND APPROVED BEFORE MANUFACTURE	REVISION	BY	DATE

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WELLINGTON, LOWER HUTT, AUCKLAND & DUNEDIN

DETAILS OF ROCK ANCHORS

WILKINS AND DAVIES CONSTRUCTION CO. LTD

SCALE As shown		DATE 13.10.72	
DRAWN GCA	TRACED TA	DWG. No. 5952	
CHECKED KNG	APP'D KNG		

G.C.A.

Chairman: *Dr. M.R. Johnston*

Section 5: REMEDIAL MEASURES AND CASE HISTORIES

PRESENTATION AND DISCUSSION

Mr. N.S. Smith, said the purpose of his paper "A Contractor's Viewpoint and Recommendations", was twofold, firstly to compare the various methods of prevention and correction of landslides, and relative costs; secondly to emphasise the necessity for early and thorough investigation of hillsides prior to residential subdivision in known unstable areas. In saying "known" unstable areas he referred to the fact that most local authority engineers would be aware of certain areas in their own locality where they knew that landslide problems were highly probable and would inevitably occur. As a remedial contractor, he said they were aware of many such areas and it was not unusual for them to be called in, not only during the earthworks of a subdivision, but again when excavations were being carried out for individual houses and driveways, and again after heavy rainfall when some slopes became unstable. It was clear that in some cases sections were being sold that should not be developed unless under the supervision of an experienced soils engineer. He had found many inexperienced people, and on occasions engineers, architects, surveyors and builders excavating with little regard for neighbouring properties. He felt there should be more control at local body level - preferably funded locally so that, when the problem occurred, work could commence quickly. He then showed slides to illustrate a case where a swimming pool was built on the head of a slump and where the slide had involved five or six houses.

With regard to the use of horizontal drains, Mr. Smith said this was very simple and an excellent way of getting water out of slopes quickly. He showed a slide depicting a flow of about 100 gallons per day which was typical and flowed continuously.

Another technique for stabilisation of batters was by grassing using sprayed-on seed, but the situation must be suitable. Also a suitable mixture of seed, nutrients and adhesive should be used. Clover should not be used on very steeply battered slopes because the clover could hold the moisture, from rainfall, causing sheet slips of banks. It was important to round off the top of the batter and get a good marriage of the sprayed-on grass with the existing grass on the top.

Mr. Smith agreed with previous speakers that there should be more control at local body level and that prompt assistance should be available to householders with landslip problems.

Mr. East, in speaking to his paper "Inclined plane slope failures in the Auckland Waitemata Soils", (p5.17) said he had discussed three typical landslips in Auckland. In the Auckland Waitemata residual soil they were the result of man-made excavations and were not anywhere near the size of the natural slips that had been discussed. Mr. East then showed slides of the slips described in the paper. In concluding his presentation, he stated that there were three steps in remedial measures against instability:-

1. Site investigation; 2. Appraisal of the failure mechanics; 3. Design of remedial measures.

Some engineers he suggested, tended to leave some of those steps out.

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Mr. Gillespie, then presented his paper, "Temporary support of vertical excavation in weathered sedimentary deposits", (p.5.57).

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Dr. I.M. Parton, said his paper "Assessment of slope stability at Poro-O-Tarao tunnel south portal", was a preliminary assessment of the stability of a slope in natural ground. He said he had put himself out on a limb in some respects as the two previous case histories had described remedial measures but the slope he would discuss had only been analysed and although remedial measures had been proposed they had yet to stabilise the slope. In that respect he could not comment on costs nor on the effect of the remedial measures. The slope was on a slightly different scale to some of those already described involving residential development, and could involve a whole hillside of some hundreds of acres. The analyses presented in the paper had been performed after extensive site investigation and back-up laboratory testing. Factors of safety were determined by three different methods which demonstrated that the role of groundwater in determining the stability of a slope could not be over-emphasised. In attacking the problem of devising suitable corrective measures to stabilise existing natural slopes many engineers opted for impressive retaining structures, without considering the more obvious solution of de-watering. A few simple sums would quickly reveal the ineffectiveness of these measures when ground water was not controlled. Dr. Parton said one example had already been given by Mr. East in his Selwyn College case history. It would appear from this account that investigation and analysis showed drainage to be the most productive measure of stabilisation. However, an on site decision dictated that a rock buttress be constructed without due consideration of the overall effect, and Mr. East had described the resulting event. Dr. Parton said that if one looked at the mechanics of the situation, a lot of rock, heaped on to the toe of an inclined failure plane, had little effect. He said it was fortunate that a few engineers published their failures as well as their successes and it was from these the best profit was gained. There were documented cases, like Mr. East's, where drainage had been installed only after the second failure and the slope had been subsequently stabilised. Generally, drainage was not considered effective initially. Investigations at Poro-o-Tarao had been necessary because tunnel replacement had dictated that a new track alignment be placed upslope of the existing track. Here was a case with an existing track and a very old tunnel constructed in the 1890s and the tunnellers who had built the first tunnel had picked the best spot for it. The new alignment would mean a deep cutting across the toe of a colluvial slope known beforehand to be of marginal stability and later verified by computation. Obviously the toe of the slope could not be just chopped off. He said the solution which first came to mind was a retaining wall to prevent movement but analysis revealed that should the groundwater level upslope rise during the winter months the shear resistance developed by a single retaining wall was insignificant compared to the driving forces acting. The solution proposed and which would be implemented this summer was to de-water the slope and increase its resistance to sliding. (This was illustrated by slides). The de-watering was done in two phases, the first the construction of a lined collector drain to intercept all surface runoff into the unstable area and elimination of all swampy and water-retaining areas above the drain. This was an expensive solution but seemed to be the only one. Then immediately downslope of the collector drain, a deep cutoff trench would be constructed to intercept the sub-surface flows known to exist along perched water tables within the colluvium. The trench would be constructed by installing large diameter vertical sand drains from the surface, draining into a driven drainage gallery located in the hard underlying mudstone. The rail cutting would be retained by means of rigid struts between parallel rows of cylinder piles installed from the surface. Excavation would proceed from within the retaining structure once the cylinder piles had been constructed and the struts placed. The success of the scheme in the long term would depend upon the water table in the colluvium being lowered and consequently the resistance to sliding at the toe of the slope being increased. The scheme could only succeed if the corrective measures installed were maintained, not only in the immediate vicinity of the slope, but on a much larger scale. Dr. Parton then further commented on the slide at Utuku which had been mentioned by Mr. Hancox. He said that in correcting or stabilising a slope in natural ground it may not be sufficient to concentrate one's efforts in the immediate vicinity of the slip. Water-retaining areas some distance from the area of greatest interest may have considerable bearing on the stability. Only by re-grading and proper drainage over a wide area could effective de-watering be achieved.

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Mr. Webley, opened discussion on Mr. Smith's paper by asking about the practicability of horizontal drains up to 200' long. Mr. Smith said he had found that in practice usually 30' to 100' lengths were ideal. Lengths up to 200' might be necessary in deep-seated slips to try to get the water from the back of the slip plane or tension cracks at the back of the slip. Where the bank was perhaps up to 30' in height, then with an 80' drain one could expect to lower the water table substantially, say to 30' or 40' back from the face of the batter. Any free-draining layers were good to use. In some cases they had found that when the bedding was away from the road, water was trapped behind.

Mr. Depledge asked Mr. Smith if there were ways of beautifying gabions.
Mr. Smith suggested creepers, such as ice plants and that the larger nurseries would have some very good suggestions. Another speaker enquired about bonding of gabions. Mr. Smith said that it was preferable to see them laid as blocks and bonded. Basically a gabion could be 4 metres long and that was divided into four cells so the next gabion would be over the middle diaphragm of the lower gabion.

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Mr. Mitchell noted the use of jute mesh for encouraging grass growth on slopes and asked if this had been tried in New Zealand.
Mr. Smith said that he was not aware that it had been tried. Jute or polypropaline mesh had been used overseas initially, the idea being that each mesh acted as a dam on a batter. In N.Z. the technique of spraying bitumen and paper which adhered to the soil grains was used instead. This particular method was spreading overseas very rapidly.

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Dr. Eyles referred to section 4.30 of Mr. Smith's paper where it was implied that vegetation could be allowed to increase indefinitely. He said that in Wellington this past winter many slips had occurred on very steep slopes which had been stable for many decades, slopes which were often vegetated by mature gorse or regenerating broad leaf native bush, thick scrub. It seemed to him in slopes of 45° where the weight component of the vegetation was acting downslope and therefore increasing the tendency to movement rather than the so-called stick component in increasing stability, that the optimum vegetation should not be a regenerating forest but a vegetation of low bio-mass - one which protects the ground from run-off and from soil cracking and therefore penetration below the surface by water and yet which would not contribute to the downslope component. Mr. Smith agreed that the vegetation cover was a controversial subject.

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Mr. Riley mentioned that rockbolts were used overseas and he asked Mr. Smith to comment on the use of these in the slopes around Wellington. Mr. Smith said he had had no experience at all in rockbolting but he thought that such techniques as this together with wirenetting would be very suitable for some of the steep rock slopes.

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Mr. Carryer said that in his experience rockbolting was useless in areas where intense jointing was found, but were most useful in failures along bedding planes. Mr. Smith disagreed. If the rockbolts were deep enough and large enough they should be satisfactory. Rockbolting in conjunction with wirenetting should solve the problem.

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Mr. Chave said that rockbolting had been used with great success on jointed schist rock which predominated in some areas in Dunedin.

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Mr. Buhla said he was intrigued by the suggestion of a local body tax to help local authorities overcome some of these problems. He asked if Mr. Smith had given any thought as to what form this tax should take and how it could be levied.
Mr. Smith quoted Mr. Gill's figures for earthquake and landslide cover. He suggested that a similar arrangement be adopted for land stability. He felt the insurance companies could collect this for the local authorities who would then have control of this fund. When a slip occurred the council should be empowered to take immediate action to remedy the situation. They should also have power to bring in two experienced engineers to assist - unbiased, non-council members.

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Mr. Gunson suggested as an alternative to the insurance companies collecting tax, the government allocating subsidised funds for these particular needs, for investigation and remedial works.
Mr. Smith said that if that was done he felt it would be too long in getting any action. He felt it was a matter to be left entirely in the hands of the local bodies. In cases where considerable funds would be required, perhaps then there could be a central fund to call on. It was necessary to have prompt action to prevent further damage to neighbouring properties.

Mr. D.K. Taylor referred to the slides Mr. Smith had shown where surface erosion on slopes had occurred as a result of water coming from the top of the slope and suggested there should have been some water control. Mr. Taylor asked for some details of how far back the ditch should be, should it be lined, open or rock-filled and how did one cope with the maintenance problem. Mr. Smith said it had been noticed when doing remedial work that a large number of slips occurred where there had been a field tile drain or a sewer along the top of a slope. The water got into those trenches. The water unfortunately did not all go down the field tile drain. It tended to lie on the bottom of the trench under the invert of the pipe and continually seeped into any weak zone which may pass under that pipeline. It appeared to him that the drains were being put within the potential slip circle and therefore all the drains should be well behind the potential slip circle and not within it. The drain could take the form of an open drain well back behind the slip circle or it could be piped, but it must be well back from the top of the batter and only the engineer's analysis could determine how far back it should be from the top of the slope. Cut-off drains, for the same reason should be well back and should be sealed at the top to prevent ingress of surface water but they should have a small half drain at the top, or preferably a flexible type of bitumen surfaced drain so that all the surface water would collect in it and be led away from any potential slip area.

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Mr. Bargh asked what was felt about the drainage of step batters.
Mr. Smith said, if the drainage above the step batter was done properly then there was seldom the problem of the batters or the steps themselves failing. He said he was against drains in batters, but if it was a big catchment area then he would agree that a drain go in but it should be sealed. It was necessary to get rid of the water quickly.

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Mr. Kayes said that in one of the slides Mr. Smith had shown, the gabion blocks going in were being placed below a house which had had fill coming down on the top and had stated that this should not be allowed to happen. He asked how Mr. Smith thought this should be controlled - should a building permit be required for any private owner to put fill over a certain depth anywhere on his property? Mr. Smith said this was a controversial subject. He said he had suggested that local bodies should control excavations in known unstable areas. It was his opinion that local authority engineers should say that a certain area was unstable and that in such areas no machine excavation should be allowed without a permit. He could tell the owner that he should get a qualified engineer to come and supervise the works and then he could excavate. The problem was that anyone could hire a bulldozer and cut into the natural slopes.

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The following discussion took place at a later session, chaired by Mr. T.J. Kayes:

Dr. Hughes opened discussion on the last three papers in this section, by addressing Mr. East, asking how long after construction did failures occur. Secondly, he suggested that in the Waitemata series as well as a tension crack there had to be some zone in which the soil was failing. As well as having water-retention cracks there must be high pore pressures so that the point of drainage was really to relieve the high pore pressures and increase the effective stress at that level. There were three drainage possibilities: counterfort drains, a cut-off drain at the top, or horizontal drains. He wondered whether anyone had used other sorts of drains and whether they were being successful. He then directed a question to Dr. Parton who had tested some rock at a confining pressure of about 6 MPa. That was equivalent to an overburden stress of 300 meters. He asked why it was tested at such high stress.

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Mr. East said that as far as the timing of the slips was concerned the Massey slip occurred some two years after construction, the Henderson substation slip was after 3-4 years and the Selwyn College slip was a mystery. He believed it occurred almost as it was constructed. He said it was a new playing field constructed through an old slip. He said he agreed about the importance of reduction of high pore pressures. He did not necessarily counterfort to horizontal drains. Although there were some conditions where it was impossible to use counterforts, they did give added site investigation.

Dr. Parton said with reference to testing intact samples of mudstone and sandstone there was relevance to tunnel design and using conventional soil mechanics testing equipment with pressures up to 2000 kPa, they could not get a large enough pressure range to define the failure envelope. To solve the problem it had been put in the hands of Professor Taylor.

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Professor Taylor said that it was generally true that in testing the materials one had to go well above the practical range of stress in order to define the failure envelope reasonably closely.

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Mr. Ireland said that slope failure adjacent to playing fields was found in Wellington as well as Auckland, and suggested that the Works Department's approach to embankment construction might be the problem. Mr. East did not disagree, but pointed out that it was the mechanism of failure which he was primarily considering.

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In reply to a question from *Mr. Smith*, the depth of counterforts at the Massey slip was given by Mr. East as 15 to 20 ft. Mr. Chave enquired about the construction method. This was by hydraulic backhoe. The trenches had proved stable. If not, horizontal drains would have been used. In response to a question from Mr. Riley, concerning drains in the counterforts, Mr. East said there were no pipe drains in the counterforts themselves. A toe drain collected water from the counterforts. Mr. Slimin raised the topic of slotted drain pipes installed at depths up to 24 ft. with a mole plough, which he had seen done in the U.K.

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Mr. Crampton said that in Nelson after the 1962 landslide where houses were damaged they had tried to find methods of preventing people from building on some of the affected sections and had found there was no way in law in which they could have an endorsement put on the title showing the section was no longer suitable for building. As a result of this they had introduced a system called the Conditions Book in which they had entered such items. It was not used extensively until 1970 when they had a lot of stability problems in the city and found that many of the sections were suspect. People could refer to this book at the council and make enquiries about the stability of sections for building. The problem was that most people did not know it existed. He felt it desirable that an endorsement should be put on the title. He understood that at the present time some thought was being given to Memorandum of Encumberances but not much progress had been made. Mr. Crampton said that after 1962 a by-law had been introduced at Nelson requiring everyone making an excavation in ground that was steeper than 1 in $3\frac{1}{4}$ to obtain a permit from the local authority but it had not been very successful. Mr. Crampton said there had also been a system of Registered Engineers' Certificates introduced for new subsidivions. When new subdivisions are being considered a registered engineer is required to certify that sections are suitable for residential development. These could be given at the scheme plan or at the final plan stage. With reference to drainage, Mr. Crampton said that they had become more strict on drainage measures and had found it desirable to have drains affecting an individual property within that property boundary, not on one next door. If it was at the bottom of a section, the person above would not be very concerned about it, whereas the person below would be. With regard to stability and stormwater problems, they had tried to get the several residents to work together to arrange a scheme but where one or two of them were not prepared to do anything the scheme just did not proceed.

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Mr. Couch (Hamilton) asked Mr. East whether they had ever measured the depth of tension cracks and compared them with their forecasts.

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Mr. East said there seemed to be no way of doing so. They had tried to push a steel tape down one or two of them up to something like 15 ft. but they were very irregular.

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Mr. Couch said that this was a very critical factor in assessing instability particularly in the Bay of Plenty and Waikato where the stronger soils were the upper layers which had high linear shrinkage characteristics. He said he would be interested to hear from anyone who knows something more about it. This concluded the session.

DETERMINATION OF RELEVANT INFORMATION FOR THE ASSESSMENT OF THE STABILITY OF SOIL SLOPES

J. P. Blakeley

1.0 INTRODUCTION

As distinct from slopes in filled ground, slopes in natural soil deposits are rarely formed in what can be regarded as essentially uniform material. The most important of the stages in correctly assessing the stability of an earth slope in natural ground is to determine the significant soil properties and the position of the soil strata or discontinuities which are most likely to cause instability. Both with regard to the siting of boreholes and the laboratory testing programme, this must be firmly borne in mind.

2.0 INFORMATION REQUIRED TO DEFINE THE PROBLEM

2.1 Evaluation of Nature and Distribution of Strata

Before any worthwhile analysis can be carried on the stability of a slope in natural ground, it is essential not only that reliable soil strength data is available on all the soil deposits which are present adjacent to the slope and which may influence stability, but also that a reasonably accurate idealised soil profile can be drawn on which to base the analysis. This is rarely an easy task. Often it is very difficult, requiring a greater amount of judgment than the development of the soil strength properties to be used in the analysis.

For any complex slope problem, the engineer would be well advised to seek assistance from a competent geologist. A good understanding of the mechanisms by which a natural slope has been formed or the manner in which soil strata have been laid down in the area of a proposed cutting, can help immeasurably in a development of an idealised soil profile for the natural slope or for the cut. Geological advice is usually obtained for the analysis of rock slopes, but is frequently overlooked or ignored completely if the slope is predominantly or entirely in soil deposits. It is essential that no geological factor of importance in the engineering analysis of a soil slope is overlooked in the analysis. Comprehensive discussion of such geological factors is contained in References 1 and 2.

The soil layers to be used in the idealised soil profile drawn up for analysis are not necessarily limited to horizontal or dipping layers, it may well be that the critical soil layers are steeply inclined or even vertical. Methods of slope stability analysis available today have the facility to handle layers of this type.

2.2 Determination of Slope Geometry

Another important factor in obtaining the idealised soil profile is to obtain an accurate definition of the geometry of the slope. Ideally this should be obtained from an accurate cross section surveyed down the slope at the position to be analysed for stability. If this is not possible, cross sections may be drawn up with lesser accuracy from contour plans of the slope.

For most types of stability analysis, the idealised soil profile must be reduced to a two dimensional problem for analysis. If the cross sectional properties vary considerably within a short distance in the direction perpendicular to the section being analysed, this must be taken into consideration in interpretation of the results of the analysis. Generally the section chosen to be analysed for stability will be that considered to be the most critical. Hence if there is considerable variation in the third dimension this will generally tend to increase the factor of safety rather than to reduce it.

3.0 SITE INVESTIGATION PROCEDURES

3.1 Categories of Site Investigation

There are two main categories of site investigations carried out into the stability of soil slopes.

(a) Those carried out at a construction site (power projects, reservoirs, highways, housing subdivisions etc.), to determine if a slide is likely to occur.

(b) Those carried out where a slide has already taken place to determine the mechanism by which the slide occurred and what remedial measures should be taken either to stop the slide or to prevent similar slides occurring in the future.

In the second type of investigation analysis will be continued until it has been positvely established how the slide occurred because it is obvious to all that at the time of the slide the safety factor was less than one. It is essential that thorough analysis be carried out for investigations of the first type, or else the safety factors obtained will be meaningless and will give a false sense of security.

In the case of investigations of previous slips, it is important to note that the factors which triggered off the initial slide may not be the same as the factors which cause subsequent movements. Once a slope has been weakened by an initial movement it may be much more susceptible to future movement than a slope which has not moved in the past (due to general weakening of the ground).

3.2 Field Reconnaissance

Before any site investigation drilling is commenced, it is essential that a close study of the area surrounding the slope or proposed cutting is made. This study should be aimed at detecting any topographic features on the ground surface which may indicate instability. These may include leaning or curved tree trunks which can suggest either ground movement or soil creep, an irregular ground surface profile or irregular fence lines, areas of slope where vegetation appears to be younger than elsewhere, large cracking in the ground surface (particularly where there is an obvious ground displacement across the crack) and surface seepage in the face or at the base of a slope.

If a previous slip in similar strata in the general area of the site can be detected, this may give valuable information as to the likely mechanism of slope failures in the area and hence may be of assistance in drawing up the idealised soil profile. In this regard, studies of the aerial photographs can be most valuable, particularly if a series is available which extends over a period of time.

In areas of residential development, any previous damage to houses within the area will often have been well documented. Also it is important to find information on damage which has occurred to roads and drains or other underground services in the general area due to slipping. This information would generally be available through the Local Authority.

Observation of cut batter slopes used in previous development of an area will often give valuable information as to the safe angle for cut batters. Erodability of cut batters should also be studied as the selected cut batter angle may often be a compromise between stability against erosion and stability against slipping.

3.3 Site Investigation Drilling

Because of the frequent non-uniformity of soil strata in a slope in natural ground, the actual soil conditions to be encountered generally cannot be reliably predicted in advance of site investigation. Hence, if at all possible, it is recommended that under these circumstances site investigation should be carried out as a two-stage or even as a multi-stage process with careful review of the results obtained from each stage before the following stage is planned in detail.

The first stage should consist of the best possible coverage of investigation bores over the area with continuous coring to define the soil strata which are present as completely as possible, and hence to define the significant soil strata. It is especially important to observe any thin very weak layers or layers which could be weakened by water intrusion - these are often referred to as "greasybacks". Hence, any slope investigation in cohesive soils must be open to question if continuous coring is not carried out in the initial drillholes.

The objective of the second stage should be to put down further investigation bores either with insitu testing, or with best quality undisturbed soil samples, or both, in order to define the properties of significant soil layers for subsequent soil analysis. In many cases these second stage bores will be adjacent to previous bores so that required sampling depths can be accurately known in advance. The second stage investigation may also include further bores with continuous coring in order to clarify any anomalies, or any discontinuities in the areas between the first stage bores.

If for any reason it is not possible to conduct a site investigation programme in stages, then the investigation must allow for maximum flexibility for changes to be made as the results come to hand, with careful engineering input throughout the investigation.

Although it may be possible for any one stage of a site investigation to be carried out to a set programme, it is just not possible for a complete site investigation of a slope in natural ground to be treated in this way if a meaningful result is to be obtained. Engineering input and consequential changes during the investigation are essential.

3.4 Undisturbed Soil Sampling

If reliable answers are to be obtained regarding the stability of a slope, it is essential that undisturbed samples of the best quality for laboratory testing should be obtained during the site investigation drilling, and that the handling and transportation of samples to the laboratory should be carried out in a most careful fashion. These aspects are covered in References 3 and 4 and are not discussed in this paper. Although sample disturbance will generally lead to a slope stability analysis giving conservatively low answers, there is no way of determining just how conservative the analysis may be and hence such an analysis must be regarded as rather unsatisfactory.

Also, it is doubtful if sophisticated slope stability analysis by computer can generally be justified if the input data used for the analysis is regarded as unsatisfactory because of sample disturbance. In this situation a computer analysis can lead to a most misleading view being obtained as to the accuracy of the results produced.

It is essential both prior to taking undisturbed soil samples for testing and subsequent to taking these samples, that the available borelog information and soil profiles are carefully studied to ensure that the samples taken are indeed likely to be representative of a particular layer in the idealised soil profile to be used in the analysis. If there is any doubt on this question, then either the idealised soil profile should be modified, or (preferably) further samples should be taken and tested to obtain additional information on the strength properties of that particular layer within the idealised soil profile.

3.5 Insitu Soil Testing

Insitu soil testing is mainly of use in determining relative strengths of various soil strata, particularly in soils which are likely to be sensitive to sample disturbance and hence where it is difficult to get a good indication of insitu soil strength from the core recovered.

Information on relative strengths of soils can be extremely valuable in defining the boundaries of the various soil layers which will make up the idealised soil profile even if they are of limited use in determining the soil strength properties of any particular layer.

Insitu shear vane testing is mainly applicable to soft cohesive clay. If the angle of internal friction ϕ can be assumed to equal zero, then shear strength values can be obtained from these tests. However, these will be total stress values whereas effective stress parameters are more likely to be relevant to the analysis of slopes in natural ground, as will be discussed later in this paper. Insitu shear vane testing can, however, be most useful in assessing the effects of sample disturbance on laboratory results of tests carried out on soft cohesive clays which are generally very susceptible to sample disturbance. The vane test results can be used directly to assess the sensitivity of the clay if both the undisturbed and the remoulded clay strength are measured. Also, if the vane shear strength is compared with a laboratory unconfined compression strength test or undrained triaxial test carried out on soil taken from a particular sample tube at a similar depth, the likely amount of sample disturbance can be gauged.

Other methods of assessing insitu soil strength (and hence the relative strengths of various soil strata) include standard penetration tests (good for cohesionless soils but less reliable for cohesive soils) cone penetrometer tests - both static (Dutch Cone) and dynamic, and the Menard pressuremeter. However, none of these methods give shear strength parameters directly for use in stability analysis, although estimates of approximate shear strengths can be made from published correlations of test results.

4.0 LABORATORY TESTING AND APPLICATION OF RESULTS

4.1 Principals of Total and Effective Stress

In order to understand the basis of the types of laboratory triaxial tests used in slope stability analysis, as well as to understand the analyses themselves, it is necessary to understand the principals of total and effective stress. These are outlined briefly below but can be found explained in more detail in soil mechanics textbooks.

The effective stress acting on an element of soil at any particular depth in a soil mass is equal to the total stress acting at that depth minus the pore-water pressure acting at that depth. In steady state conditions the pore-water pressure depends only on the height of the ground water-table above the element under consideration. However, during transient conditions after the shear stress acting on the soil element is increased or decreased, there will be either a building up or a dissipation of pore-water pressure.

Total stress shear strength parameters obtained in the laboratory are dependent on the drainage conditions applied during the triaxial test. However, effective stress parameters are independent of the drainage conditions applied during the triaxial test - either the test is carried out so that no pore-water pressure is allowed to build up or, alternatively, the pore-water pressure build-up is measured and allowed for in obtaining the parameters.

Two basically different approaches are then available for the analysis of slope stability:

(a) If effective stress analysis is being used, the pore-water pressures which exist on the potential failure surface are estimated and used in an equation involving effective stress parameters, in order to obtain the strength on the failure surface.

(b) If total stress analysis is being used, the laboratory tests are performed in a manner which is designed to simulate the conditions in the slope and the shear strength is determined in an equation involving total stress parameters without pore-water pressure measurements being necessary. It is assumed that the pore-water pressures which develop in the sample during the laboratory test will be the same as those which would develop in the embankment at failure.

4.2 Immediate and Longterm Stability Analysis

For any man-made slope, the stability must be considered both immediately after construction and in the long-term. Immediately after construction there will be transient pore-water pressures due to the loading or unloading of the slope, but in the long-term these will dissipate to steady state pore-water pressure conditions. In the case of a cut slope in natural ground, the construction will tend to unload the natural ground and this will generally lead to transient pore-water pressures which are negative with respect to those achieved in the long-term after construction. This means that immediately after construction the soil will be stronger than in the long-term. Hence, the long-term stability will be the critical consideration for a cut slope in natural ground. In the case of a natural slope, immediate stability is not relevant and long-term stability only need be considered.

For immediate stability problems, total stress analysis is generally used as it is easier for analysis to simulate the field situation in a laboratory triaxial test than to predict the transient pore-water pressure which may be present under these conditions. On the other hand, for long-term stability problems, effective stress analysis is generally used as it is relatively easy to predict pore-water pressures under steady state conditions.

Hence it can be said that, since for the analysis of both natural slopes and cut slopes in natural ground, it is long-term stability which is critical, then it follows that the analysis of the stability of these slopes should always be in terms of effective stress.

4.3 Types of Triaxial Test

There are basically three types of triaxial test:

(a) Unconsolidated Undrained (or Undrained) Tests

Drainage is not permitted during either the application of the consolidation pressure or the deviator stress.

(b) Consolidated Undrained Tests

Drainage is permitted during the consolidation phase but not during the application of the deviator stress. All pore-water pressure generated during the consolidation phase must be allowed to dissipate before the deviator stress is applied. The pore-water pressure generated during application of the deviator stress may be measured if so required.

(c) Consolidated Drained (or Drained) Tests

Drainage is permitted during both the application of the consolidation pressure and the application of the deviator stress. All pore-water pressure must be allowed to dissipate before the deviator stress is applied and the deviator stress must be applied at a sufficiently slow rate for there to be no pore-water pressure build-up during test.

4.4 Application of Triaxial Test Data

Tests of Type (a) and Type (b), as defined in Section 4.3 above, give total stress parameters, whereas tests of Type (c) give effective stress parameters. However, effective stress parameters can also be obtained from tests of Type (b), if pore-water pressure build-up during application of deviator stress is measured and this has the advantage that tests can be carried out in a much shorter space of time than tests of Type (c). Hence this type of test is more widely used than tests of

Type (c), although pore-water pressure measuring equipment is required.

For the analysis of natural slopes and of cut slopes in natural ground, since as has been stated in Section 4.3 the long term stability is the critical consideration, drained triaxial tests or consolidated-undrained triaxial tests with pore-water pressure measurements should be carried out on selected representative soil samples in order to obtain suitable effective shear strength parameters for analysis.

5.0 PIEZOMETRIC LEVELS

5.1 The Need for Measurements of Ground Water-Table

In order to carry out an analysis of the long term stability of slopes using effective stress analysis, it is necessary to make the best possible estimate of the steady state pore-water pressures which will act on potential failure surfaces. This in turn will involve taking measurements of ground water-table over the slope area. These measurements should be as accurate as possible and preferably continued over a period of time in order to obtain seasonal fluctuations and to obtain the highest expected water-table levels which will generally occur during the winter months.

5.2 The Effects of a Rise in Ground Water-Table

The effects of seasonal variations in ground water-table level on the stability of a slope adjacent to the sea or a river or lake is illustrated in Fig. 1.

It can be seen that a rise in ground water-table decreases slope stability in two ways:

(a) an increase in pore-water pressure acting on a potential failure surface (the major effect)

(b) an increase in the overturning moment due to increasing soil bulk density as the water-table rises (a very much smaller effect).

5.3 Methods of Ground Water-Table Measurement

Ground water-table measurements are frequently carried out in site investigation boreholes over a period of time following the investigation. Often the boreholes are left uncased and if the borehole has collapsed at a level above that of the potential failure surface, a misleading high water level may be obtained as an indication of the pore-water pressure acting on the failure surface. Hence at the same time as the water level readings are taken, the depth to which the borehole is open should be recorded. The installation of slotted plastic tube in the boreholes to help prevent collapse is a recommended procedure.

On the other hand, a ground water-table measurement in an uncased borehole may give a misleading low water level if there is a permeable layer in the slope above the level of the potential failure surface, and this is a more serious problem as it will lead to an under-estimate of the pore-water pressure which is likely to act on the potential failure surface, and hence to a stability analysis on the unsafe side. If there is any possibility of such a situation occurring, it is essential that water level observation holes be cased off and sealed to below the level of the permeable layer, and preferably down to the level of the potential failure surface.

If the ground water-table is at a considerable depth below the ground surface, an electric water level measuring device is recommended for accurate measurements.

5.4 Changes in Ground Water-Table Level Due to Excavation

In the case of the evaluation of the stability of a cut slope in natural ground, the site investigation would generally be carried out in advance of construction. Hence it will be necessary to estimate the amount of draw-down in the ground water-table which will result from the excavation of the slope and after steady state conditions are reached. This requires the exercise of a considerable amount of judgment, and if there is any doubt, a conservatively low estimate of the amount of draw-down should be made.

It should also be noted that if the draw-down of the ground water-table level takes place at a very slow rate, it is possible that stability immediately following construction could become critical and should be given careful consideration.

6.0 EFFECTS OF SEEPAGE PRESSURES

6.1 Slopes in Sand

Lambe and Whitman (Reference 5) give an example of seepage parallel to the slope in an infinite slope in sand. The example establishes the fact that whereas, without seepage parallel to the slope the sand slope will stand at an effective angle of internal friction ϕ', when seepage is considered, the maximum possible stable slope is only about half that for no flow.

The other important fact which arises from this example is that the problem is analysed in two ways. In the first case the disturbing force due to the weight of the element is taken as the total weight of the element and no external forces are considered to act on the sides of the element. In the second case the disturbing force due to the weight of the element is taken as the buoyant weight of the element and in addition a force called the seepage force is imposed on the element parallel to the direction of flow. The seepage force equals the product of the volume of the element times the unit weight of water times the hydraulic gradient (which is established by means of a

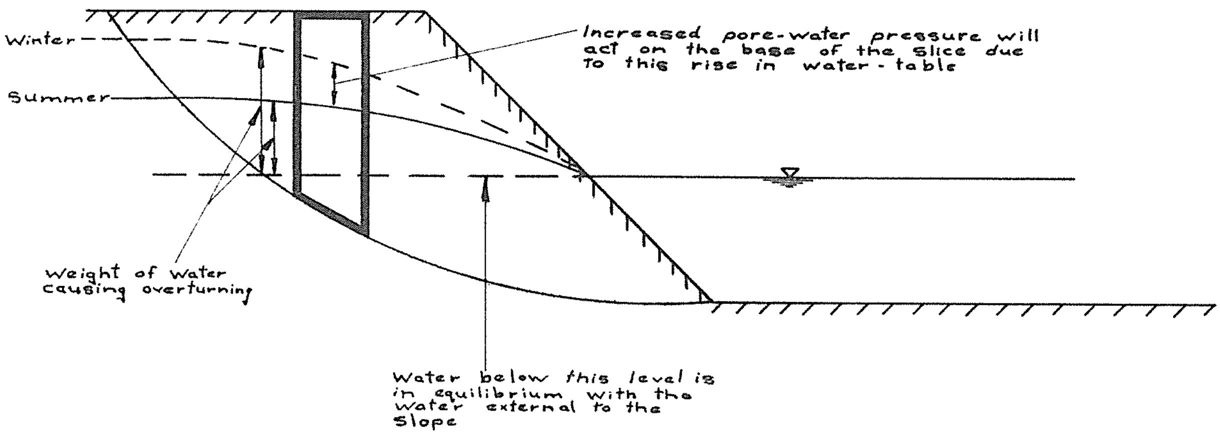

Fig 1: THE EFFECTS ON STABILITY OF A RISE IN GROUND WATER - TABLE.

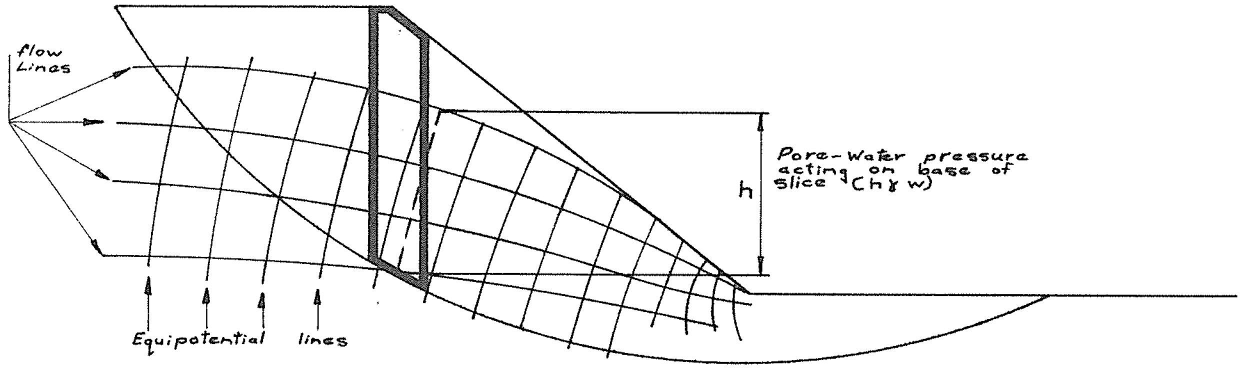

Fig 2: ESTIMATION OF PORE-WATER PRESSURE BY MEANS OF A FLOW NET.

flow net). It is established in this example that either method of analysis gives the same result.

6.2 Slopes in Clay

The analysis described above gives a result which is of important when using slip circle analysis by means of the method of slices, which is commonly used for the analysis of clay slopes. External forces on the sides of slices are difficult to allow for in the analysis (although the Bishop's simplified method can handle inter-slice forces where the resultant is horizontal). Hence it is preferable for analysis to take the first approach outlined in Section 6.1 and take the total weight of the slice with no external forces on the sides of the slice (rather than the buoyant weight of the slice plus a seepage force).

It should be noted that if there is an external water surface to the slope (as shown in Fig. 1) then below the level of this external water surface the buoyant weight of the slice should always be taken to calculate the overturning force used in the stability analysis.

6.3 Importance of the Flow Net

Where seepage forces are present it is important to draw a flow net in order to correctly predict the pore-water pressure acting on the failure surface. This is illustrated on Fig. 2. The pore-water pressure to be used to determine the effective stress acting on the base of a slice or element is not the vertical height of the ground water-table above the base of the slice, but the vertical height to the point where an equipotential line through the base of the slice intersects the ground water-table (or phreatic surface).

7.0 CONCLUSION

This paper has been concerned with the input data for stability analysis of natural slopes and of cut slopes in natural ground. The fact must always be borne in mind that no matter how refined and complicated the subsequent stability analysis may be, the end result cannot be any more accurate than the input data. Hence for meaningful results to be gained from a stability analysis, it is essential that every effort should be made to obtain input data that is sufficiently accurate for the requirements of the analysis, and also that the safety factor obtained should never be expressed more accurately than would be warranted from the reliability of the input data.

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ANALYSIS OF NATURAL EARTH SLOPE STABILITY UNDER STATIC LOADING

T.J. Kayes

1.0 INTRODUCTION

The purpose of this paper is to review methods of analysis of the stability of natural earth slopes which are available to the engineer. A brief historical review of the major developments in analysis methods will be given, followed by a discussion of the best methods currently available. Finally, limitations of the 'state of the art' of the present practice of analysis will be discussed including an indication of the overall cost of analytical stability assessment.

In the previous paper, Mr. J.P. Blakeley has reviewed the input data required for analysis; namely, definition of the slope geometry, evaluation of the distribution of the soils within the slope and the strength of the soils; measurement of piezometric levels and seepage flow; and he has outlined current practice for measuring these variables. The fundamental difference between "total" and "effective" stress has been explained. As the reliable evaluation of the stability of natural slopes can only be reached by an "effective" stress approach, only "effective" stress methods will be discussed in detail.

2.0 HISTORICAL REVIEW OF ANALYSIS METHODS

Failures of earth slopes, resulting in slips and landslides, have been occurring throughout geological history. Natural slopes, initially stable, have become unstable either because of a reduction in shear strength of the soil or because the slope profile has been altered, usually by the normal elements of erosion. Up until the nineteenth century man generally accepted the occasional occurrence of slips and landslides as part of nature, part of his environment. However, with the nineteenth century came the building of the first railways and the "trial and error" approach to the construction of cuttings and embankments became more costly, both in effort and in loss of life, as the more extensive works were matched by bigger failures. A demand for a method of slope stability analysis was created.

Collin (1846), made the first major contribution with an extensive study of 15 slips, from which he noted the cycloid shape of the slip surface. He proposed a method of analysis, based on the work on soil shear strength by Coulomb (1773). The next major advance was not until 1916 when Petterson proposed the so-called Swedish Circular Arc method of analysis. According to the concepts of the time, the clay was treated as a purely frictional material. A review of these developments is given by Petterson (1955).

The Swedish method, in terms of effective stresses, considers the equilibrium of a free body cut from the slope by an assumed circle of failure acted on by:-

(1) Gravity

(2) Pore water pressure acting on the failure arc

(3) Normal effective stresses acting on the failure arc

(4) Shear stresses acting along the failure arc

A study of the forces acting on the sliding body leads to the conclusion that the system is statically indeterminate as there are four unknowns and three equilibrium equations and an exact solution is impossible without consideration of the deformation properties of the soil. To overcome this indeterminacy, methods of solution based on the Swedish method all include a simplifying assumption to reduce the number of unknowns.

A further development of the Swedish method was advanced by the Swedish Geotechnical Commission

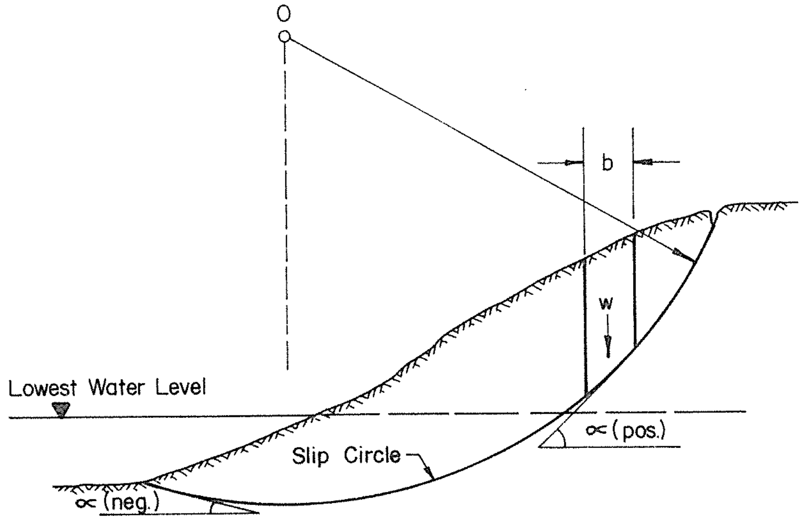


FIGURE I

BASIS OF SLICES METHODS USING CIRCULAR FAILURE ARCS

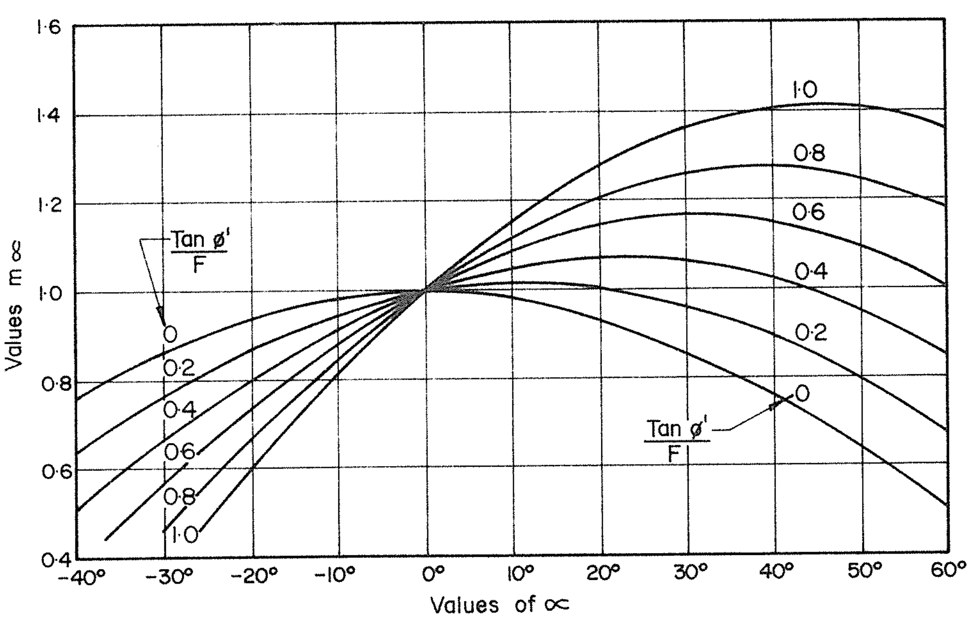


FIGURE 2

GRAPH FOR DETERMINATION OF m∝

and developed in detail by Fellenius (1927). In this method the free sliding body of the soil is divided into a number of vertical slices and the equilibrium of each slice is considered separately. This approach is extremely flexible as situations involving separate soil strata, varying pore pressure distribution, etc. can be handled. In order to achieve a solution to the statically indeterminate situation, forces acting on the sides of slices are neglected. This simplifying assumption can be thought of as implying that the resultants of all forces on the sides of slices act parallel to the bottom of the slice, as under these conditions the side forces do not affect the magnitude of the normal force on the slice base. For the situation defined as in Fig. 1 the factor of safety for the Fellenius method is given by the expression:

$$F = \frac{1}{\sum W.\ \sin\alpha} \sum \left[c'b.\ \sec\alpha + (W.\ \cos\alpha - ub\ \sec\alpha)\tan\ \phi'\right] \qquad - (1)$$

Where c' is the effective cohesion

ϕ' is the effective angle of shearing resistance

u is the pore pressure

The Fellenius method is still used widely. However, it has been shown (e.g. Whitman and Bailey, 1967) that the method may well under-estimate the factor of safety by over 20% and even up to 60% where failure surfaces are deep and pore pressures high. The fact that the method errs on the safe side is no justification for its continued use.

A method using the slices technique, was proposed by Bishop (1955), where the effect of side forces on slices was included. This method, known as the Bishop Rigorous method, is extremely lengthy in its application and is rarely used in practice. However, a modification of this method, suggested by Bishop, has found more widespread acceptance and is known as the Bishop Simplified method. The assumption made in the Simplified method is that the resultants of all forces on slice sides act horizontally, that is, the resultants have no vertical component. The error involved in this assumption has been shown by numerous examples to be negligible and thus it is still recommended as the best available simple method using circular failure arcs.

The factor of safety derived from the Bishop Simplified method is:-

$$F = \frac{1}{\sum W.\ \sin\alpha} \sum \left[(c'b + (W-ub)\ \tan\ \phi') \cdot \frac{1}{m_\alpha}\right] \qquad - (2)$$

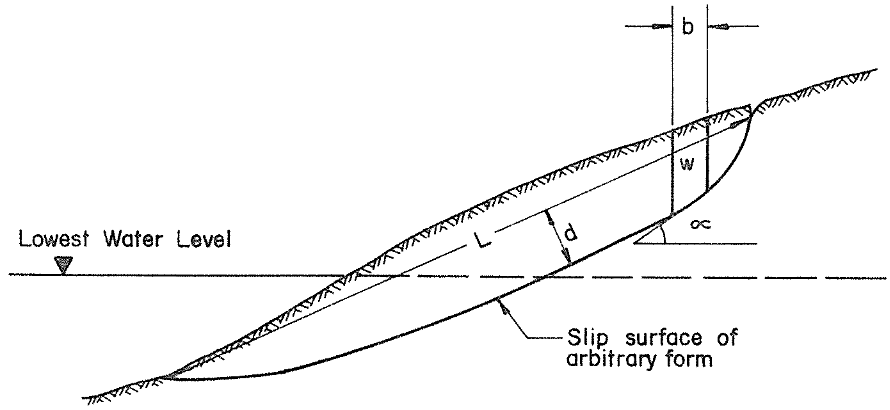
Where $m_\alpha = \cos\alpha\left(1 + \frac{\tan\alpha\ \tan\phi'}{F}\right)$

Considering a rearrangement of equation (2) to

$$F = \frac{\sum (c'b\ \sec\alpha + N\ \tan\ \phi')}{W.\ \sin\alpha} \qquad - (3)$$

$$\text{Where } N = \frac{W-ub - \frac{c'b\ \tan\alpha}{F}}{\cos\alpha + \frac{\tan\phi'\ \sin\alpha}{F}} = \text{the normal force on the slice base} \qquad - (4)$$

Both numerator and denominator of equation (4) can become zero or negative for low values of F, the former when pore pressures are high and the latter when α is high and negative. The second case is of greater practical importance but occurs so seldom as scarcely to affect the utility of the method. These conditions which create difficulties with the Bishop Simplified method also create difficulties



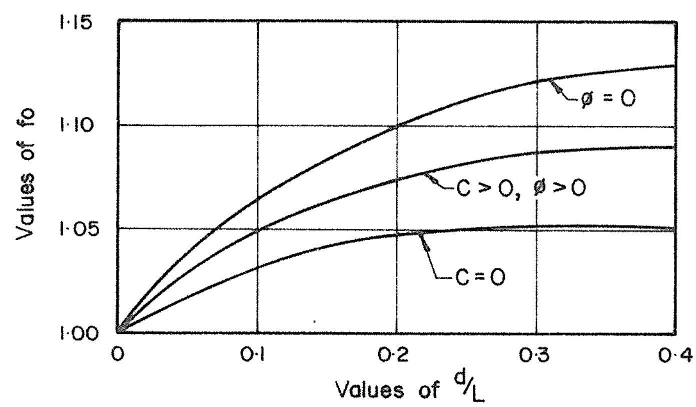


FIGURE 3

BASIS OF THE JANBU METHOD FOR NON-CIRCULAR SLIP SURFACES

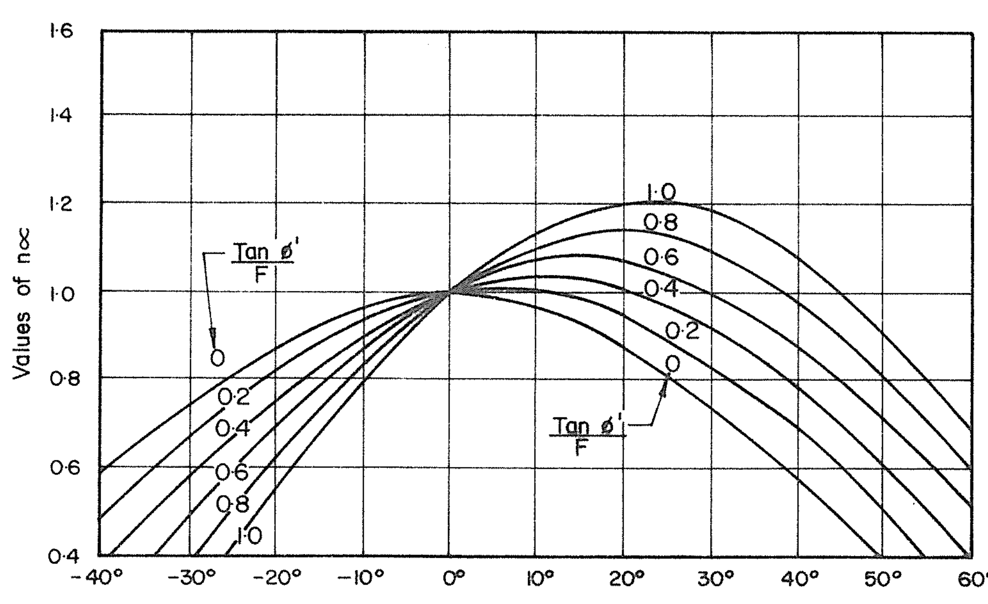


FIGURE 4

GRAPH FOR DETERMINATION OF n∝

with other methods based on limiting equilibrium mechanics, so that it is more a problem of the approach than the specific method.

With the advent of high speed digital computers, engineers have been interested in their use for carrying out slope stability analyses. Stability analyses are particularly well suited to the computer as there is much computation of a repetitive nature involved. The first computer application was made by Little and Price (1958) using the Bishop Simplified method. Since that time hundreds of programs have been written and no doubt some engineers reading this paper will have used some of them. A word of warning should be made which is applicable to the use of any computer program and that is, before using, examine the basic method on which the program is based and make sure the method is reliable.

Bishop and Morgenstern (1960) used the Little and Price computer program to produce stability coefficients for effective stress analyses similar to those presented by Taylor for total stress analyses. As with Taylor's curves these have limited application, but they can be useful in preliminary investigations when a detailed analysis would be impractical.

The logical advance on methods based on the Swedish Circular Arc system was the extension to non-circular or general slip surfaces. A useful method of analysing non-circular slip surfaces rapidly by hand was given by Janbu, et al (1956). This employs the method of slices and by resolving horizontally yields the expression:

$$F = fo \frac{\sum \frac{\left[c'b + (W-ub) \tan \phi'\right]}{n_\alpha}}{\sum W . \tan\alpha} \qquad - (5)$$

Where $n\alpha = \cos^2\alpha \left(1 + \frac{\tan\alpha \tan\phi'}{F}\right)$

fo is a correction factor depending on the shear parameters and the form of the slip, and takes account of the influence on the factor of safety of the vertical shear forces between the slices.

A more comprehensive method for non-circular slip surfaces was developed by Morgenstern and Price (1965). It employs the method of slices and satisfies all the boundary and equilibrium conditions. In order to make the problem statically determinate the assumption is made that the forces, E, on the sides of the slices are related by the expression:

$$\frac{X}{E} = \lambda . f(x)$$

where λ is a scale factor determined in the solution and f (x) is an arbitrary function concerning the distribution of the internal forces. For each solution it is necessary to examine the implied state of stress in the soil mass above the failure surface and to ensure, by suitable choice of the function $f(x)$, that this is physically admissible. As a result the difficulties of calculation are such as to require the use of a computer.

Research is continuing in the area of slope stability analysis and in the future methods, suficiently simple for routine use, may become available using the distribution of stresses or displacements throughout the slope. However, at present there is insufficient knowledge of in-situ stresses and stress-deformation-time properties of soils to make this approach practicable. As a result it is general practice to use limit equilibrium methods and recommendations as to the best methods are given later.

3.0 FACTOR OF SAFETY

There has been discussion in the past over the question of factor of safety. Discussion has centred on how it should be defined, how it should be determined and what its proper magnitude should be.

Basically, a factor of safety is a ratio that represents the results of computations made in accordance with some adopted definition. It is desirable that this ratio should reflect the influence of the most uncertain factors in the analysis so that the tolerance for errors can be seen at a glance.

A common basis for defining the factor of safety is with respect to the strength of the soil. Under conditions of equilibrium the mobilising shearing resistance in terms of effective stress is made up of cohesion and friction. These components of the resistance may not bear the same ratio of the ultimate values. Thus the shearing stress on a slip surface can be expressed as:-

$$S = \frac{C'}{Fc} + (\sigma - U) \frac{\tan \phi'}{F\phi} \qquad - (6)$$

where Fc and $F\phi$ are the factors of safety with respect to cohesion and friction respectively. For simplicity it is often assumed that Fc and $F\phi$ are equal and in this case the mobilised shear on the slip surface is given by:

$$S = \frac{C'}{Fc} + (\sigma - U) \frac{\tan \phi'}{F} \qquad - (7)$$

Where this definition is applied, the factor of safety is the same for all slices or elements along the failure surface. In addition, in the case of the Bishop Simplified method, this definition results in a unique distribution of normal and effective stresses.

Another common definition of factor of safety is applied with respect to the overturning moment as for example in the case of the Fellenius method. In this method, the factor of safety for all elements is not necessarily the same, the value of F obtained merely reflecting the average value.

Although these definitions produce the correct result ($F = 1.0$) for slopes in critical equilibrium, the results are, in general, different when the slope is stable. The main reasons for differences in factor of safety definitions are related to differences in the assumed distribution of stress along the potential failure surface, and to differences in the way failure is supposed to be reached.

In practice, the definition of factor of safety adopted is probably not of major importance as engineers tend to become accustomed to operating within a range of values which experience has shown to be satisfactory for the particular procedure used. (For example for natural slopes analysed by the Bishop Simplified method a factor of safety between 1.3 and 1.5 would be reasonable, depending upon the conditions). However, it must be remembered that the factor of safety of a slope has a certain real value and the stability of a slope cannot be changed simply by using a different analysis method which is based on a different factor of safety definition.

4.0 RECOMMENDED ANALYSIS METHODS

In this section references are given to what are considered the best methods of analysis of natural earth slopes currently available to the practicing engineer. As discussed previously the methods use "effective" stresses and are based on limiting equilibrium mechanics.

4.1 Circular Failure Arcs: If the slope geometry and soil profile suggest that failure along a circular failure arc would be a plausible mechanism then the following methods are applicable:

(a) If the slope geometry is simple and the soil homogeneous then a rapid indication of stability is given by charts produced by either Bishop and Morgenstern (1960) or Spencer (1967). The former charts indicate the factor of safety for a given slope angle, while the latter indicate the slope angle for a given factor of safety.

(b) Complex slope geometry, soil variations and external loading can be analysed using the Bishop Simplified method. As discussed previously, this is an accurate method and is recommended as the best method for circular failure arcs. The original reference is Bishop (1955).

For the general situation defined in Figure 1 the factor of safety is given by equation 2. As the factor of safety appears on both sides of the equation, one must first estimate the value of F and then correct the calculation until the estimated and calculated values of F agree. The convergence is very rapid. Values of m_α can be taken from Figure 2. The calculated factor of safety decreases with increasing number of slices up to about 10 - 15 slices beyond which little variation in F occurs. As a result it is recommended that no less than approximately 10 slices should be used. If an external water level exists on the slope then its lowest likely level should be used in the analysis. The pore pressure, u, in equation 2 is then the excess pore pressure above the low water level and in calculating the weight of each slice the weight of the part beneath low water level is reduced for buoyancy. A number of trial slip circles should be analysed for a given slope in order to get a "feel" for the critical slip circle location and its corresponding factor of safety.

The time required for an analysis, by hand, using the Bishop Simplified method is several hours with an experienced user. If a large number of slope profiles or variations in soil strength or piezometric profile are required to be considered then the use of a computer program (e.g. Kayes (1968)) is recommended.

4.2 Non-Circular Failure Arcs: In cases of well-developed stratification or where varying attitude and position of slip surfaces are predetermined, for example, by the position of bedrock, it is frequently impossible to follow a soft layer or other weak boundaries with a circular slip surface. In such a situation, use of the Janbu (1956) method is recommended. The likely location of the slip surface is drawn on a section of the slope and for a surface of arbitrary form (see Figure 3), the factor of safety is given by equation 5. Values of the correction factor, fo, are given in Figure 3, while values of n_α can be taken from Figure 4. The Janbu method can be carried out by hand in about the same time as the Bishop Simplified method.

4.3 Planar Slides: If the likely failure surface within a slope can be approximated by a planar surface of infinite extent, (i.e. constant slope angle, slip surface parallel to slope, ground water conditions same at all points) then the factor of safety is given by:

$$F = \frac{c' + (\gamma . z \cos^2 \alpha - u) \tan \phi'}{\gamma . z . \sin\alpha \cos\alpha} \qquad - (8)$$

Where α = angle of slope and failure surface to the horizontal

z = depth from ground level to the failure surface

For the case of horizontal seepage flow from the failure surface:

$$u = \gamma_w . z$$

or for the more common case of flow parallel to the slope and the groundwater table at a vertical height mz above the failure surface:

$$u = \gamma_w \; m.z. \; \cos^2 \alpha$$

4.4 Wedge Mechanisms: As an alternative to the Janbu method, in cases where the potential failure surface can be approximated closely by two or three straight lines, then the factor of safety can be obtained by a wedge method. In this method the potential failure mass is broken up into two or three wedges and the shear resistance along the several segments of the failure surface is expressed in terms of the applicable strength parameters and a factor of safety, F, which is the same for all slices. Readers are referred to soil mechanics text books (e.g. Lambe and Whitman (1969)) or Seed and Sultan (1967), for examples of the method.

5.0 OVERCONSOLIDATED CLAYS

Typical stress-strain curves for clays show a peak shear strength followed by a decrease in shear resistance with increasing displacements. Soft silty clays show little difference between peak and residual strengths, however, stiff (overconsolidated) clays show a marked decrease in strength from peak to residual. For a potential failure surface in a slope in overconsolidated clay, the shear stresses and normal stresses will vary along the failure surface. Hence, if the loading on a stable slope is altered (say by erosion or excavation) so that the shear stresses increase, then the strength will be exceeded at some points before others. Only for soils which act perfectly plastically at post-peak strains could a state of limiting equilibrium develop in which the peak strength is mobilised all along a potential failure surface.

The fact that stiff overconsolidated clays show a marked difference between peak and residual strengths means that for those clays, stability analyses based on measured peak strengths and using a limiting equilibrium analysis method will not indicate the true long term factor of safety. This is demonstrated by the case histories of slope failures presented by Skempton and Hutchinson (1969). The processes causing the gradual reduction in strength within a slope to the residual value are complex and are termed "progressive failure". The mechanism of progressive failure is the subject of considerable research effort and is outside the scope of this paper.

For the assessment of the stability of slopes in stiff fissured clays the residual strength parameters should be used if failure is possible along an old slip surface. If slips have not previously taken place in the area then the appropriate shear strength parameters will be somewhere between peak and residual values. A number of recommendations for establishing design strengths between the peak and residual values have been proposed, (e.g. Skempton (1970) recommended using the "fully softened" effective strength parameters), but all can be criticised on some count.

6.0 COSTS OF INVESTIGATIONS AND ANALYSIS

The analysis of the stability of natural slopes is an expensive process, primarily because testing and computation must be carried out in terms of effective stresses. Let us look at what is involved in a typical case which could be, for example, a proposed subdivision where the stability has been questioned:-

Let us assume that the investigation is to be carried out as a two-stage process. The first stage, involving drilling of say 4 - 5 boreholes, supervision and logging of core by an experienced technician, installation of piezometers or standpipes, an inspection by a geologist and some simple laboratory testing, would cost of the order of $2,000 at 1974 rates. The second stage might involve 3 boreholes with recovery of thin-walled tube samples from predetermined soil zones, followed in the laboratory by 3 or 4 sets of sophisticated triaxial tests to measure effective strength parameters. The analysis computations will require several days' work by an experienced engineer in order to ensure that sufficient trial slip surfaces have been considered. The use of a computer will probably increase the number of trials and hence the order of accuracy, but the overall time and cost of analysis generally will not be affected significantly. The second stage of the investigation, including analysis, might cost in the order of $1,500 to $2,000.

The total cost of the exercise, including the two-stage site investigation, laboratory testing and analysis could be of the order of $4,000 at 1974 costs. More complex studies would involve higher costs but it would be difficult to do justice to a relatively small problem for less than $3,000.

7.0 LIMITATIONS OF ANALYSIS METHODS IN NATURAL GROUND

It must be emphasised from the outset that the greatest uncertainties in stability problems arise in the selection of the pore pressure and the strength parameters. An elaborate mathematical treatment of the analysis can lead to a ficticious impression of accuracy and the engineer can lose sight of the real uncertainties in the problem. In natural ground the usual variability of the soils within a slope raises the first problem. For analysis an often complex lithological profile involving variable weathering must be rationalised to a simplified slope cross-section which hopefully reflects the true nature of the soils. Usually a slope cross-section for analysis must be based on 2 or 3 boreholes.

The second problem concerns the soil strength parameters used, which have suffered from sampling disturbance, stress-release, and usually are all tested vertically and do not reflect any in-situ strength anisotropy. The soil sampling and testing methods can be a fundamental source of error in an analysis. Even if the "peak" strengths have been assessed accurately, with stiff fissured clays, as discussed, the problem arises as to whereabouts between peak and residual strengths is it realistic to assume a strength for analysis purposes.

The third and vitally important variable is what to assume for operative pore pressures. A piezometric profile measured during drilling is unlikely to represent the worst conditions. We all know that generally it is when we have a period of prolonged heavy rain that the failures occur, so we must input a worst likely piezometric profile to account for these circumstances.

The following example serves to illustrate the significant variation in factor of safety which results from a relatively small variation in input parameters. (The slope was analysed by computer using the Bishop Simplified method). The results are summarised in Table 1:-

TABLE 1: SUMMARY OF ANALYSES WITH INPUT VARIABLES ALTERED

Input Variables for Analysis	Factor of Safety
Laboratory 1 data	0.977
Laboratory 2 data (no triaxial test corrections)	1.318
Laboratory 2 data (including triaxial test corrections)	1.207
(c' + 1 psi) (ϕ' + 0°)	1.382
(c' - 1 psi) (ϕ + 0°)	1.029
(c' + 0 psi) (ϕ' + 1°)	1.223
(c' + 0 psi) (ϕ' - 1°)	1.191
(c' + 1 psi) (ϕ' + 1°)	1.401
(c' - 1 psi) (ϕ' - 1°)	1.013
(γ - 1 pcf)	1.211
(γ + 1 pcf)	1.204
Parallel flowing pore water	1.334

(a) A surprising variation in F (i.e. .977 vs 1.207) results from testing samples in two different laboratories. Both laboratories produced comparable ϕ' values but the c' values tended to be higher in the second laboratory.

(b) The effect of the correction for filter paper drains in the triaxial test is also surprisingly large,

i.e. with correction F = 1.207

without correction F = 1.318

(c) The next six analyses summarised are for cases where, using data from the second laboratory, the c' values were raised or lowered by 1 psi and ϕ' was altered by $\pm 1^{\circ}$. (This is the order of magnitude of the variation which could arise from normal experimental error). The marked influence of effective cohesion compared with the far smaller influence of the effective angle of internal friction is apparent.

(d) The effect of the small variation in the soil density values is shown to have little effect on the computed factor of safety.

(e) Finally, the marked influence of the pore pressure distribution assumption is demonstrated. A conservative assumption that groundwater flow was horizontal resulted in F = 1.207. Without altering the assumed groundwater level but simply assuming the pore water flow to be parallel to the slope rather than horizontal raised F to 1.334. The actual pore pressure distribution under field conditions will probably be somewhere between these two cases.

In conclusion, I suggest that an assessment within a geological framework is a fundamental element in analysing the stability of natural slopes. Engineering assessment with mathematic analysis should follow, but the geological approach cannot be omitted. Geomorphologists and experts in air-photo interpretation have pointed out the significance of old landslide areas and have gone as far as to suggest that, if there are no old landslides in an area, it is unlikely that a moderate construction operation will start a new one. On the other hand, if old landslides abound, it is quite likely that even minor construction operations will lead to sliding. An over-emphasis on the analysis of sliding movements can lead us to overlook the practical significance of these deductions.

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STABILITY OF NATURAL SOIL SLOPES DURING EARTHQUAKES

P. W. Taylor

1.0 INTRODUCTION

The assessment of the stability of natural slopes is difficult for static conditions. To assess stability of such slopes during earthquakes is even more difficult. Analytical methods now available show great promise, particularly for earth dams where materials have been placed in a specified manner under controlled conditions. For natural slopes, however, the vagaries of nature can produce minor geological features, easily missed during a site investigation, which may exert a controlling influence on stability during earthquakes. Detailed studies of slope failures in recent earthquakes have led to a much better understanding of the mechanisms involved. With careful site investigation it is now possible to recognise slopes of doubtful stability and, it the circumstances justify it, a meaningful analysis can be made.

2.0 Classification of Observed Failures (after Seed (Ref.1.))

The following grouping of past failures, based on soil type and failure mechanism is useful:

(i) Flow slides caused by liquefaction of cohesionless soils.

(ii) Slides caused by liquefaction of thin seams or lenses of sand.

(iii) Slides in clay deposits facilitated by liquefaction of sand lenses.

(iv) Shallow slides in cohesionless slopes.

(v) Slides in cohesive soils

(vi) Slumping of fills on good foundation materials.

(vii) Collapse and cracking of fills on poor soil foundations.

As this Symposium is concerned only with natural slopes, types (vi) and (vii) need not be considered further. Of the remaining five types, three are related to the phenomenon of liquefaction. There are remarkably few examples of slides in cohesive soils, unassociated with liquefaction. Seed (Ref.1.) lists only the failure of the banks of the All-American Canal during the El Centro earthquake of 1940.

3.0 Methods of Analysis for Slope Stability during Earthquakes

Most stability problems are analysed using a "limiting equilibrium" approach; that is, the forces or moments which can be resisted by the structure (lope) are assessed and compared with the forces or moments actually applied under the most unfavourable loading conditions. This ratio gives the "factor of safety." If this is less than unity, failure is predicted.

This method has been traditionally applied to slope stability during earthquakes. The effect of the earthquake is assumed to be a horizontal acceleration applied uniformly on the potential failure zone. Soil strength is taken as that determined from conventional tests and, provided that the calculated factor of safety is somewhat greater than 1.0 the design is assumed to be satisfactory.

This is termed the 'pseudostatic' method.

3.1 Selection of Seimic Coefficients

If the slope behaved as a rigid body, the maximum acceleration experienced would be the maximum ground acceleration during the earthquake. Earth dams and slopes, however, behave in a dynamic manner, with higher accelerations near the crest. This has been clearly demonstrated by earthquake records on dams and the limited theoretical work that has been carried out suggests that this happens to some extent on natural slopes also. This is discussed in more detail in Reference 2.

3.2 Significance of the Factor of Safety

With continuously applied (static) loads, it is clear that a factor of safety less than unity indicates failure. With earthquake loading, however, the factor of safety may momentarily fall below unity without serious consequence. Assuming the material to be elastoplastic, some permanent deformation may occur but provided it is not excessive, this may be considered permissible during the infrequent occurrence of a large earthquake.

The idea is illustrated in Figure 1(a). The concept was introduced by Newmark (Ref.3.) and was later applied to cohesionless slopes (Ref.4.) to predict failures of type (iv).

If, however, the soil strength decreases during the disturbance, successive earthquake pulses may be accompanied by a rapid reduction in the factor of safety which may lead to failure as illustrated in Figure 1(b).

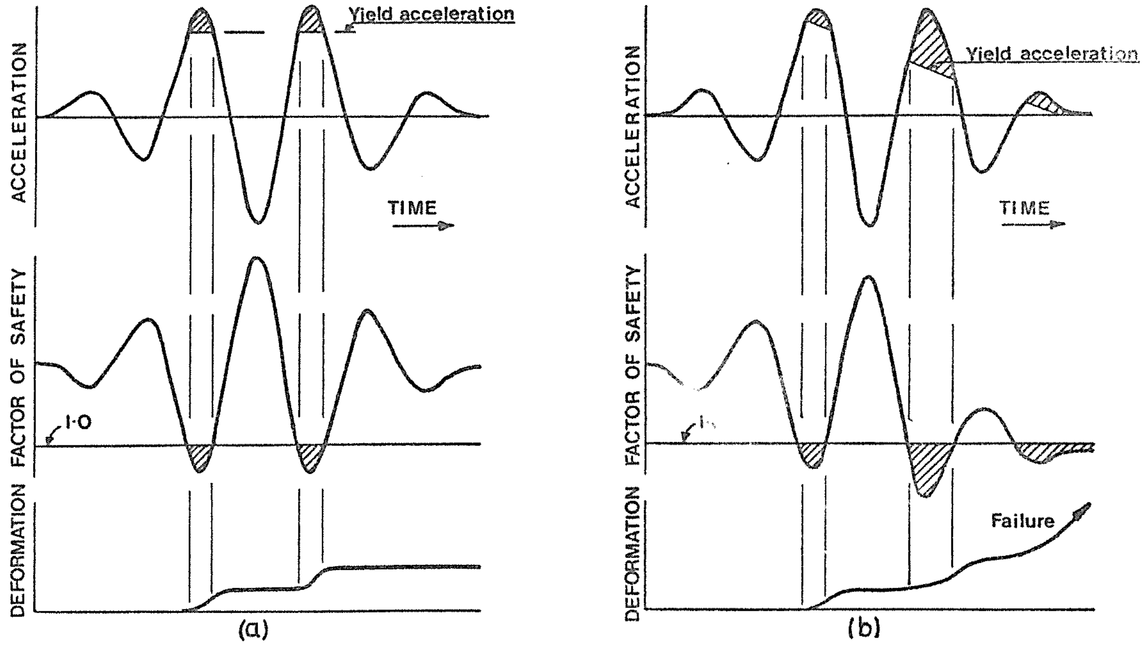

FIGURE 1. ACCELERATION, FACTOR OF SAFETY AND DEFORMATION OF EARTH SLOPES DURING EARTHQUAKES

(a) Insensitive cohesive soils
(b) Soils whose strength deteriorates during dynamic loading.

3.3 Soil Strength during Earthquakes

The assumption that soil strength under dynamic loading is the same as measured in conventional tests is reasonably satisfactory for insensitive cohesive soils (Ref.5.). This assumption is, however, far from acceptable for loose sands, sensitive clays and some silts which develop high pore pressures when loaded dynamically under undrained conditions. Such pore pressures result in a substantial reduction of strength after a number of loading cycles and in some cases in complete liquefaction.

Clearly, any satisfactory method of analysis must be based on dynamic tests of the soils, wherein the deformation or failure of the soil is related to the static and dynamic stresses applied, and to the number of cycles of loading. Fortunately, it is found that the frequency of dynamic loading is not a significant variable.

4.0 The Ideal Method of Analysis

Given the time history of the earthquake, dynamic analysis of the slope would give a time history of the stresses at many points within the slope. Undisturbed soil samples could then be subjected to similar stresses and their deformations determined. Ideally it might then be possible to determine the deformations of the slope as a result of the earthquake.

Such a method is still far from realization. There is, as yet no satisfactory method of relating sample deformation to slope deformation. The stress changes applied in laboratory tests only approximate those in the slope. As with many other 'ideal' situations encountered in civil engineering it is something that can be aimed at even though its realization may never by achieved. Moreover, such a method would require a very considerable amount of work in the field, in the laboratory and in analysis and would be economically feasible only in a few cases.

5.0 Simplified Approaches

5.1 Methods Assuming a Circular Failure Surface.

A method presented by Seed (ref.6.) attempts to approach the ideal, even though the obstacles mentioned above are not overcome. Still a vast amount of dynamic testing is involved and the method is not likely to be widely applied.

In 1971 a method was presented by Martin and Taylor (Ref.7.) in which the extent of detailed analysis and the number of dynamic tests are considerably reduced. Briefly, some suggestions for suitable design earthquakes appropriate to New Zealand conditions are made, then stresses within the slope can be assessed assuming a circular failure surface. Suitable static and dynamic stresses can then be determined for application in dynamic triaxial tests. The appropriate number of cycles of loading can then be applied in 'proof'tests. If the samples withstand these dynamic tests without excessive deformation, the slope is expected to remain stable in an earthquake of the intensity assumed in design.

This method has been applied to motorway cuttings in Auckland. The tests were carried out on the stress-controlled dynamic triaxial testing facility at the University of Auckland. In this device (described in Ref.8) a sinusoidally-varying axial deviator stress is superposed on a static deviator stress to simulate the dynamic loading occurring in a slope during an earthquake.

5.2 Methods Assuming a Non-Circular Failure Surface.

If there are layers or lenses of saturated sand underlying cohesive soils, the assumption of a circular failure surface is inappropriate. The probable mode of failure is the almost horizontal movement of a massive block, sliding over a liquefied sand layer, followed by the formation of a graben (as in the L-Street slide at Anchorage) or by the disintegration of the block (as at Turnagain Heights). Such analytical techniques are described in Ref.9.

6.0 Observations of Slope Failures in Earthquakes

The Alaska earthquake of 1964 provided some of the most spectacular examples of landslides in earthquakes. Liquefaction of cohesionless soils caused flow slides at Seward, Valdez and at Kenai Lake (Ref.9.).

Several large landslides occurred in Anchorage, in the same earthquake. In the L-street slide, a large block moved bodily about 3m towards the coast on a slope of only 1 in 25 while a smaller block behind the first slipped downward about 3m, forming a graben.Subsequent analysis showed that this was caused by the liquefaction of thin seams of sand. A similar explanation was given for the slides at 4th Avenue and Government Hill. This Alaska earthquake was of unusually long duration, strong shaking continuing for at least two minutes. Had the earthquake been of shorter duration liquefaction may not have occurred with the smaller number of cycles of dynamic loading.

The landslide at Turnagain Heights in which over seventy houses were destroyed, occurred in deposits of sensitive Bootlegger clay (sensitivity 5-30) but here also liquefaction of lenses of sand played a major role in causing instability. Comparison of dynamic tests on sensitive clays and on loose sands shows that the reduction in strength after cyclic loading of the clays is fairly small, while, in the sands, liquefaction implies a complete loss of strength.

In examining slopes for potential instability during earthquakes, apparently minor geological features such as thin layers or lenses of sands must not be overlooked.

6.1 Stabilization

Even if a natural slope is recognised as being potentially unstable, any method of stabilization is usually beyond the bounds of economic feasibility, unless the problem is on a fairly small scale. The information may be of use at the planning stage and capital-intensive development kept clear of unstable areas.

7.0 Conclusion

During the past decade there have been considerable advances in knowledge of stability of slopes during earthquakes. Much has been learned from the detailed study of failures. Combined with research on the properties of soils under dynamic loading, a much deeper understanding of the mechanisms of failure has been attained. With this knowledge it is now possible to recognise conditions of potential danger in earthquakes. The analytical approaches currently available are greatly superior to earlier methods in which a seismic coefficient was incorporated in a static analysis,but for economic reasons their use is likely to be restricted.

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THE USE OF INSTRUMENTATION IN EVALUATING THE STABILITY OF NATURAL SLOPES

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1.0 INTRODUCTION

Frequently, in assessing the stability of an earth slope there arise the questions;

"Has movement occurred in the past, is it occurring now, and, will it occur in the future?", and if so;

"What are the magnitudes of movement?"

From a visual observation of the condition of a particular slope it could be suggested that movements of say 20cm have at some time in the past, occurred; but when? If they appear to have now terminated, can we be sure that the slope is still not slowly creeping? Furthermore, if the slope does appear to be moving, is the rate of movement increasing or decreasing?

In the appraisal of the stability of any soil or rock slope, each of these questions must be satisfactorily answered. Such answers can only be achieved if slope movements are systematically measured using equipment which is well designed, properly installed and the resulting data evaluated to produce meaningful conclusions.

The importance of the selection and installation of proper field instrumentation, together with the planning of continuous collection and assimilation of meaningful field data, cannot be too strongly stressed. If insufficient thought is given to the location of instrumentation and the corresponding monitoring program, critical movements of ground and hydrostatic pressures may go undetected. Furthermore, weak zones or minor geologic defects within an outwardly appearing stable hillside may go undetected during conventional subsurface investigations, often only becoming apparent through the subsequent use of instrumentation.

It is the purpose of this paper to review the various instruments and techniques currently available for the monitoring of slope movement and groundwater levels.

2.0 PLANNING OF THE INSTALLATION

In the planning of the installation of instrumentation, adequate allowance must be made in providing sufficient funds to cover the complete monitoring program, which may often extend over several years. Where unexpected movements occur, a considerable amount of time and effort can be spent in locating the cause and determining suitable techniques for halting the movements. The corresponding increase in frequency of readings will rapidly increase project costs. Thus, frequently, one of the main criteria governing the scope of any evaluation of a slope's performance is the question of cost.

The time available for the study is another criterion which will influence the scope of a monitoring program. A request for an immediate assessment of the hillside's stability will often preclude a full and adequate monitoring program. In this situation, monitoring of the slope over a short time interval will be of little value and in fact may lead to a false sense of security, particularly if such monitoring is restricted to less critical times of the year. In any event, monitoring programs of less than a year's duration are generally not recommended, except for where their purpose is to actually locate a plane of failure.

2.1 General Requirements

The extent and magnitude of a monitoring system will usually be governed by the size of the slope under consideration, the value of the structures in the immediate vicinity of the slope, and the consequences of a slope failure occurring.

In any monitoring system there are several basic requirements for an adequate program and they are as follows:

Firstly, the instruments used should be reliable, in that they must be capable of producing consistent results over extended periods of time. It may be more desirable to utilize an instrument of slightly less accuracy or sensitivity but which has a proven record of operating consistently under unfavourable conditions, rather than use a highly sophisticated instrument of unproven capabilities. The sensitivity of the instruments must, however, exceed by a considerable margin, those movements of ground surface of soil strata which are considered as being critical to the stability of the slope. In many cases it is the change in movements rather than the absolute movements which give a true picture of the events which are occurring.

Secondly, the components of the system, whether they be survey bench markers, slope marker stakes, inclinometer casing or even piezometer standpipe cover plates, must be sufficiently stable to render the observed movements of slope or groundwater to be entirely representative of those of the slope as a whole and not as an isolated part.

Thirdly, the instruments, markers and piezometers must be carefully located in the field bearing in mind all known geologic surface and subsurface information about the site. Ground adjacent to the area under study must be carefully reviewed and specific areas located which are adjudged to be sufficiently stable to provide an adequate base to which all slope movements can be referred. Piezometers must be located so as to provide accurate information on levels of groundwater within and around critical areas.

Finally, and possibly the most important requirement of all is that a schedule for reading the instruments must be prepared and conscientiously carried out. Evaluation of the results of each survey must be undertaken as soon as possible after the measurements are taken. Recordings of data are of little use if the slope happens to fail while the latest field record sheets await analysis at the bottom of the instrumentation engineer's in-basket.

2.2 Installation Personnel and Equipment

An important task in planning an instrumentation programme is the selection of field personnel and the installation of equipment. Whereas the approximate position of piezometer holes, inclinometer holes and survey markers may be planned in the office, usually with the assistance of aerial photographs, the ultimate location of these devices must be determined in the field. Therefore it is imperitive that a geologist or engineer, completely familiar with all aspects of the site, is present on site during this important phase of the project. Drilling equipment must also be completely capable of recovering all information relevant to the proposed installation. All boreholes put down, even those drilled for the sole purpose of installing piezometers, must be logged and viewed in the light of providing a continuing understanding of the geological characteristics of the slope under study.

3.0 INSTRUMENTATION

Instrumentation for the monitoring of natural earth slopes may be divided into three distinct groups: Those which measure surface movements, subsurface movements and groundwater levels. While there are a considerable number of instruments available on the market today for use in each of these three areas, only a limited number are suitable for use within natural slopes, the remaining being more applicable for use in earth and rockfill embankments. Furthermore, the cost of the individual instrument becomes of greater importance in the monitoring of natural slopes, since the overall capital cost of any structure to be constructed or protected is usually much less than for an earth or rockfill embankment and therefore the percentage of total cost devoted to instrumentation is correspondingly greater. In some instances, the cost of recording and plotting the data may greatly exceed the combined initial cost of the instrumentation and its installation. Therefore the ease in the recording and reducing of data is often an important criterion in the selection of the instrument to be used.

3.1 Surface Movements

Above-ground monitoring of slope movements are usually undertaken with the assistance of conventional surveying equipment, with the selection of equipment being governed by 1) its availability, and 2) the magnitude of expected movements. Several techniques are available, such as triangulation, offset measurement and downslope measurement.

Triangulation surveys for earth deformation measurements undertaken in New Zealand reportedly (Jenks 1968) have attained accuracies in the order of 1:50,000, i.e. about ± 3 mm over a 150 m line. This survey was undertaken in conjunction with the construction of Benmore Dam and thus a high degree of control was achieved. Theodolites were set on fixed pedestals with steel centering pins used to constrain the theodolites to ensure exact centering at all times. In most cases, however, where conventional survey equipment is used, an accuracy better than 1:10,000 (i.e. ± 10 mm over a 100 m line) may be difficult to achieve.

It must be stressed that the results of any deformation survey are no better than the quality of the control, which in turn depends on the stability of the control points. Local surface conditions which are not directly related to the overall slope stability may cause movements of improperly installed slope marker pegs, which in turn may result in incorrect conclusions regarding the slope's performance to be reached. Vulnerability of the control points and slope markers to damage is another problem which may effect the overall accuracy of deformation measurements. The following are some of the techniques commonly used in assessing the surface deformations of earth slopes.

3.1.1. Lateral Movements from Offsets: In this technique, a permanent line of sight is established lying approximately perpendicular to the expected ground movements. At each end of the line, a control point is established consisting of either a concrete monument or a section of steel pipe or rod driven into the ground with its top located a short distance below ground level and covered with a removable protective cover plate. A theodolite is set over this mark, and sighted onto the second control point located across the zone of slope movement. Intermediate pegs are then located approximately on-line across the slope face. Deformation measurements are taken by sighting directly onto a rule which is in turn set against the peg and aligned perpendicular to the line of sight. Where the sight distance exceeds about 20 m, a target is attached to the rule in order to present a clearer target. Typical details are sho n Fig 1.

One of the problems normally associated with this method is to find an area of stable ground close to the slope which is suitable for locating the permanent control points. In the future, as laser equipment becomes more readily available and sight lines are able to be increased, the selection of control points will become easier. At present, where conventional equipment is being used, additional control points are often set on line some distance beyond the instrument control point and used for backsight checks.

3.1.2 Triangulation: Where difficulty is experienced in finding suitable control points immediately adjacent to the slope, it may be necessary to locate two control points off the slope and establish an accurate base line between them. Horizontal bearings measured with a well adjusted 1 second theodolite to markers set on the slope, are then recorded. Gould and Dunnicliff (1971) report that slope movements of ± 10 mm can be measured with a base line chained to 1/10,000, theodolite closure to 10 seconds and sight distances less than 150 m.

Recently developed electronic distance measuring equipment used in conjunction with a 1 second theodolite, can considerably minimize the amount of field and office time required for each set of observations. With this equipment, which is rapidly becoming common place among surveying firms, a modulated infra-red or visible light beam is directed to a portable reflector target set over the slope markers. The reflected beam is electronically compared with the beam transmitted by the instrument, and almost immediately the distance between instrument and target is presented as a direct visual display read-out. Accuracies attainable with this equipment are typically in the order of ± 5 mm over sight distances up to about 1000 m.

3.1.3 Down Slope Measurement: One of the simplest ways in which slope movements can be detected is by taping directly downslope between targets placed on the slope and a control point placed either above or below the area of anticipated slope movements.

The conventional survey band (chain), draped in cantenary between target and control point, enables measurements to an accuracy down to ± 2 mm to be made, providing corrections for tension, sag and temperature are applied. Over relatively long distances and where interference from vegetation occurs, a wider range of accuracy would be expected.

Frequently, when the scope of the investigation is restricted by cost factors, a situation may arise where the owner of the property concerned is able to take his own measurements of down-slope movements. A 30 m steel builder's tape would be used in place of the survey chain, with which accuracies in the order of ± 10 mm could reasonably be expected. Where a simple tape tensioning arrangement and temperature corrections are applied, the accuracy range for change in distance measurement could be lowered to about ± 5 mm under normal conditions.

Electronic distance measuring devices, with or without a base theodolite, could also be used in place of the survey band. However, for short lines, the survey band is generally preferred.

3.1.4 Tensioned Wires: In locations where animal movements are restricted, one or more wires may be stretched across or down the slope and left in place. A permanent spring is attached to the wire and a mark on the wire is compared with a rule which is also permanently set in the ground adjacent to the wire. For protection, the wires may be set within tubes buried a short distance below ground level. Wilson and Squier (1969) describe an inexpensive device of this type, details of which are given in Fig 2.

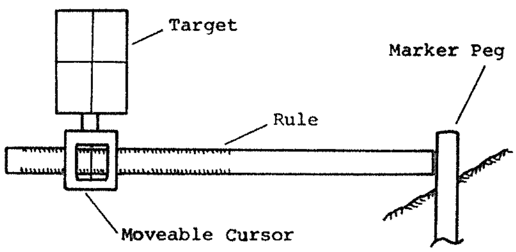


Fig 1. Cursor Target Gauge for Offset Measurement of Surface Movements

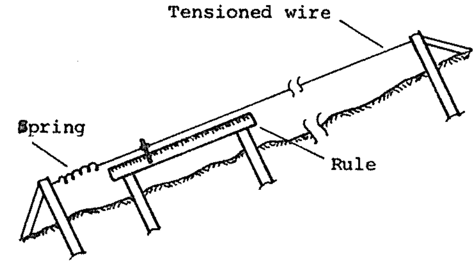


Fig 2. Inexpensive Tension Wire Device for Measuring Downslope Movement

These devices may also incorporate circuits to give automatic warning in the event movement exceeds a preset amount. Thus they are often used to warn of slope failures within isolated railway cuttings.

3.2 Subsurface Movements

In the investigation of any major slope instability, recognition of the location and depth to the surface of movement is of paramount importance. If remedial works are proposed, such as a rockfill buttress, an ultra-conservative design can be avoided, and considerable savings likely to be achieved if the total quantity of earth in potential motion can be accurately estimated. Instruments used in determining the extent of these mobile masses include the inclinometer and the extensometer.

3.2.1 The Inclinometer: The inclinometer (or slope indicator) is probably the most common instrument for detecting subsurface horizontal movement and has been used extensively both overseas and to a limited extent in New Zealand, in monitoring movements of natural slopes and earth and rockfill embankments.

The device consists basically of a pendulum activated transducer enclosed in a watertight torpedo which is lowered down a special casing, installed in a pre-drilled borehole. Horizontal movements of the soil deflect the casing and the instrument is able to measure the corresponding change in inclination of the casing. The bottom of the casing is generally assumed to be fixed so that the actual shape of the deflected subsurface profile is obtained by integrating the observed inclinations of the casing.

Several inclinometer models are currently being produced, with major differences being in the type of transducer and the shape of casing used. The most common type uses a resistance coil transducer in conjunction with a Wheatstone Bridge circuit, located in a control box at the top of the hole. Vibrating wire transducers (strain meters) are also used with moderate sucess. For work of higher precision, inclinometers containing null-movement servo-accelerometers have been developed. Automatic punch or magnetic tape read-outs are a recent modification which enable data to be fed directly into a computer for data processing.

Care must be taken with the installation of the specially extruded casing to ensure that the annulus between the borehole wall and casing is completely filled with either a sand or weak grout backfill.

Absolute movements of the surface of the slope can usually be obtained with the instrument, although where there is doubt as to whether the casing extends below the zone of possible movement, additional ground control must be employed in order to accurately measure the magnitude of total deformations at the top of the casing.

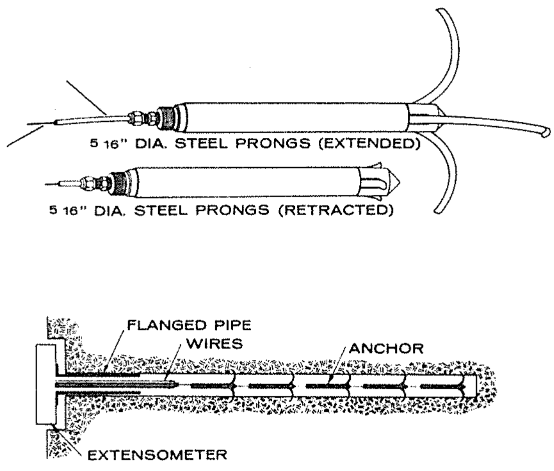
The accuracy of the inclinometer is effected by several factors, (Gould and Dunnicliff, 1971), such as sticking of the pendulum bearings etc, but most important is the casing installation itself and the care with which each set of readings is taken to ensure that the exact depth of former readings is repeated.

Accuracies in the order of $\pm 1 \times 10^{-4}$ radians have been reported using the more precise inclinometer models, (Lambe 1968), although with the common type of inclinometer, an accuracy of about $\pm 1 \times 10^{-3}$ radians, or 3 mm over a movement zone of 3 m can readily be achieved.

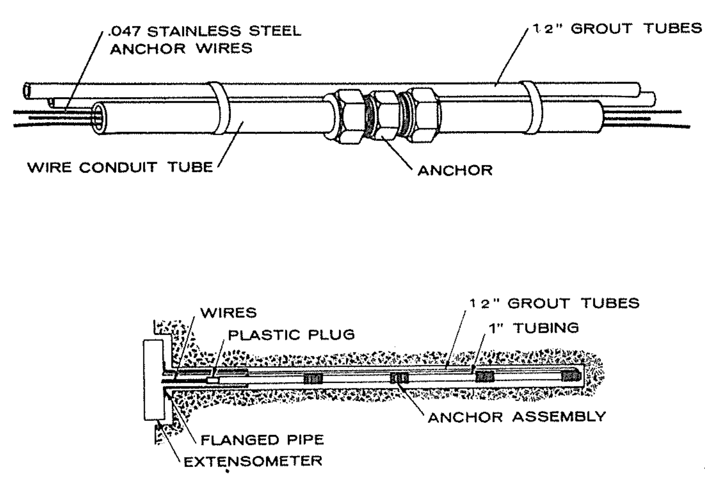
3.2.2 <u>The Extensometer</u>: Where movements of low magnitude are expected, such as in rock slopes, the extensometer is used for detecting horizontal movements. Basically the device consists of a tensioned wire or wires installed in a hear-horizontal boring. The inner ends of the wires are grouted or wedged in place and the relative movement between these fixed ends and the rock face measured by means of strain gauge potentiometers located at the rock face, as shown on Fig. 3.

Relative horizontal movements of as low as 0.05 mm may be measured with this device. An alarm system may also be incorporated into the readout box which can be pre-set to go off when a given displacement occurs.

3.2.3 <u>Accoustic Monitors</u>: A recently developed highly sensitive listening device is available which reportedly can detect sub-audible noise resulting from distress of rock and soil slopes. Difficulties in separating background noise such as from groundwater flow and in correlating potential or actual movements with noise amplitude and frequency are two problems associated with the device, although studies are continuing.



Hydraulic Rock Anchor



Grout-in Rock Anchor

Fig 3. Extensometer Installation Details

3.3 Groundwater Monitoring

Whenever an effective stress analysis is conducted on a soil or rock slope, accurate determination of groundwater levels in all subsurface strata is of prime importance. Several instruments which are capable of measuring groundwater levels in the ground are available, and basically they may be divided into two groups: observation wells and piezometers.

3.3.1 Observation Wells: The observation well, or open standpipe as it is often called, is used to record the water level which would rise in an open borehole, after all drilling fluids and equipment are removed. Water levels are recorded which may be influenced by piezometric levels from several different soil layers.

In some instances, particularly in remote areas and where animals are excluded and soil conditions permit, the observation well can merely consist of the open hole remaining after drilling has concluded. In many cases, however, it is desirable to backfill the borehole as an accident prevention measure, and to avoid the sluffing of soil around and within the open hole. An open standpipe, consisting usually of 12 mm PVC or iron pipe and perforated at the lower end is commonly used. Frequ[illegible] a short section of perforated pipe of about 25 mm diameter is attached to the lower end of the standpipe. This enables time lag factors to be minimized and makes the installation less prone to vandalism.

Water level observations are generally made by lowering an electrically activated probe down the standpipe. When water is reached, the circuit is closed and a light on the water level recorder is displayed.

3.3.2 Piezometers: Piezometers measure the hydrostatic pressure of the pore-water which exists in a confined layer of soil. They are particularly useful for measuring perched and artesion groundwater levels.

Considerable care needs to be exercised in installing these devices, particularly in the sealing of the waterbearing strata to be monitored. Typically, sand is used as backfill around the piezometer and a flexible impervious seal, such as bentonite, placed immediately above the piezometer.

Multiple position piezometers are often placed in a single borehole with varied success. However, where the determination of groundwater levels is critical, piezometers installed in several boreholes will provide a greater degree of confidence in the accuracy of the piezometric levels measured. Furthermore, the associated additional cost is usually minimal.

Piezometers which are commonly used in natural soil slopes are as follows:

Perforated Tip Piezometer - A perforated or porous tube of about 25 mm diameter and about 0.2 m long is attached to a riser pipe of 12 mm diameter and placed in the open borehole. Sand surrounds the tip and the impervious seal placed immediately above the tip. (Fig 4)

In soft soils it is not always necessary to drill a borehole and instead the piezometer may be installed by pushing or driving it to the desired depth. A standard driven well screen is often used to protect the piezometer, or alternatively, specially manufactured piezometers are available for this purpose (Parry 1971). (Fig 5) Piezometric levels are usually measured with an electrically activated probe as described above.

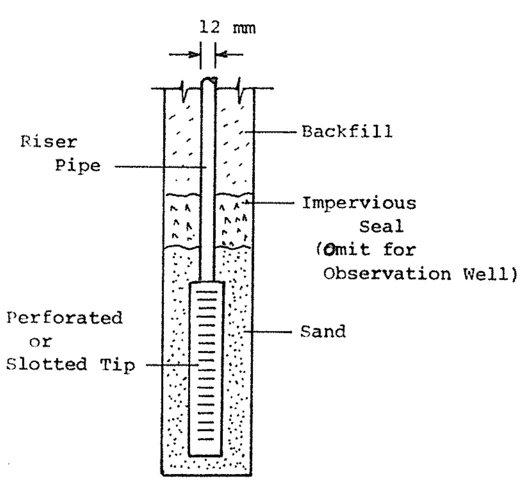


Fig 4. Perforated Tip Piezometer

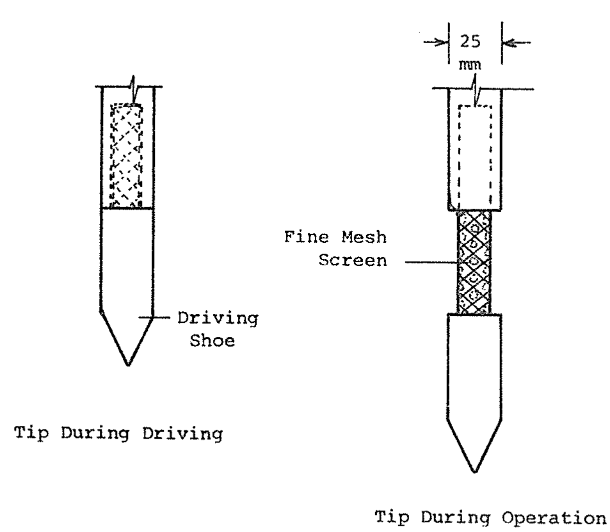


Fig 5. Driven Piezometer

Zero-Volume Change Piezometers - Where an immediate response in the recording of groundwater levels is required, it is necessary to use an instrument which has zero time lag between the change in pore water pressure and the measurement itself. Several instruments are available for this purpose, including the electrical type, which records the deflection of a sealed membrane, and the pneumatic type where a pressure-sensitive valve is activated by a calibrated pressure applied at the ground surface. Two air, water or oil-filled lines are used to transmit the pressure from the instrument recorder to the piezometer tip. In the majority of situations, however, the simple perforated tip piezometer is the most suitable for long-term observations of groundwater levels.

4.0 PERFORMANCE MONITORING

The systematic and regular recordings of instruments after they are installed and calibrated, together with all relevant information such as rainfall data etc, is of equal importance to the selection and installation of the instruments themselves. Collection of this data is considerably simplified if standard forms are prepared in advance and its evaluation is more readily visualized if up-to-date time-scale plots of movements are maintained in the office.

Monitoring schedules often require frequent adjustments because of changing conditions or related construction activities. The monitoring itself requires an individual who is not only aware of the limitations of the instruments but is also capable of sorting out erroneous observations and is able to note any other on-site visual information which may be relevant to the continuing understanding of the problem.

Methods of presentation of data usually employed is demonstrated by the examination of case histories which have been reported in two overseas publications.

Minneapolis Freeway Slide - Wilson (1970) describes a slide which was activated by cutting through dense glacial drift in the course of a Minneapolis freeway construction. Excavation slopes had been conservatively designed at 1 vertical on 2 horizontal and site explorations had revealed no weak layers within the natural slope. As the cut progressed, some heaving was noted at the base, with subsequent cracking in the slope above. Inclinometers were installed on order to determine the failure surface at typical locations shown on Fig 6. Subsequent explorations revealed that movements were taking place along a very thin layer of bentonite within the underlying shale deposit.

Data from the inclinometer recordings also indicated that the slide was progressive in nature and thus a temporary buttress of sand and gravel was placed at the toe of the slope in order to halt the movements while remedial measures were studied. Final remedial treatment consisted of the construction of a series of underground cast-in-place concrete buttresses, 1 m wide at 4 m centres. The design of the buttresses was materially assisted by the analysis of inclinometer readings both prior to and during temporary buttress construction.

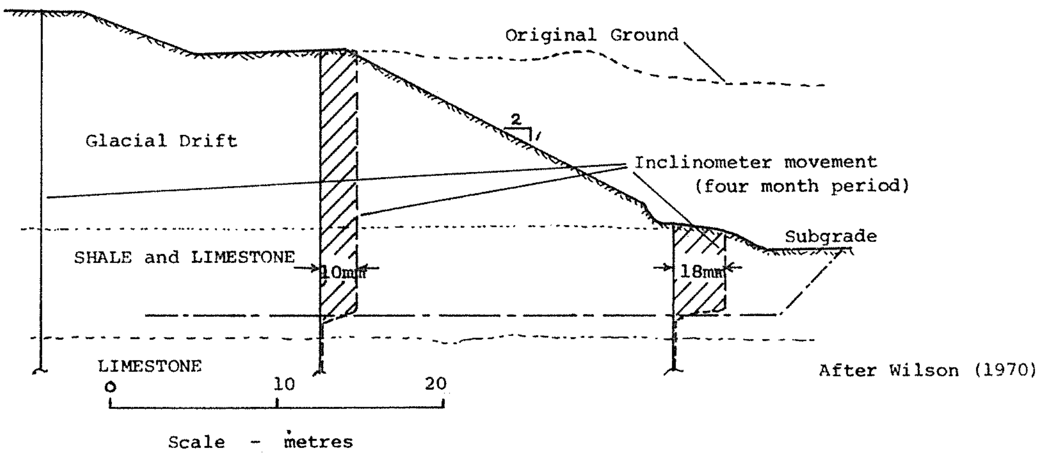


Fig 6. Typical Section, Minneapolis Freeway Slide

Waiomao Slide - Correlations between daily precipitation and slope movements can generally be expected, although in many cases the variation in daily rainfall is too erratic to be co-ordinated satisfactorily with actual movements. Peck (1967) suggested that the rainfall which accumulates over a period of several days is more likely to be significant if effecting the movements of the mass, with the actual period adopted for correlation depending on the size of the sliding mass and its average permeability.

Peck studied the Waiomao Slide in Hawaii and found an excellent correlation between the rate of movement of the sliding mass and the rainfall in the preceding 10 day period. Figure 7 shows a comparison between accumulated rainfall and slope movement during 1959 and 1960.

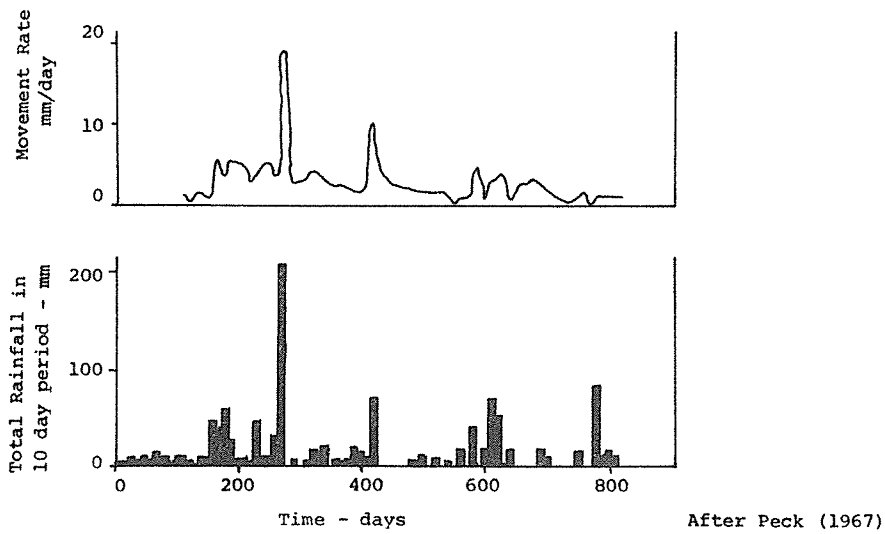


Fig 7. Relation Between Rate of Horizontal Movement and

10-Day Accumulated Rainfall, Waiomao Slide, Honolulu

In some instances, actual failure of a slope may occur several weeks or even months after precipitation, and also when all movements of significance have practically stopped. Peck (1967) presents the Lower Baker Slide as an example of a slope of this nature where the major precipitation occurred some 3 to 4 months prior to actual failure. In fact, the low rainfall and corresponding relatively small displacement within the 2 months immediately preceding failure gave the appearance that temporary stabilization of the soil mass had been achieved.

5.0 TOLERABLE DEFORMATIONS AND GROUNDWATER LEVELS

At the commencement of a slope monitoring program, some consideration must be given to deciding items such as what magnitude of slope movements can be tolerated, when should the time interval between measurements be increased or decreased, and what levels of groundwater can be tolerated? For the latter case, data from effective stress slip circle analyses will presumably give a theoretical tolerance of the slope's ability to withstand changes in groundwater levels. The former two items present some difficulty in determining an exact assessment, however.

Where only surface slope markers are used, total mass movements must be disassociated from soil creep and other local disturbances. Gray (1974) compares creep profiles in soil beneath living trees and soil on a tree-felled slope within a volcanic tuff residual clay and has observed creep rates of about 0.6 mm/year for the former and about 1.5 mm/year over a 10 year period. In a weathered glacial till of 30 degree slope, Wilson (1970) reports a creep rate of average 8 mm/year over a ten year period. Thus considerable variations in creep values may be expected, depending largely on slope inclination, soil type, vegetation cover and rainfall.

In the case of cut slopes, some deep-seated movements must be expected as the soil mass responds to the differing stress conditions. Finite element techniques enable an approximation to actual deformations to be made, providing of course that all input data relating to the stress-strain relationships of the natural soils is reasonably representative. These techniques have been used with some success in predicting embankment deformations (Resendiz and Romo (1972), Penman and Charles (1973) and others), however in a natural soil slope, variations in material properties and the possibility of a lack of recognition of weak layers may often give misleading results.

Gould and Dunnicliff (1971) present a simplified analysis of shear strain as detected from inclinometer measurements. While their analysis is centred around fill embankments, it can readily be applied to natural slopes, as indicated on Figure 8.

The shear strain in the field can be approximated to shear strain in the triaxial test, which in turn is related to vertical strain by the equation:

$$\gamma = \cos\phi' \left(\left(\varepsilon_a + \frac{1 \pm \frac{\Delta V}{V}}{1 - \varepsilon_a} \right)^{\frac{1}{2}} - 1 \right)$$

where γ = shear strain

ϕ = angle of internal friction (effective)

ΔV = volume change

ε_a = axial strain

or, in the usual case:

$$\gamma = 0.75 \text{ to } 1.25 \;\varepsilon_a$$

or γ = 0.03 to 0.1 radians at failure

It is further reported that maximum measured inclinometer deflections in satisfactorily performing embankment dams has been of the order of 0.01 radian, indicating that the above analysis can be assumed to be reasonably correct.

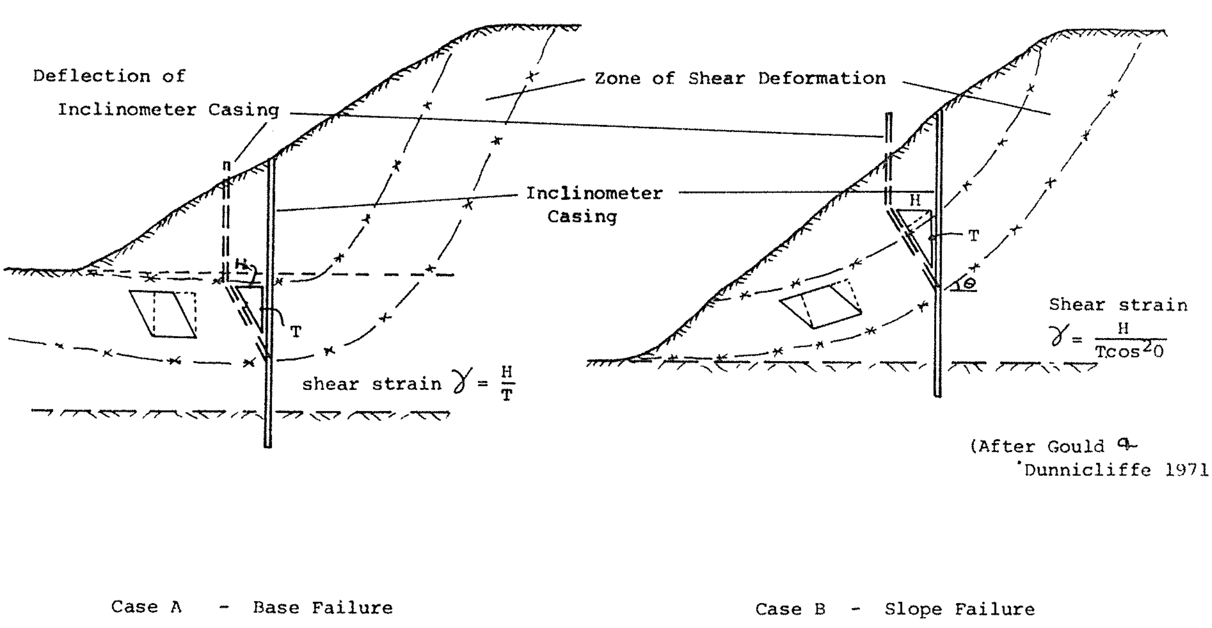


Fig 8. Shear Strains From Inclinometer Observations

6.0 CONCLUSION

A complete study of the future performance of a natural soil and rock slope must always include an evaluation of its present condition. The study should include, in addition to the collection of data relating to soil type and shear strength, some monitoring of groundwater fluctuations and ground deformations. In areas where stability is questionable or where the effects of construction on adjacent land must be determined, monitoring of slope movement is essential.

In locations where piezometric levels and ground deformations indicate areas of potential instability, such data should be supplemented and extended in order to accurately determine the lateral extent of the problem. It is only by the adoption of this approach that remedial works of adequate size and minium cost can be soundly designed and constructed.

It is probable that in the future, ground surface deformation measurements will become more widely used as electronic distance measuring equipment becomes more readily available. However, such measurements must remain as only a supplement to below-ground deformation measurements if true failure surfaces are to be accurately located and remedial works constructed accordingly.

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Chairman: *Mr. G.L. Evans*

Section 6: ENGINEERING ASSESSMENT

PRESENTATION AND DISCUSSION

Mr. J.P. Blakeley, introduced his paper, "Determination of Relevant Information for Assessment of Stability of Soil Slopes", (p 6.1). The first and probably most important point in the paper was that it would be most unwise to assume prior to a site investigation that any natural slope would be in a uniform material. To quote the paper, "Stability of Natural Slopes" by Professor Peck, "Few natural slopes exist above homogeneous material or above simple materials of which the shearing strengths can be stated in terms of one or two parameters. Indeed natural slopes above homogeneous soft clays are so rare that such a slope would be considered to be a geological anomaly". Hence the objective of any site investigation programme must in the first instance be directed towards defining the soil strata beneath the slope, and in particular the critical soil layer or layers to be used in the analysis. Here geological advice could be most important to the engineer. An understanding of the manner in which soil strata had been laid down in an area and of the local slope-forming processes could be most useful. He made the point that engineers generally seemed much more willing to seek geological advice for slope stability problems in rock than for slope stability problems in soil. Along with studies of the soil strata, slope geometry must be carefully studied by obtaining surveyed cross-sections down the slope or from contour plans. Also it was important to appreciate the three-dimensional nature of most slope stability problems. In the case of site investigations carried out where the slope had already failed, the investigation and analysis is carried out until it has been proved that at the time of failure the safety factor must have been less than 1. He believed it very important that the same thorough approach be taken to investigations and analysis in slopes which had not yet failed. He again quoted from Professor Peck's paper: "The likelihood of reactivating old slide areas by apparently minor construction activities is not a new conception. Experts have been pointing out the implications of old slide areas for years. They have gone so far as to suggest that if there are no old landslides in an area it is fairly unlikely that a moderate construction operation will start a new one. On the other hand, if the old landslides abound, it is quite likely that even minor construction operations will lead to sliding. Possibly emphasis on the analysis of sliding movements has led us to overlook the practical significance of these deductions". Regarding site investigation procedures, Mr. Blakeley said that in areas that had already been developed, particularly subdivisions, the observation of cut batter slopes would be most useful. Also documentation of previous damage in a subdivision or a road area would be most useful. Mr. Blakeley then mentioned the importance of carrying out site investigations for slope stability in two or more stages and the futility of trying to organise a complete soil testing programme until the actual soil strata which are present in the slope have been defined. Hence, he believed that the first stage of a site investigation should always consist of the best possible coverage of bores over the site area with continuous coring in order to detect any thin, weak layers and in this stage there should be only a limited amount of laboratory testing. The second stage of site investigation should have the objective of obtaining the best quality samples from the laboratory testing or carrying out in situ tests in particular soil layers and possibly also further exploratory boring to clarify any anomalies in the soil strata. If a site investigation could not be staged in this matter, then there must be a maximum flexibility to change the investigation as it progressed and constant engineering input during the field investigations. With regard to soil sampling and the handling and transportation of soil samples and the logging of drill holes he felt the best reference was the proceedings of the Symposium on Site Investigation held in Christchurch in August 1969. Mr. Blakeley concluded by mentioning the Lower Baker slide which had completely destroyed a power house in Seattle, in May 1965. The site was located in an area where annual rainfall was fairly large and concentrated in the winter season. It seemed from the outset that rainfall was a significant factor in the development of the slide. Movements of the slide were noted as early as 1945 when the area was stabilised using surface drains. The slide area was observed frequently thereafter and remained completely stable until late in the winter of 1964 when major cracking occurred south of the initial slip area. During the spring and summer of 1964 borings were put down and slope indicators were installed to detect movement. Horizontal drains were also installed in the upper part of the slope to try and lower high piezometric levels. No movements of the slope were

detected during this period, but in November 1964 in the late autumn after ten days of heavy rain, the slope again became active and the slope indicators were sheared off before measurements could be made throughout their entire length, and the central part of the slide moved as much as 3 ft in 1 month. Emergency measures were taken to drain the slope including surface ditches (lined with plastic to prevent water from entering the cracks) and large diameter vertical drainage wells each equipped with a pump. There were some minor movements during the rest of the winter when further heavy rain fell, but by 10 April 1965 the movements appeared to have stopped entirely, and it was believed at this time that the dry season was now available for reconstruction and stabilisation of the slope. Suddenly, with very little warning on 18 May 1965, the entire hillside appeared to join one major mass movement which completely destroyed the powerhouse. Professor Peck was one of three leading consultants who had been engaged to study the slide and his conclusion was as follows:-
"Although the details of the Lower Baker failure were studied with great care,...... we simply do not understand the reasons for the rapid development of the slide in what was expected to be a period of grace during which remedial measures could be carried out. Hence, we have lost much of our confidence in our ability to predict the behaviour of a natural hillside or in the results of our remedial measures. On this project, it is evident that nature was able to outwit us, and we fear she can and will do so on similar occasions in the future. This, I submit, is the present state of the art".

Mr. Kayes said that his paper, "Analysis of Natural Earth Slope Stability under Static Loading" (p. 6.13) outlined the most commonly used methods of stability analysis for soil slopes. He hoped that engineers may at least have a starting point and be guided to a method of attack. He then added a few words about the factor of safety. He said it should be realised that the factor of safety was simply a ratio which defined the results of computations in accordance with some adopted definition. In slope analysis the definitions varied, e.g. some methods based their definition on strength, the ratio being the available strength along the failure surface to that required to maintain stability. Other methods defined the ratio in terms of restoring moments to overturning moments along the failure surface. The two definitions would give the correct result for a slope at critical equilibrium, that is $F = 1$, but for stable slopes the variation in definition would give variations in factors of safety. Fortunately the way of defining factor of safety was not critical because engineers tended to become accustomed to working within a range of values which they had found by experience to be appropriate for a particular method. Mr. Kayes then considered the various methods of analysis available for the practising engineer. The first approach was applicable to simple slopes and homogeneous soils. If the slope could be so rationalised there was a rapid method of solution by use of charts. For the more typical case where there was stratified soil and complex geometry the most appropriate method was the Bishop Simplified method. In his paper the definition of the factor of safety was given by Equation 2 for this method and for hand analyses the chart given in Fig. 2 would speed up the whole process. To maintain accuracy he would recommend the sliding body should be divided into at least ten vertical slices. There was a simpler definition of factor of safety known as the Fellenius method. He pointed out that this method was not recommended as it had been shown that there were errors, under certain conditions.

It was important to have some knowledge of costs. At 1974 rates he estimated that site investigation and analysis would cost about $4,000 for a typical slope. Finally he said that in his opinion the geological part of any slope assessment was the most important part.

Professor P.W. Taylor in introducing his paper "Stability of Natural Soil Slopes During Earthquakes" (p 6.25) said that both Mr. Blakeley and Mr. Kayes had shown that it was essential to use effective stress methods to assess long-term stability. It was possible to get factors of safety of 3,4 or even up to 22 for slopes which had actually failed by using total stress methods when they were inappropriate. In such cases a total stress analysis based on unconfined compression tests was worse than no analysis at all becuase it was misleading.

While it was difficult to analyse the stability of slopes for ordinary static conditions, it was even more difficult to predict the effects of an earthquake on stability of slopes. Current practice was that analytical methods were applied to the study of failures. This was useful in checking the validity of methods of analysis. Very small features could be quite significant in earthquake effects on slopes, particularly the presence of lenses or pockets of sand which might liquify in an earthquake and cause fialure. Professor Taylor said people were accustomed to limiting equilibrium methods of analysis of slope stability where a factor of safety was determined. For static problems if this was sufficiently above 1, then it was expected that the slope would be stable. These methods had been carried over into the earthquake analysis of slope stability where they were not truly appropriate. Also, one had to decide what earthquake the slope should be designed to resist. It was necessary to know the actual accelerations involved before making an analysis and it was not easy to determine what was a reasonable, probable earthquake and what its return period would be. The fact that the slope acted as a structure multiplied the horizontal acceleration near the top. Professor Taylor said that earthquake analysis of slopes was not a routine matter at all. It had been largely limited to study of failures which had occurred in the past. It was not economically feasible in an ordinary subdivision slope analysis and the times when it was economically justifiable now were when there was a large amount of capital being invested in a small area. The study of the failure mechanisms was beneficial in itself in that one knew what to look for, and then one could make some sensible judgement on whether or not it was worthwhile to go in for some special study to investigate earthquake effects.

Mr. M.T. Mitchell at a later session, introduced his paper, "The Use of Instrumentation in Evaluating the Stability of Natural Slopes" (p. 6.31). He followed the text fairly closely, illustrating his remarks with slides. He emphasised the desirability of stability assessment based on both field measurements and theoretical analysis. Too often the possibility of instrumentation being used as a means of investigation was neglected till a project was well under way and was then omitted because either the funds were short or the time limited.

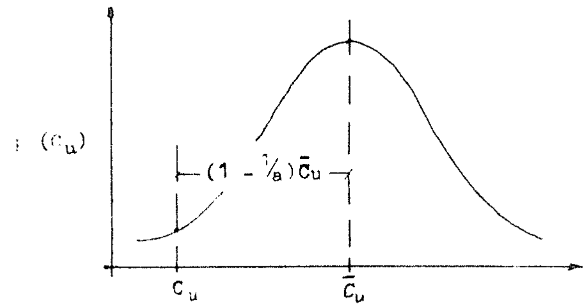
As far as the future was concerned, it was probable that ground surface deformation measurements would become more widely used as the electronic equipment became more readily available. However, he cautioned that such measurements must remain only as a supplement to below-ground deformation measurements if the true failure surfaces were to be accurately located and the remedial works constructed accordingly.

Dr. Pender opened discussion on Mr. Blakeley's paper by referring to the closing comments, "........ no matter how refined and complicated subsequent stability analyses may be, the end result cannot be any more accurate than the input data". While we all had an intuitive appreciation of the significance of the variability of natural materials, Dr. Pender presented some very simple calculations to elucidate this intuition. The well known result for the factor of safety of a vertical cut in intact soil is:-

$$a = \frac{4\,C_u}{\gamma H} \qquad (1)$$

where a is the factor of safety
C_u the undrained strength
γ the bulk unit weight
H the height of the cut

For a given H, a depends on C_u and γ. In general the variability of C_u is much more significant than that of γ so it is convenient to consider γ as fixed deal only with variations in C_u. We now seek a numerical measure for the consequences of such variations in C_u. This can be obtained easily enough if one calculates the probability of failure for various design factors of safety as a function of variations in C_u. If a number of specimens are tested a distribution of C_u values is obtained which is characterised by a mean value, $\bar{C}_u$ and standard deviation σ. This distribution may take various forms, but the calculations are particularly simple if a normal distribution is used. For a given factor of safety based on the mean strength the probability of failure is the probability that C_u is less than the value required for a factor of safety of unity. The difference between $\bar{C}_u$ and the value at failure is $(1 - 1/a)\ \bar{C}_u$, the probability of such a difference existing is easily found from tables for the normal distribution.



for a = 1

The various probabilities are tabulated below:

Factor of Safety	(1 - 1/a)	COEFFICIENT OF VARIATION, $\sigma/\bar{c}_u$			
		0.40	0.30	0.20	0.10
		Failure Probability			
1.25	0.20	0.310	0.250	0.160	0.020
1.50	0.33	0.200	0.160	0.050	4.0E-4
2.00	0.50	0.100	0.050	0.006	-
3.00	0.67	0.050	0.012	4.0E-4	-
5.00	0.80	0.023	0.004	3.0E-5	-
10.00	0.90	0.013	0.001	-	-

These probabilities emphasize the critical importance of the variability of the material, (Wu, T.H., Kraft, L.M. 'Safety Analysis of Slopes'. Proc. ASCE, Vol. 96, No. SM2, March 1970, p. 609-630) report values as high as 0.40 for the coefficient of variation. Adopting a value of about 0.001 as an acceptable probability of failure (still a very high value in comparison with values regarded as acceptable for structural safety) it can be seen that surprisingly high factors of safety are needed to ensure the stability of the cut when the coefficient of variation is large.

These calculations are to some extent limited by the assumptions involved, the normal distribution does not express the variability of the soil properties at large deviations from the mean, the assumption that the soil is homogeneous is doubtful and there is rarely one random variable involved. Nevertheless assumptions have the advantage of great simplicity and the calculations illustrate very clearly that the concept of factor of safety may be of value when the soil has little variability, but provides a false sense of security when the coefficient of variation is high.

Mr. Blakeley said that this highlighted the need for field measurements to check the uniformity of any particular soil layer. Obviously triaxial or any sophisticated laboratory tests could only be done on very limited samples of the ground. Therefore, he felt [illegible] very important to have some idea of the uniformity of the layer within which one had taken the soil samples for laboratory tests and this was where he thought the Standard Penetration test, Dutch cone, in situ vane test or any other test which could be done at regular intervals could be most useful.

Mr. Brickell said he was fascinated by the end of Mr. Blakeley's talk where he had referred to the refreshingly honest conclusion of Professor Peck. He referred briefly to the history of the use of soil mechanics in New Zealand. The period when he had been primarily concerned with this terminated in 1960. At that time there were moistly people who observed, were fascinated by the things they observed and had gone to soil mechanics to try to explain their observations. Then there was the arrival of the computer and the emphasis had changed to analysis. Professor Peck was in fact saying 'these problems would be solved if they were to be solved at all'. Mr. Brickell then outlined an experience where he and his colleagues were involved in the design for stability of a very high cut slope. They had done a lot of field measurements, slip circle analysis, wedge analysis. Eventually, they built a photo-elastic model to determine the pattern of stress. Using this method, Mr. Brickell said, he believed they came very close to a valid answer.

Dr. Webster said that the extent to which discussion had been restricted to limit state methods for the analysis of slope stability had been highlighted by the interesting contribution from Mr. Brickell. While physical modelling of the elastic stress field through the method of photoelasticity had been largely superseded by mathematical modelling based on the finite element method, the principle of approaching the limit state without imposing a preconceived failure mechanism remained valid. The finite element method could readily take account of joints, tension cracks anisotropic material properties, nonlinear stress-strain relationships and nonlinear cohesive and frictional properties.

A good deal of work in this field had been carried out by the Systems Laboratory of the Ministry of Works and Development which had mainly attacked the problem by developing an integrated system which could analyse the interactive processes of stress transfer and fluid flow in parallel, and had achieved a good deal of success. The importance of pore water pressures had been emphasised by many speakers. The above approach not only permits a reasonably accurate assessment of long-term settlement patterns in the natural slope but also allows alternative drainage systems to be investigated and their effectiveness to be compared.

There was a common misconception that finite element methods were complex and required considerable effort for data collection and preparation. The methods were in fact conceptually simpler than the accepted limit state methods, and, given automatic mesh generation, required very little additional data. The finite element approach made the best use of the parameters which had to be experimentally derived in any case. The moral would appear to be that, if an analysis of some particular slope was deemed necessary, then that slope, and not some other slope which happened to be easier to analyse, should be investigated. Dr. Pender had pointed out that, when experimentally derived parameters were known to be accurate only within a certain margin of error, computed factors of safety could give a misleading impression of the actual probability of slope failure. The finite element method was particularly well adapted to an application of the type of probability assessment which he had described. There was no need to decide in advance on the type of failure surface which might be expected - the program was able to determine the failure mechanism from an examination of the basic material data and the computed stress field.

In conclusion, it should perhaps be emphasised that the techniques advocated were currently available to engineers within the Ministry of Works and Development, and that Dr. Pender had already made use of the stress analysis package. The procedure did not impose heavy demands on computational resources, and it was entirely within the bounds of possibility that a slightly modified version could be implemented on the type of desk computer to which Mr. Gillespie had referred. The limit state methods described by Mr. Kayes had served the profession well, and no doubt would continue to be used extensively for a number of years. However, it should not be assumed that they represented the last word in analytical techniques available to the practising engineer.

Dr. Hughes discussed the problem of obtaining useful information from poor samples. When testing over-consolidated clay, drained tests were necessary and should be done at a confining stress much higher than in situ.

Mr. Mitchell said that it had been mentioned that where there was seepage parallel to the slope, the safe slope angle became equal to about half the natural slope angle. This was particularly noticeable in the hydro lakes with their fluctuating lake levels. Referring to Fig. 2, it could be seen that near the toe drainage was parallel to the slope. If one used Mr. Smith's gabions as a filter in this critical area, the problem was resolved and seepage was no longer parallel to the slope but was nearly horizontal. The safe slope angle thus reverted back to the steeper situation.

Mr. Thomson said that slope stability analysis and collection of data was based upon the static situation, but the situation was far from static once failure commenced. Speakers had blamed water as the major cause of failures and because of an apparent phobia of water, had used drainage as a major remedial measure, and considered the static situation. But in fact, drainage may accentuate the wetting and drying of slip material thus imposing a dynamic effect which could accelerate weathering with resultant loss of soil strength.

Mr. Thomson, in referring to Mr. Kayes remarks about the non-uniformity of slipping along the slip plane, said this could be accentuated by non-uniform drainage in the vicinity. Therefore, internal stresses developed within the slip material causing a readjustment of stress loadings, and failure occurred in a way that invariably defied analysis.

Regarding Professor Taylor's remarks on the dynamic effects of earthquakes, Mr. Thomson said that earthquakes occurred over a short time period and Professor Taylor suggested the use of an analysis incorporating dynamic loading. In many situations, the only difference between earthquakes and other slower changes was in the time scale, and that dynamic effects may be added to the normal static analysis. Referring again to drainage, he said he would be interested to hear if anyone had considered water control instead of drainage. Surely the major need was to create consistent soil moisture conditions within the slip throughout the year. In some soils it was important to eliminate summer shrinkage.

Irrigation to improve soil moisture uniformity and to establish deep rooting vegetation that would lead to consistent soil moisture conditions, may often be a more desirable remedy than drainage and should not be discounted. He referred to a situation where severe tunnel gulley failure of a slope had been eliminated by major reworking of the material and elimination of gullies that were providing extensive non uniform drainage. As a consequence, the water table had been raised, thus allowing improved growth of vegetation, reduction of shrinkage cracks and the creation of a more homogeneous material with greater stability.

Professor Taylor referred to the previous remark that rapid load fluctuations in earthquakes differed only in time scale from other fluctuations due to seasonal movement etc. This, he said was superficially so but the time scale had an important effect. In an earthquake the fluctuations were rapid enough that a certain layer of sand was effectively undrained so that any water pressures that accumulated or appeared in that sand at that time, did not have time to dissipate during the earthquake. Any long-term fluctuations, seasonal for example, by comparison with the rate of drainage from the sand layer, would be very slow and the sand would be effectively drained. So the difference in time scale had a very important effect of making a sand layer either drained from the long term ones, or undrained from the short term ones.

Mr. Blakeley agreed with Mr. Thomson that to aim for a uniformly high water table level in the area had some advantages providing the high water table level in itself was not going to endanger stability. He agreed also that shrinkage cracks were a very serious cause of slipping and a high water table level would help to control this.

Mr. Kayes referred to his paper para. 2.2 where he had tabulated the variations in resulting factor of safety on a variation of input variables. It could be seen that the unit weight was a factor which did not alter the factor of safety very much.

Mr. Toynbee asked if satisfactory allowance could be made for future weathering in the analysis, in cuts, when new materials would be exposed to the elements.

Mr. Blakeley said he had shown one or two diagrams showing the stress-strain curves going up to a peak strength and then down to a residual strength. Possibly to allow for effects of weathering, some conservative views of what the shear strength in this zone in which weathering might occur, should be taken. One would have to look at similar weathered materials elsewhere. He did not think this was commonly done but perhaps should be considered more.

Mr. Carryer said he would like to reiterate Mr. Blakeley's comment on staged investigations in field investigation. He believed that if the first drill survey was designed on a geological basis (i.e. the survey was laid out to test the geological analysis of the area), then future testing for soil mechanics purposes would have more meaning. In this respect such a survey could often be significantly reduced in cost. With regard to Mr. Blakeley's comment on continuous coring, this was very pertinent in such a survey in that only from continuous coring could a geologist do a realistic interpretation of the conditions under the surface which would be tested for analysis by the engineer at a later date.

Mr. Gillespie congratulated Mr. Kayes on his excellent and rational presentation and particularly drew attention to Table 1. One could appreciate the geotechnical aspects better when that Table was understood. He said he had recently heard an engineer give details of a very costly retention device for cut batters on a major civil engineering project involving factors of safety to 1.0 to 1.1. However, on reviewing his input data, he had found cohesion difficult to measure so he had assumed it to be zero. For the angle of internal friction he had obtained values in the range of 27-38° and had taken an average figure of 34°. For bulk density he had taken an arithmetic average of a whole series of random samples. Whilst the analysis may well have been reasonable and safe, one could not start referring to factors of safety in the range of 1.0 to 1.1 when the input data was very rough.

Mr. Slimin referred to steep highway cuttings which resulted in some ugly looking roads, the idea presumably being to use the least land necessary. He made a plea for very shallow slopes for highways. Environmentally a road should produce little impact on the surrounding countryside. Perhaps slopes of 1.6 - 1.8 could be aimed at.

Mr. O'Loughlin commented on the role, particularly of forest vegetation in natural slope stability. Many instances of instability had arisen on slopes where the soil profile had developed under forest cover which had long since disappeared. Outside urban, suburban and residential areas forestry work had created a range of slope stability problems which in some areas were quite serious. On the steep hill country in Coromandel where there were clay loams, deforestation had accelerated landsliding quite dramatically. This had also occurred in some parts of the West Coast. Very limited quantitative data had been obtained on the true effects of root networks on shear strength but some data was coming to hand from Japan and North America. It was illuminating, he said, to learn that substantial tree root netowrk could increase the shearing resistance of shallow non-cohesive soils by 20-fold or more. Simple soil stability analyses indicated that in some of the steep hill country areas in New Zealand the soil only remained on the slopes because of the forest vegetation.

Mr. East said that Auckland clay soils had relatively high shrinkage properties, especially if the dessicated crust was removed. Where drainage was undertaken for stability reasons, care was taken to avoid decreasing the water table below the summer level under adjacent structures. If this could not be undertaken, a one year weathering period was recommended.

The Chairman thanked the Authors and those who had contributed to discussion.

The following discussion was primarily concerned with Mr. Mitchell's paper, "The Use of Instrumentation in Evaluating the Stability of Natural Ground".

Mr. Slimin (Hamilton) mentioned the Trans-Pennine motorway in the U.K. where massive cuttings about 200' deep were monitored by photogrammetry. A photo theodolite was used monthly. This was a comprehensive alternative to the methods for surface displacement mentioned in the first part of the paper.

Mr. Mitchell said he believed there was only one photo theodolite in New Zealand, at Twizel

Mr. Olsen commented on the operation of the inclinometer. Ministry of Works and Development staff from Central Laboratories were currently monitoring three inclinometer tubes installed in the Wellington area. The depths of the tubes was from 20 to 25 meters. The instrument being used was the Soil Instruments Series 600 Inclinometer with a digital readout box. The torpedo consisted of a bridge circuit of four temperature compensated resistance strain gauges actuated by a stiff cantilever pendulum.

It was important in any field instrumentation project to have accurately established zero readings and thus several readings were carried out on each tube over a period of two days to obtain the initial shapes of the tubes. With careful operation, the measurement of horizontal displacement of the top of the tube relative to the bottom of the tube was repeatable in each case to ± 3 mm. It was necessary to establish the range of initial zero readings so that any significant change of shape of a tube could be recognised when the average value of a subsequent series of readings was outside this range. With the three holes being studied, however, further readings over a period of four months had remained within the ± 3 mm range, and it can be concluded with a reasonable degree of confidence that these tubes had not undergone any change in shape to date.

Repeatability to this accuracy on holes to depths greater than 20 meters could only be obtained when particular attention is given to the following matters:

1. During the installation of the tube it was important to ensure that the distances between centres of the couplings remained the same for the whole depth of the tube. The datum, from which the measurements are made, could then be fixed for each tube so that at the measurement points the torpedo did not coincide with or straddle a coupling.

2. One operator should carry out all the readings for the duration of the project. This ensured that readings were taken in exactly the same positions each time and enabled the operator to become familiar with any irregularities in a particular tube.

For any inclinometer survey it was imperative that sufficient readings were taken immediately after the installation of the tube to accurately define its initial shape. Unless this was done, it might be impossible to know when significant movement had taken place. These comments applied to almost any field instrumentation programme. Too often subsequent results were rendered meaningless because inadequate zero readings were obtained.

Dr. Parton said he would reinforce some of Mr. Olsen's comments. Firstly, it was advisable that the operator knew the capabilities of his equipment and in the case of inclinometer tubes the particular characteristics of the tubes and where he may have to take extra caution with reading the instruments. The geodimeter had been used at Poro-o-tarao and over a long term the results were very consistent using the computerised least squares solution, and in fact had resolved displacements to within 3 mm over quite a large distance.

Mr. McKellar, in reply to a question from Mr. Mitchell said 8"diameter holes were used for inclinometer tubing with sand poured around the tube. In reply to a question from Mr. Yorkat, Mr. Mitchell said that polyethylene inclinometer tubes tended to twist and therefore extruded aluminium was preferable. With piezometer installation there would be a mortality rate from about 20-50%. There was no way of preventing failures so for long term security it was better to use a combination of different types.

Mr. East drew attention to the slip plane indicator. said the one shown in his paper was cheap, a piece of alkathene pipe in a bore, tested with a mandrell on a piece of string. This had worked very effectively on every slip they had dealt with. The inclinometer would be useful on some slides, but not on the type he had described.

Mr. Mitchell said that when using a tube instrument people should be aware of the limitations. The value of an inclinometer was that one could monitor the remedial work. Using a simple slip plane indicator, it was not possible to tell whether there was stabilisation.

In summary, the Chairman said that the case history papers presented had shown remedial measures which had been successful, unsuccessful and in some cases, untried. Essential to all these, were the three stages, i.e. site investigation, appraisal of the mechanisms of slope movement and then the design of the remedial measures. A common theme had been control and reduction of the excess pore water pressures.

The examples had varied in scale but were all substantial in size. It had been interesting to see what could be done on projects of this scale and where finances were not as limited as was the case for the house owner. The paper on instrumentation had shown that this could be invaluable in indicating underground mechanisms. He thanked the four speakers and all those who had contributed to the discussion and questions, and called the session to a close.

STABILITY OF SLOPES IN WEATHERED AND JOINTED ROCK

G.R. Martin and P.J. Millar

1. INTRODUCTION

This paper examines stability problems associated with slopes of weathered and jointed rock which are commonly encountered for example in the greywacke and argillite rock slopes of the Wellington region. Such weathered rock profiles are bounded by -

(a) Completely weathered rock or residual soil, where stability problems may be assessed using the conventional soil mechanics approach, and

(b) Slightly weathered or fresh rock, where stability is controlled completely by the geometry and strength of discontinuities.

Problems associated with the latter "hard rock" slopes are considered in further detail in a companion paper by Riley, while soil slopes have been the subject of other papers presented at the Symposium. The specific case of the stability of slopes in soft tertiary rocks, is covered in a further companion paper by Brown.

For moderately or highly weathered jointed rock, the strength of the rock mass and mechanism of slope failure is governed by both the nature and strength of joint patterns or discontinuities and the strength of rock fragments. These factors together with the heterogeneous nature of such slopes and the difficulties associated with measuring strengths mobilized insitu, make design based on analysis feasible or economical only for simple geological environments or for slopes of significant size and importance. For most cases, designs based on precedent modified by some exploration, testing and engineering judgement, are required.

2. VISUAL CLASSIFICATION OF WEATHERED ROCK

The weathering profile is the sequence of layers of materials with different physical properties which have developed insitu as the result of chemical and mechanical weathering processes. Profiles are bounded by residual soils at the surface and by fresh or unweathered rock at depth and may vary considerably from place to place due to variations in rock type, rock structure, topography, climate and groundwater conditions. Weathering profiles in various types of rock are dealt with in some detail by Deere and Patton (1971).

For visual classification purposes, the weathering sequence is divided into a number of discrete weathering stages, each of which may be considered to have characteristic physical properties. A large number of classification schemes have been proposed. One scheme which appears to have wide acceptance is that proposed by Fookes and Horswill (1970). Table 1 shows a modified version of the scheme, where the descriptions have been written to indicate the predominant characteristics of weathered greywacke. The depth of the weathering profile in greywacke varies considerably, but may reach depths of up to 30-40 metres.

TERM	GRADE	DESCRIPTION
True residual soil	VI	Original rock fabric completely destroyed. Rock completely changed to soil, generally light or yellow-brown sandy clay.
Completely weathered	V	Original rock structure completely weathered - crushable to light brown sandy silts under finger pressure. Original rock fabric still visible, with joint patterns marked by iron or black manganese dioxide stains.
Highly weathered	IV	Original rock structure retained but generally weathered to light brown colour right through. Most of material can be crushed to silt and sand sizes under finger pressure, but harder lumps remain. Rock structure generally open and closely jointed.
Moderately weathered	III	Original rock structure retained. Brown weathering extends part way through rock fragments, leaving grey unweathered central core. Rock structure tighter. Rock fragments easily broken with light hammer blow.
Slightly weathered	II	Hard jointed rock. Brown colour extends inwards a short distance on joint planes. Interior has colour and texture of unweathered greywacke. Separate pieces require moderate hammer blow to break.
Fresh rock	I	Unweathered greywacke. Shows no discolouration, loss of strength or any other effects due to weathering.

Table 1. Classification Scheme for Weathered Greywacke

ROCK TYPE	GRADE	SCHMIDT HARDNESS	DRY DENSITY Kg/m^3	POROSITY %
GREYWACKE	V	0	1500	> 23
	IV	0-10	2020-2540	17-25
	III	10-20	1930-2420	9-20
	II	15-25	2320-2450	7-13
	I	25-40	2540-2570	< 9

Table 2. Variation of Schmidt Hardness, Dry Density, and Porosity with Degree of Weathering

Studies of dry density and porosity of greywacke also indicate the physical changes associated with the weathering profile, as shown in Table II. The general tendency for a decrease in dry density and increase in porosity with increasing weathering is apparent. Schmidt rebound hardness tests on joint surfaces also show a good correlation with degree of weathering, as shown in Table II.

3. CHARACTERIZATION OF THE ROCK MASS

The assessment of the stability of a weathered rock slope requires in the first instance, a general characterization of the rock mass as a whole by way of a description of significant variables influencing the stability. Such a characterization should include:

(a) A visual classification of the weathering profile as described above.

(b) A description of the rock structure (geometry and nature of discontinuities such as faults, bedding planes and joints)

(c) Information on groundwater conditions in the area.

Visual classification of the weathering profile may be readily acheived by way of cored drill holes.

The extent of detail in the rock structure description depends on the nature of the stability problem being undertaken. The orientation of major discontinuities will determine the mechanism of potential slope failures at least in the case of Grades I - III rock. The strike and dip of major structural features are often exposed on outcrops of rock and may be mapped and plotted on stereonets for later analysis. However, where residual soils and vegetation overly a site, large diameter boreholes or trial trenches are essential in order that major structural features can be visually inspected and mapped.

The strength of rock joints obviously plays an important role in determining strengths that can be mobilized along insitu failure surfaces. However, except for major stability analyses, direct measurements of joint strengths either insitu or in the laboratory are not warranted. For routine stability assessment based on precedent or modified precedent (as discussed later), orientation, spacing, openness, roughness and nature of filling in joints should be noted. Such information may enable strengths along potential failure planes to be estimated or comparisons made with existing case histories, and potential failure mechanisms to be identified.

Groundwater conditions can be reliably assessed only by piezometer installations, and are regarded as one of the greatest uncertainties in stability calculations involving jointed rock, particularly in view of the highly variable permeability characteristics.

4. JOINT STRENGTH CHARACTERISTICS

As noted above, the strength characteristics of rock joints are an important factor in the overall assessment of the stability of a rock slope. A brief outline of equipment suitable for determining shear strength characteristics is given below together with a description of the nature of joint strength variation with a degree of weathering

4.1 Direct Shear Apparatus

Portable direct shear equipment suitable for testing rock joints in the field, was first developed and used at Imperial College (Hoek, 1970). The principle of the latter equipment was adopted for laboratory apparatus developed at the University of Auckland, a schematic diagram for which is shown in Fig. 1.

Test specimens having a maximum dimension of 150mm are set in a quick setting plaster or cement poured in a perspex mould. The cast sample in the form of a cube is then placed in the lower half of the shear box, and the upper half seated on top leaving the joint plane exposed over a gap of 5-10mm. Vertical loads of up to 250 kN may be applied by means of a hydraulic ram, and horizontal shearing loads of up to 250 kN applied by means of displacement controlled hydraulic rams activated by a fuel injection pump. Ram displacement rates of between 0-12mm per minute may be applied, the advantage of the displacement control being the capability of defining post failure load-deformation characteristics. The use of two horizontal rams permits load reversal. Further details of the apparatus are given by Millar (1974).

4.2 Strength Characteristics of Weathered Joints

Fig. 2 indicates the nature of a typical test result on a rock joint, where shearing is continued until residual strength is obtained, the direction of shearing then being reversed. Both shear stress and vertical displacement are shown plotted against horizontal displacement, vertical displacement indicating dilatancy on shearing. The magnitude of dilation on shearing depends on the roughness of the joint surface and on the normal stress, and decreases with increasing normal stress and with more highly weathered joint surfaces. As it is impossible to obtain several identical jointed rock samples, the use of stage testing is necessary in order to define a peak strength failure envelope. In this procedure an initial test is performed at the lowest value of a selected normal stress range, and the test stopped on reaching peak shearing resistance. The sample is then returned to its zero displacement position, the vertical load doubled, and the test procedure repeated. In this way, assuming that any surface irregularities on the joint surface sheared off at lower stress levels would not affect the strength at the higher stress level, a failure envelope for the joint may be defined.

The range of joint strength envelopes for weathered greywacke measured in tests by Millar (1974), are compared in Fig.3. The range of envelopes for weathering Grades II and III are seen to overlap which could be expected as physical differences between the two grades are governed largely by the extent to which weathering processes have penetrated the rock (a factor which will affect comparative intact strengths) with differences in the weathering on joint surfaces being somewhat of a lesser degree. The wide range of strengths observed might be expected in view of the variation of joint surface characteristics and the nature of weathering on each joint.

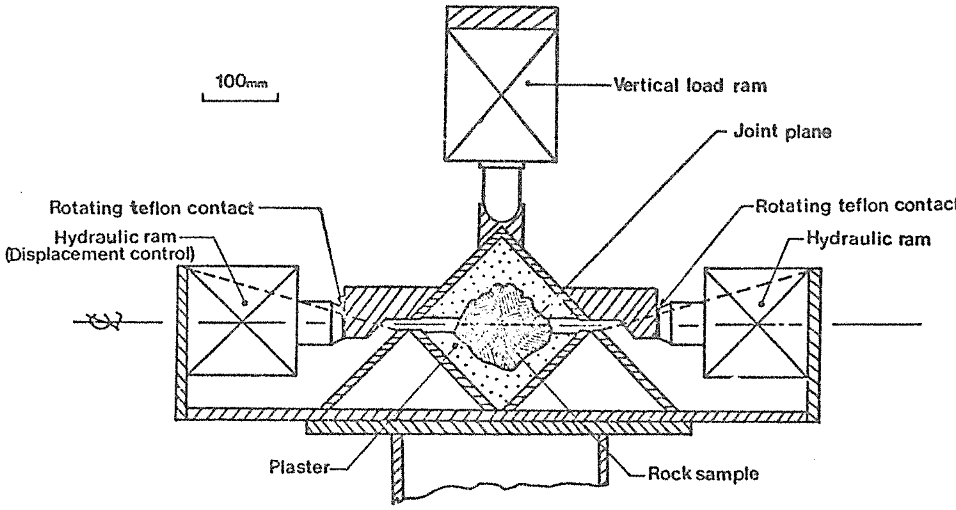


Fig. 1. Schematic Diagram of Direct Shear Apparatus

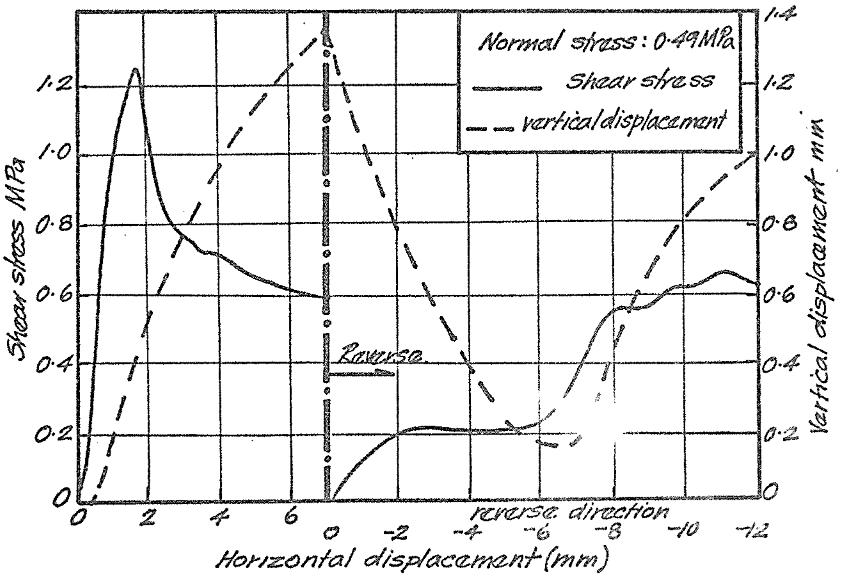


Fig. 2. Typical Results from Joint Strength Test

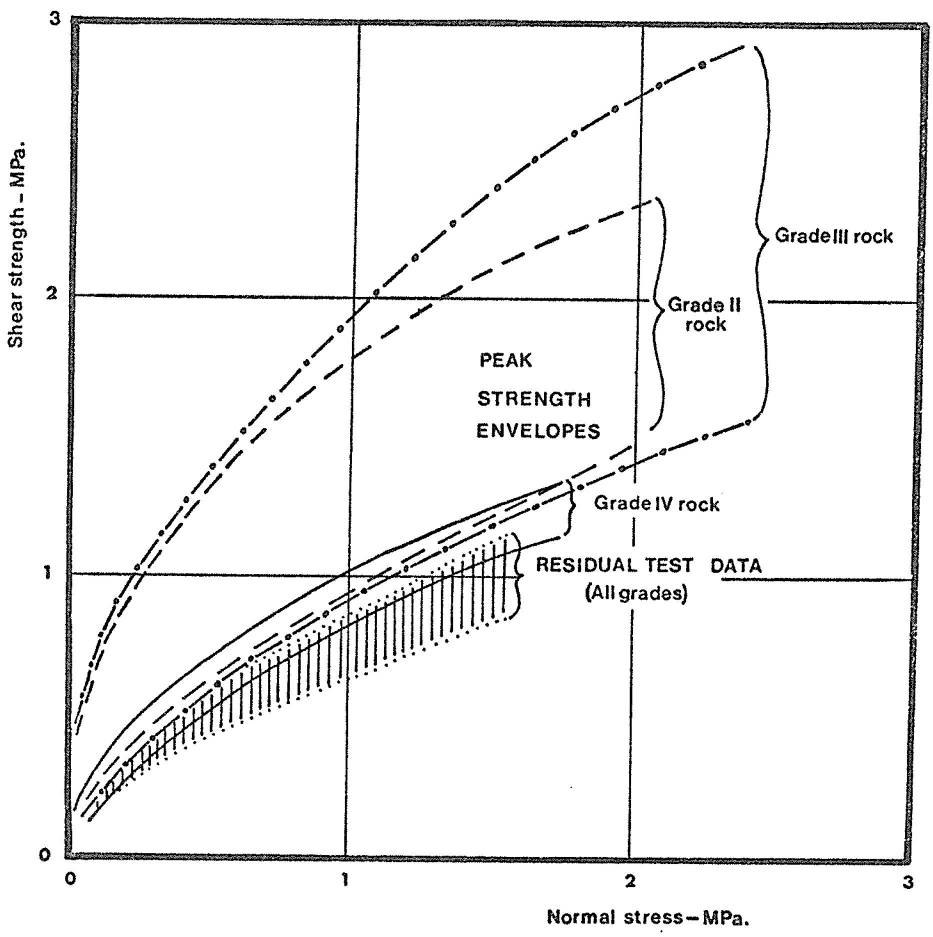


Fig. 3. Comparative Range of Joint Strength Envelopes for Weathered Greywacke

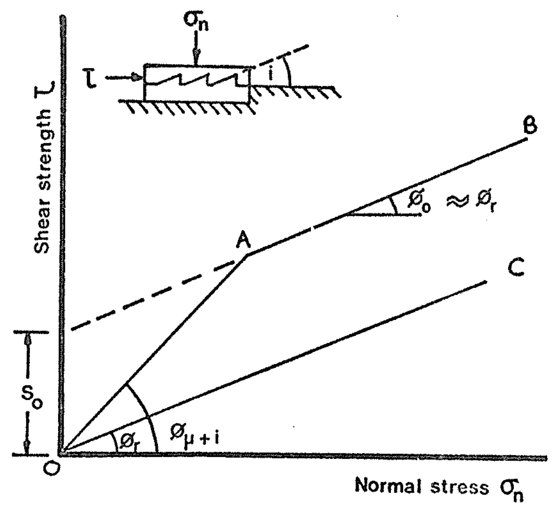


Fig. 4. Failure Envelopes for Multiple Inclined Surfaces (after Patton 1966)

The joint strength envelopes for Grade IV joints fall in a narrow range at the lower bound of Grade II and III joints. Measured intact rock strengths for the Grade IV rock are in many cases comparable with the range of joint strengths while intact rock strengths for Grade II and III rock are greater than joint strength over the normal stress range used. Petrographic studies have indicated that expansion associated with clay mineral formation in the fine grained matrix of the rock has probably broken grain interlock and cementation sufficiently in the case of Grade IV rock to produce the comparable strengths between intact and joint strength. Hence the strength characteristics of Grade IV rock may be regarded as approaching those of a soil and the effects of joint structure on failure modes in slope stability problems, may be considered as becoming of less significance.

4.3 Mechanistic Model of Joint Strength

The effects of dilation and the curved nature of failure envelopes may be appreciated by reference to the simple bi-linear model proposed by Patton (1966). The model is shown is Fig. 4, an irregular joint surface being idealized by a series of regular teeth or asperities of inclination i. At low normal loads the teeth remain intact on shearing, the joint will dilate and the shear strength of the joint may be written as

$$\tau = \sigma_n \tan (\phi_\mu + i) \qquad (1)$$

where ϕ_μ = angle of friction along the planar surface of the teeth. At high normal loads dilation will not occur, and the teeth will shear off at the base. The shear strength of the joint may then be written as

$$\tau = s_o + \sigma_n \tan \phi_o \qquad (2)$$

where s_o and ϕ_o are the Coulomb shear parameters for the rock material. For actual joint surfaces, the failure envelope could be expected to curve, reflecting varying intensities of the two modes of failure occurring simultaneously. Also at intermediate normal loads, dilation would be expected to occur until the internal shear strength of the asperities is less than the frictional shear strength and hence the reduced observed dilation at failure with increasing normal load may be envisaged as reflecting the shear of asperities at lower levels. With increased weathering, the dilatant component of shear strength at a given normal stress becomes less, and the curvature of the failure envelope decreases. A more detailed examination of the dilatant component of shear strength for greywacke joints has been presented by Martin and Millar (1974).

5. MECHANISMS OF SLOPE FAILURE

Given the often complex weathering profiles and associated discontinuities of jointed and weathered rock slopes, it is not suprising that many slope failures involve complex failure modes which would be difficult to anticipate beforehand. However for the purpose of design simplified failure modes are often assumed.

In the case of completely weathered rock and residual soil (Grades V and VI) in the absence of any domminant weak structural feature, circular are failures are generally assumed, and

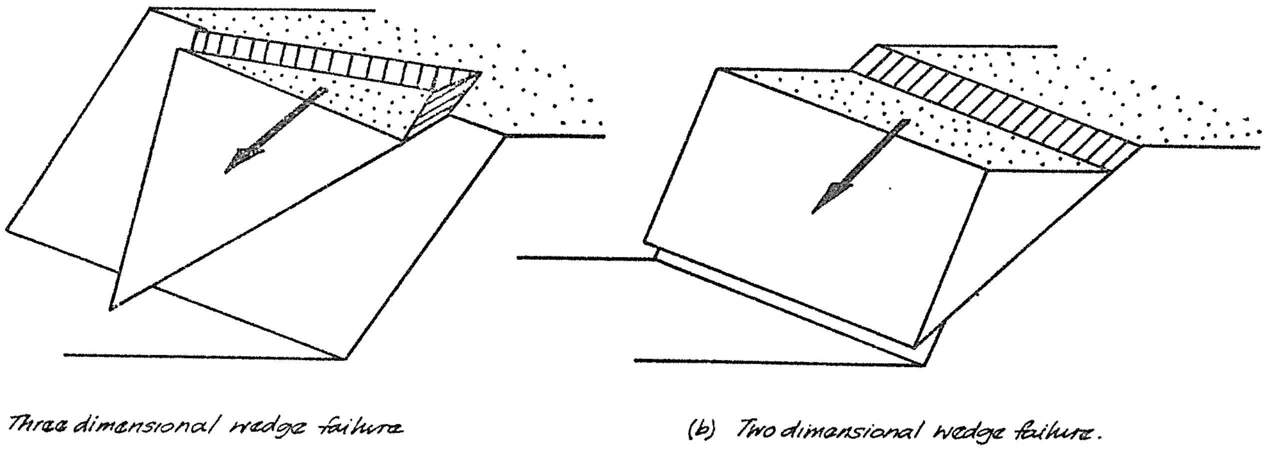


Fig. 5. Simple Wedge Failures

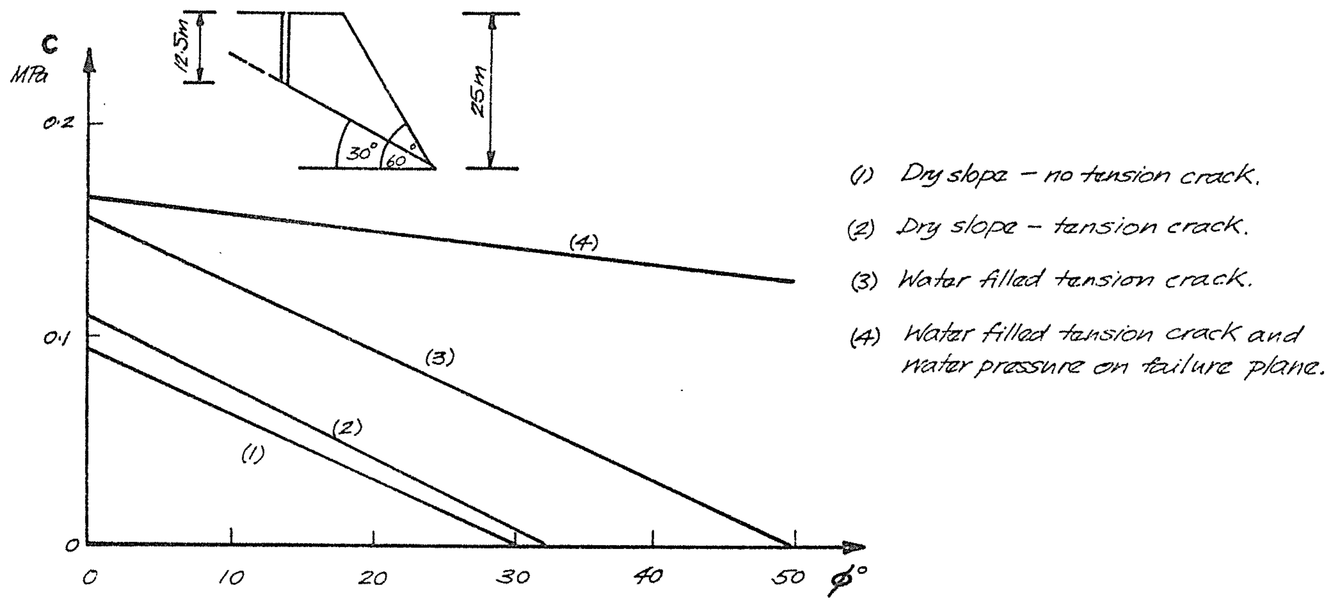


Fig. 6. Shear Strength Mobilized for Various Conditions of Two-Dimensional Plane Failure (after Hoek and Londe 1974)

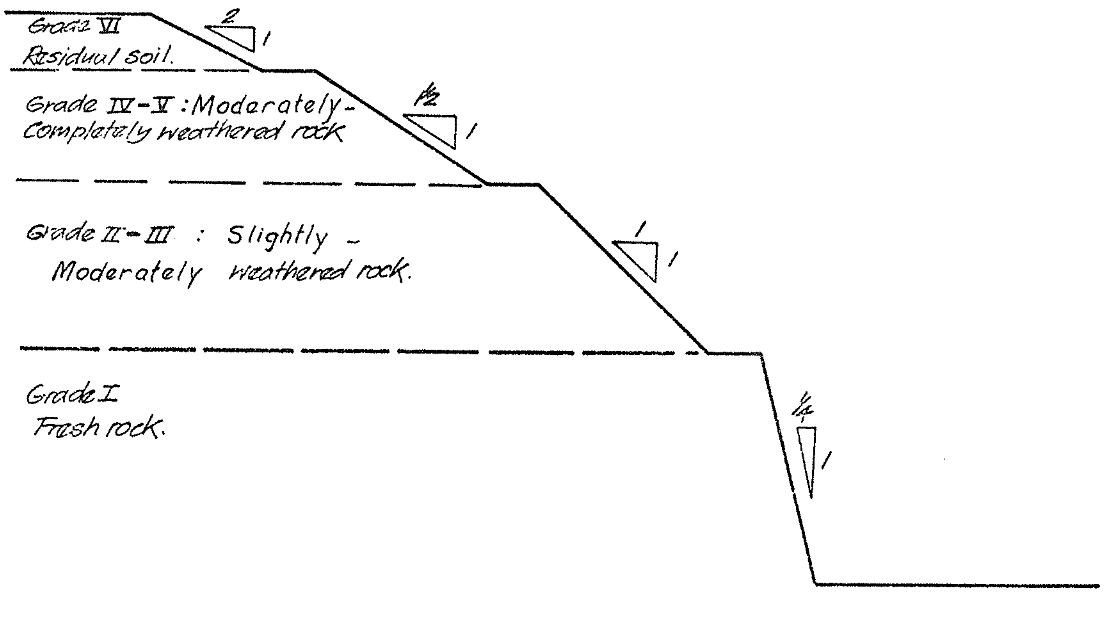


Fig. 7. Typical Slope Profile in Weathered Rock

stability analyses may be carried out as for conventional soil mechanics practice. Deeper seated potential failures in highly weathered and fractured greywacke (Grade IV) may be also assumed circular for analysis, where strength of joints and intact rock may be assumed comparable. However for shallower slips in Grade IV jointed greywacke joint strengths may dominate, and the nature of the discontinuities could define the failure mechanism.

In the case of less weathered rock (Grades 1-111), discontinuities (faults, bedding planes, joints) will determine potential modes of failure, and stereontets showing plots of all major structural features need to be examined in order to assess likely failure modes for analysis. Simple wedge failures as shown in Fig. 5a are common in benched cuts, and involve sliding along the line of intersection of two planes of weakness. Often one plane comprising the failure is the dominant bedding plane (continuous) while the other is a discontinuous plane involving, "stepped" joint surfaces. Where the strike of discontinuities is similar to that of the slope face, a simple two dimensional wedge as shown in Fig. 5b may be used for analysis.

Many deeper seated failures observed in the field involve multiple block failures initiated by hydrostatic pressures in tension cracks. A greater understanding of the mechanism of some of the more complex forms of slips in weathered rock is needed, and requires better documentation and study of the many existing case histories.

6. DESIGN PROBLEMS

Problems associated with the design of weathered rock slopes are both numerous and complex, and have been discussed in detail in a comprehensive paper by Deere and Patton (1971). Methods of analysis and general design factors, particularly for the harder rock slopes, are reviewed in a recent "state of the art" paper by Hoek and Londe (1974). Some of the more basic aspects of slope design principles, which are applicable to weathered and jointed rock slopes, are briefly outlined below.

6.1 Design Methods

Design methods may be broadly grouped into three categories, as suggested by Deere and Patton (1971):

(1) Design by Precedent. Many existing slopes in weathered and jointed rock have been designed on the basis of experience and observations of the performance of other slopes in similar geological and climatic conditions. The method is most applicable to slopes comprising massive homogeneous strata or for shallow cuts in a more complex geologic environment where occasional slips can be tolerated. However, there is considerable danger in extrapolating precedent into different geologic conditions, or using empirical design rules based on precedent for major cuts where a slip would result in serious consequences.

(2) Design by Modified Precedent. This method uses precedent but modifies the design to take into account features peculiar to the site,such as local groundwater conditions and structural configurations. The approach entails detailed surface exploration and a limited amount of sub-surface investigation, and is most suited to highly complex slope structures where it would not be economic to develop sufficient data to permit more detailed stability analyses. Elementary stability analyses might be attempted based on conservative strength parameters, with particular attention paid to adverse orientations of planes of weakness. Conservative groundwater conditions should be assumed, with full hydrostatic pressure acting in potential tension cracks. Careful observation during construction by the designer and modifications or remedial measures if required should be regarded as an essential facet of the design process.

(3) Design by Stability Calculations. This method entails detailed site investigation, and requires a full knowledge of potential major structural defects in the less weathered rock, together with a knowledge of strength parameters for all weathering zones encompassed by the slope. Because of the expense involved, detailed stability analyses can only be justified in the case of major works. This applies particularly to the case of complex weathered and jointed rock slopes, where the mechanisms of failure are often difficult to anticipate and the strengths capable of being mobilized insitu are difficult to assess accurately from laboratory tests. Hence the accuracy of analyses is not high, and in many cases refinements to design suggested by an analysis would not justify the expense.

Care must be taken in extrapolating joint strengths measured in the laboratory to continuous joint surfaces in the field due to scale changes in roughness factors. Continuity of joint surfaces in the field is also difficult to assess and strengths mobilized along potential "stepped" joint failure surfaces cannot be reliably predicted at present from separate joint strengths. As an approximation equation (1) may be used with an appropriate value of i in the case of a dilatant failure mode over a "stepped" joint surface. However, for more weathered rock joints, partial failure through intact rock could occur, particularly for weathering grades III and IV, and under these conditions the relative contributions of intact and joint strength need to be estimated. In the case of completely weathered rock and residual soil (grades V and VI), soil strength parameters such as reported by Pender (1971) for weathered greywacke, can be used in stability analyses.

Another complexity is introduced by the use of the Mohr-Coulomb failure criterion in available methods of stability analysis. This requires the curved failure envelopes of joint surfaces to be approximated by a straight line over the working stress range.

Methods of stability analysis involving wedge failures such as shown in Fig. 5 have been well documented by Hoek and Bray (1973), where design charts for simple cases are presented. A general review of graphical methods using stereonets, such as the method described by John (1968) together with other methods and aspects of stability analyses in jointed rock has been presented in a report by Richards, Millar and Martin (1972).

In veiw of the many uncertainites associated with stability analyses in jointed rock, the use of probabilistic techniques for assessing stability have been suggested, such as described by McMahon (1971). However, the factor of safety approach still appears to be favoured by most engineers, Whereas doubt can be cast on the significance of a single calculated value of a factor of safety in view of the range of variation of input parameters, the method enables the computation of the sensitivity of the factor of safety to variations in each input parameter. The effect of changes in any parameter on the rate of change of the factor of safety is possibly of more significance in slope stability assessment than the factor of safety itself.

6.2 Influence of Water Pressure

Because of the difficulty in establishing accurate flow nets in jointed rock, precise calculations of the effects of water pressure on slope stability are impractical and the normal approach is to base calculations on the worst set of conditions anticipated.

The significance of water pressures on the magnitudes of strength parameters required for stability are graphically illustrated by the simple example shown in Fig. 6, where it can be seen that significant increases in strength are required when tension cracks become water-filled during heavy rain or due to poor control of surface drainage. Where tension cracks are visible at the tops of critical slopes, it is advisable that the cracks are filled with permeable material such as gravel, and then sealed at the top of the crack with impervious clay.

Drainage is perhaps the most effective means of improving stability. The use of drilled horizontal boreholes to intersect water bearing fissures for the purpose of ensuring positive drainage should be used more often as a routine design procedure. In heavily fractured rock, holes should be regularly spaced and drilled a horizontal distance approximately equal to the height of the slope.

6.3 Slope Geometry

In addition to the slope height and angle, the ge ry of a slope may have a significant influence upon stability. Relatively small changes in alignment of slopes will respect to structural features can result in considerable improvement in stability. Whenever possible, the creation of "noses" in slopes should be avoided, as under these conditions structural features which daylight become inherently less stable.

The use of benches is a common design feature in many jointed rock slopes, and are provided to restrict slope failures to one zone only, to provide catchment areas for slide debris and to reduce erosion of cut surfaces. However, it is felt that their value is often over-emphasized. A series of low near vertical benches cut to form an average shallow slope angle often generate numerous small wedge failures whereas if perhaps just a few uniform slopes were used separated by berms, no unsightly small failures would occur, and overall stability would be just as adequate.

The use of berms in a typical slope profile for a deep cutting covering several weathering grades, is shown in Fig. 7. Slope angles are average values suggested by precedent, and are similar to those noted by Deere and Patton (1971) for example.

6.4 Rock Bolting

Improving the stability of rock slopes by the use of rock bolts or tensioned cables is generally only feasible for smaller slopes due to the forces involved. It should be noted that rock bolting is often an inefficient method of stabilizing a slope, as much of the rock strength may have been lost due to opening up of fractures and displacement along joint planes.

The use of "passive" fully grouted reinforcing has been largely overlooked as a means of improving stability, and offers considerable promise as a means of effective control of potentially hazardous structural features of cut rock slopes. If installed at the construction stage, dilation of the rock mass is inhibited, and the effectiveness if the reinforcement is greatly enhanced. The technique is discussed by Hoek and Londe (1974).

6.5 Prediction of Slope Failure

The formation of tension cracks on the top of the slope are the first indication of a deep seated failure, and must be regarded as warnings of instability. Tension cracks may occur several years before failure takes place, and frequent measurements of the opening with time may give valuable information on the behaviour of the slope. Measurements generally indicate that the rate of opening increases with time, and close correlations between opening and recorded rainfall have been observed. A rapidly accelerating rate of opening or movement of a displacement target point on a slope is indicative of incipient failure.

6.6 Earthquake Effects

The underlying principles of slope design to resist ea[illegible]akes have been discussed in a Symposium paper by Taylor. In the case of jointed rock slopes where deformation behaviour is of a "brittle nature", it is desirable that limiting equilibrium should not be exceeded at any stage during the earthquake. If peak strengths are exceeded during the earthquake, static strengths could be reduced and resulting post-earthquake failures could occur. Hence under these circumstances lateral seismic coefficients in pseudo static stability analyses should reflect maximum lateral ground accelerations in the design earthquake.

7. SUMMARY

The design of slopes in weathered and jointed rock is by no means a clear cut problem. In most cases design methods based on modified precedent are appropriate, where following an investigation of the slope site, the prime features influencing the stability are identified and their influence on the slope design assessed in a conservative manner. Careful observation during construction accompanied by appropriate remedial measures if required, should follow.

Undoubtedly further advances in methods and reliability of stability analyses will be made in coming years, particularly by way of careful documentation of case histories and perhaps larger scale field testing.

ACKNOWLEDGEMENT

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STABILITY OF HARD ROCK SLOPES

P. B. Riley

1.0 INTRODUCTION

A hard rock slope is here taken to mean a slope where structures which form discontinuities in the rock mass have a dominating influence on the stability of the slope. The strength in shear along these discontinuities may be several orders of magnitude less than the strength of individual rock pieces making up the mass. This paper will present methods of evaluating the stability of such slopes.

The following sequence has been developed for the design of slopes for open cast mines, where slope heights may reach thousands of feet e.g. Bougainville Copper mine projected face height of 3,000 ft. This program for slope analysis would only be fully carried out on a very large project where the economic incentives are high. However, the principles will hold for small slopes encountered. The operations described would be scaled down to satisfy the economics and the order of accuracy required for a particular project.

2.0 PROCEDURE FOR ANALYSIS OF SLOPES

1. Preliminary collection of geological data from air photos, surface mapping and borehole cores.

2. Preliminary analysis of geological data to establish major geological patterns. Examination of these patterns in relation to proposed slopes.

3. Slopes where no unfavourable discontinuities exist identified. Slope angles determined from convenience. Check that circular failure through intact rock is not possible.

4. Slopes with unfavourable discontinuites identified for detailed study.

5. Detailed geological investigation of critical slope areas on basis of surface mapping and drill core logging. Special drilling, adits, or large diameter bore holes may be required.

6. Shear testing of discontinuity surfaces - particularly if clay covered or slickensided.

7. Installation of piezometers in drill holes to establish groundwater flow patterns and pressures and to monitor changes in groundwater levels during excavation.

8. Reanalyse critical slope areas with above information using limit equilibrium techniques for circular, plane or wedge slides. Examine possibility of other types of failure induced by weathering, toppling or damage due to blasting.

9. Where risk of failure is [illegible] options are :-

   (a) Flatten slopes

   (b) Stabilise slopes by drainage, rock-bolts, or tensioned cables. The economies of flattening can be evaluated against the cost of implementing these improvements.

   (c) Accept risk of failure and implement monitoring program for failure prediction.

3.0 COLLECTION OF GEOLOGICAL DATA

An analysis of a slope can only be as accurate as the basic geological data it is based on. It is therefore essential that geological data be collected systematically. A two stage geological investigation is often advisable, so that critical discontinuities can be examined in greater detail when the potential for failure on them has been established.

3.1 Information Required: Ideally, the following information is required for each discontinuity to assist in deciding on failure modes and to assess shear strength: Location and elevation, dip and dip direction (this is preferred to strike, as it is unambiguous), frequency of spacing between adjacent discontinuities, continuity or extent, width or opening and gouge or infilling between faces, waviness or curvature of discontinuity surfaces and description and properties of intact rock between discontinuities.

3.2 Collection of Data: Data collection should start from a general examination of the area, e.g. with aerial photos, and work towards an examination on the ground. A freshly exposed rock face in the area will give good information on structural patterns. These patterns often persist over large regional areas. Weathered surface outcrops give only limited information as only the harder outcrops will be exposed, and the weaker zones which are more likely to be critical, will have been eroded away.

Borehole information is very useful in showing rock quality at depth, but to enable structural data to be utilised, the core from the borehole must be orientated - and this is a major problem. Where this information is essential, it may be better to sink a larger diameter (say 2 ft) bore, which would enable a visual inspection of the rock to be made. Discontinuity positions and orientations can be recorded, and samples may be taken for testing.

A very good method of establishing geological patterns in an area is to plot the collected data of dips and dip directions on a spherical projection, called a stereonet. Large numbers of joint measurements can be handled in this way to enable the structural pattern in an area to be defined. Stereonets can be very useful in indicating which discontinuity sets could cause problems in slopes, and are particularly useful in indicating possible wedge failures. However, description of the use of a stereonet is beyond the scope of this paper. A good book describing the use of the stereonet is that by Philips: "The Use of Stereographic Projection in Structural Geology" Third Edition - by F. C. Phillips.

4.0 SHEAR STRENGTH DETERMINATION

The strength in shear along these joints or shears must now be measured or estimated e.g. by shear box testing of samples from the discontinuity, to establish a basic friction angle for the material making allowances for the roughness of the sample. Shear box testing is covered by Martin and Millar in a companion paper.

4.1 Roughness Angle: The influence of various factors must now be taken into account in deciding the shear strength for slope design on an insitu joint or plane. Irregularities or waviness along the joint plane will cause the joint planes to separate as shearing takes place. The initial angle of separation can be as much as 30°. This roughness angle (i) can be added to the friction angle, giving the shear strength on the joint as

$$S = c + \sigma \tan (\phi + i).$$

The roughness angle will be modified by weathering and shearing, reducing the strength of interlocking asperities, allowing shearing to take place through them. It is obvious that estimating the roughness angle can never be very precise. Large scale shear tests can be carried out to give a more accurate answer, but these are very expensive, and therefore seldom used. The best large scale field test will be that obtained from similar failures in the field. Back analysis of these failures will give realistic strength values under field conditions.

4.2 Cohesion: The cohesion values obtained from small shear tests on discontinuities can be misleading if applied to large surfaces, where the actual cohesion applying may be much lower. If cohesion is to be taken into account in a slope design, factors which can influence cohesion strength must be understood. Faults which contain clay gouge material can decrease in cohesive strength with time; a value of $c = 0$ may be best here. Water may reduce the cohesive strength - samples should be tested wet. When shearing along a rock joint is continued until all interlocking asperities have been ground off, usually after a cm or so of movement in a shear box test, the shear strength drops from the peak strength to a residual value. Back analysis of actual failures indicate that these residual strength values are the ones that should be applied in slope design. An explanation for this may be that failure is a progressive process, with small movements on joints preceding the major collapse of the slope and reducing the friction angle to its residual value.

4.3 Strength on a discontinuity: A suitable procedure for determining the strength on a discontinuity for a slope design would be as follows:-

(i) Carry out shear box tests to determine the basic friction angle of the material. The roughness angle of the tested sample must be estimated.

(ii) Estimate the roughness angle of the particular discontinuity.

(iii) Back analyse an existing failure in the material at a factor of safety of 1 to determine the strength. Using the values of friction angle plus roughness angle determined from (i) and (ii), a value of cohesion can be obtained for the discontinuity as it acts in the slope.

5.0 ESTABLISHMENT OF POSSIBLE MODE OF FAILURE

After examining the discontinuities in a slope, va[illegible]lure modes can be postulated and their probability of occurrence in the slope assessed. The common types of failure are :-

5.1 Plane Failure: A slope can fail on a discontinuity which strikes roughly parallel to the face, and dips towards the face at a shallower angle than the face. The discontinuity must daylight above the slope toe. This plane failure case can usually be easily recognised.

5.2 Wedge Failure: Where two discontinuities intersect and form a wedge which dips towards the slope, a wedge failure can be formed. These are more difficult to detect, and the geological data must be carefully examined to see what wedges occur.

5.3 Toppling Failure: Where joints are near vertical on steep slopes, failure may occur by rotation of the top of blocks towards the cliff face. No satisfactory analytical techniques are available for this, however base friction models provide some qualitative design techniques.
Ref. Goodman, R.E., Geological Investigations to Evaluate Stability. Proc. 2nd Symposium on stability for Open Pit Mining, Vancouver, Nov. 1971.

5.4 Circular Failure: This may occur partly on discontinuities and partly through intact rock, or may follow several discontinuities to reach the slope face, or may occur entirely through intact rock. This type of failure is more likely where the rock is soft, and can be analysed using charts or Soil Mechanics techniques.

5.5 Other More Complex Failure Modes: The actual mechanism of failure may be more complex than the above simple cases. Multiple wedges may form, with friction acting between wedge blocks. These are difficult to analyse and can often be approximated by one of the above cases.

6.0 GROUNDWATER CONDITIONS

Slope failures almost invariably occur during periods of heavy rainfall; information on groundwater conditions is therefore very important in slope design.

6.1 Water Pressure Measurement: Direct water level measurement in boreholes, or pressure measurement with piezometers installed in boreholes and the results correlated to rainfall, will enable the magnitude of pore pressures on a potential failure surface to be estimated with a fair degree of accuracy. The luxury of this direct measurement is seldom available however, and groundwater pressures must be deduced from the relative permeability of the slope, and possible sources of water. In many cases, particularly with small slopes in relatively impermeable ground, and where rainfall may be very intense, the slope may become fully saturated. This will often be the critical case for slopes in New Zealand.

6.2 Pressure Distribution: In hard rocks water will flow through joints in the rock, and the permeability will depend on the frequency and opening of the joints. Pressures along any potential failure plane will be influenced greatly by this relative permeability, which will almost certainly be unknown in the slope. Assumptions must therefore be made on a suitable pressure distribution, and that suggested in the plane failure section is recommended.

In considering the effects of water on a slope, it is very important to realise that the pressure of the water inside the slope is what does the damage, not the flow. A slope may be nearly saturated, but appear dry when its low permeability allows less water to reach the surface than is able to be removed by evaporation.

7.0 PLANE FAILURE

This is in fact a special case of a wedge failure, and a true plane failure rarely occurs in practice. However plane failures are relatively simple to analyse, and are often used to simulate conditions in a slope. Analysis of this simple case allows the sensitivity of the slope to changes in shear strength and groundwater conditions to be assessed more readily than with more complex three dimensional cases.

7.1 Conditions for failure to occur:

(a) Sliding plane must strike within 20° of slope face.

(b) Failure plane must "daylight" in the slope face.

(c) Dip of failure plane must be greater than the friction angle along the plane.

(d) Release surfaces must be present at the ends of the failure plane.

7.2 Tension cracks:

A tension crack will usually occur at the top of the slope. Sometimes the crack can be observed and the geometry of the situation will allow the depth to be calculated. More often, particularly in a design study, the position and depth will be unknown. Where other controlling rock structures are not present, tension cracks are thought to develop in slopes in dry conditions. It is possible to determine a critical tension crack depth for the slope by differentiating the dry slope factor of safety equation with respect to $\frac{Z}{H}$, and equating the result to zero for maximum and minimum values. This gives the critical crack depth as :-

$$\frac{Z}{H} = 1 - \sqrt{\cot \psi_f . \tan \psi_p}$$

Geometry then gives the crack position

$$\frac{b}{H} = \sqrt{\cot \psi_f . \cot \psi_p} - \cot \psi_f .$$

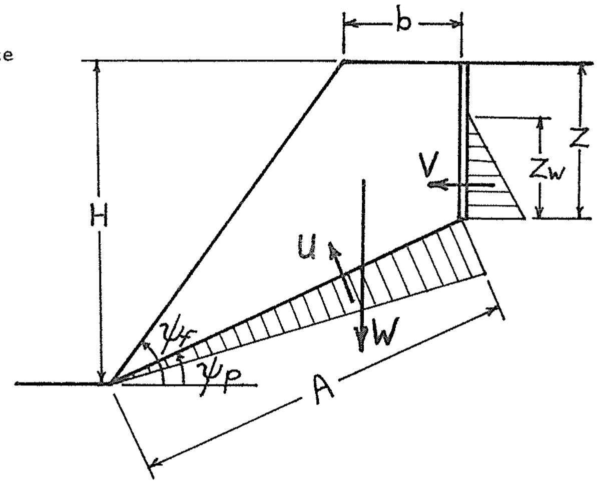
7.3 Analysis of Slopes:

The water pressure distribution on potential failure surfaces in the slope must be estimated. Water will fill the tension crack to some depth, and seep along the sliding surface. The simplest assumption for the water pressure distribution along the sliding surface is to assume it to vary linearly from full hydrostatic head at the base of the tension crack to atmospheric pressure at the slope face. Limit equilibrium analysis can then be applied. Moment equilibrium is ignored in the analysis, and failure is assumed to be by sliding only.

From the sketch:-

Resolving parallel and normal to the failure surface allows the factor safety to be calculated as :

$$F = \frac{cA + (W \cos \psi_p - U - V. \sin \psi_p) \tan \phi}{W. \sin \psi_p + V. \cos \psi_p}$$



For the geometry shown in the sketch,

$$A = (H-Z) \operatorname{cosec} \psi_p$$
$$U = \tfrac{1}{2}\gamma_w Z_w (H-Z) \operatorname{cosec} \psi_p$$
$$V = \tfrac{1}{2}\gamma_w Z_w^2$$
$$W = \tfrac{1}{2} H^2 \left[\left(1-\left(\tfrac{Z}{H}\right)^2\right) \cot \psi_p - \cot \psi_f \right]$$

This equation will enable the factor of safety to be determined of a slope on a discontinuity when the variables of cohesion, friction angle and groundwater levels are known. The sensitivity of the slope to possible variations in these factors should be determined, as these basic variables are rarely known precisely.

7.4 Earthquake Analysis: Earthquake effects may be included in the general analysis by applying a horizontal acceleration to the weight of the sliding mass and recomputing the factor of safety equation. This is in fact a gross over-simplification of what actually occurs when the dynamic waves of earthquake accelerations act on a slope, but methods of taking this into account are usually beyond the scope of general slope analyses.

8.0 WEDGE FAILURE

Wedge failures commonly occur in practice. A clean cut wedge defined by intersecting joints may form, or the wedge may consist of one well defined surface together with another poorly defined surface, composed perhaps of random rock joints with some failures through intact rock.

For a wedge failure to occur, the inclination of the line of intersection of intersecting planes in the rock must be shallower than the face angle in the slope, and steeper than the friction angle, as for plane failures. The geometry of the situation is best assessed with the use of stereographic projections, which give reasonably simple solutions. A cardboard model of the slope can be constructed, and will help greatly in visualising the effects of intersecting planes forming wedges. The friction angles on the two planes may differ, and additional complicating factors such as tension crack development, water pressure etc. should be taken into account. Several methods for the solution of the wedge problem are available in the literature; three good ones are contained in the Quarterly Journal of Engineering Geology, Vol. 6 No. 1, 1973. The methods used for the solution are too lengthy to present here.

9.0 CIRCULAR FAILURE

An approximately circular failure surface may develop in certain conditions. This may be in soft rocks, high slopes in harder rocks where no unfavourable discontinuities exist, or where the rock is broken up by a large number of discontinuities at varying angles, so that individual particles are very small in relation to the size of the slope.

9.1 Slope Analysis: Analysis of such slopes can be carried out using standard soil mechanics techniques, or standard charts. Suitable parameters should be chosen for failure through intact soft rocks, or for failure through the joint system. These parameters should be checked by back analysis of other failures in similar material - in fact this may be the only economic way of determining the strength of the rock insitu.

Failures in this mode may be very large; on the north-east corner of Little Barrier Island near Auckland in jointed andesite lava flows, a failure occurred back to a 1400 ft high cliff, with rock debris at the toe of the slide being thrust 2000 ft out to sea. The debris covers some 120 acres, and has billowed and thrust up to 345 ft above sea level. Individual rock blocks in the debris are up to 50 ft across. In this case the slope would be undercut by the sea. Failure occurred approximately 2000 years ago, and may have been triggered by an earthquake or exceptional rains.

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THE STABILITY OF SLOPES IN TERTIARY SEDIMENTARY ROCKS OF NEW ZEALAND

I.R. Brown

1.0 INTRODUCTION

Much of New Zealand is composed of relatively soft Tertiary sedimentary rocks (Fig. 1). In general they are consolidated but not to the extent that recrystallization has taken place. A variety of lithologies are present; mudstones, siltstones, sandstones, conglomerates and limestones, mainly deposited in a marine environment, with fine grained rocks predominating.

In both North and South Islands landslides are common in Tertiary sedimentary rocks with natural slopes. Many of these may have occurred at a time when climatic conditions were different from those of today. Such landslides directly affect engineering works such as roads, railways and housing subdivisions, and deserve careful consideration. Their study also gives an indication of the possible behaviour of cut slopes.

2.0 PHYSICAL AND MECHANICAL PROPERTIES OF TERTIARY SEDIMENTARY ROCK

The Tertiary sediments were deposited as loose sands or muds. With increased depositional loading pore volume decreased and strength increased. The finer grained materials, siltstones and mudstones, derive their strength from diagenetic processes. Sands have larger spaces between grains and if intermixed with fine grained interstitial materials, cement bonds are formed around and between grains. At high overburden pressures 'welding' of individual grains may occur. With unloading and weathering the reverse occurs.

Duncan <u>et al</u>. (1968) correlated void index* with lithological types, compressive strength, modulus of elasticity and swelling strain. Tertiary sedimentary rocks have high void index, low strength and modulus of elasticity and high swelling strains. Older sedimentary rocks with greater diagenetic modification have a lower void index.

Montmorillonite, present in many, if not most, fine grained Tertiary sedimentary rocks, is considered to be an important factor in slope failure. Unfortunately little quantitative data are available on the relation of such expansive clay minerals to properties such as swelling strains and rock strength.

2.1 LABORATORY TESTING

Unconfined compression tests on Waitemata Group sandstones and siltstones (Brown 1974) are related to the rock classification system of Deere (1968) in Figure 2. Siltstones and sandstones are not distinguished although the latter tend to have a greater range in strength from relatively uncemented to concretionary layers. All points plotted lie within the range labelled E 'very low strength rock' and most samples have a low modulus ratio. Strength increases with decrease in sample moisture

* void index = $\dfrac{\text{weight of water absorbed}}{\text{weight of dry rock}}$

FIGURE 1 New Zealand : Tertiary Sedimentary Rock
- after Grindley et al. (1961)

content.

Undrained triaxial tests with pore water pressure measurements are difficult to perform on siltstones because of their low permeability (in the order of 10^{-10}cm/sec). There is likely to be a considerable range of effective stress parameters, as indicat--ed by the range of unconfined compressive strengths. It may also be difficult to collect three identical samples for testing. The low range of confining pressures available in relation to the compressive strength means that small variations in strength between individual samples can cancel out the effect of increasing confining pressure. This may be overcome by the use of stage testing (Hobbs 1966; Moretto & Bolognesi 1970).

Direct shear tests using irregular samples of either intact or discontinuous rock with low strength are now possible in the laboratory (Martin and Millar, 1974). Limitations in their use result from the tendency of samples to dry while being cast in a cement mould, so influencing strength parameters. Pore pressures along the shear surface cannot normally be measured, although methods for doing so are being developed (Goodman & Ohnishi 1973). Shear testing of Tertiary siltstones indicates a change from elastic to plastic behaviour with increasing shear displacement (Fig. 3). Dilation occurs on shearing, and contributes significantly to the shear strength of the discontinuity for tests with low normal stress.

Permeability testing of core samples may be carried out in conjunction with triaxial testing in the laboratory but low permeabilities are difficult to measure. With interbedded sandstone and siltstone sequences permeabilities range from 10^{-6} to 10^{-10}cm/sec.

2.2. FIELD TESTING AND INSTRUMENTATION

Conventional drilling techniques may lead to unacceptable core losses in low strength material. Consideration should therefore be given to the use of at least one large diameter hole that can be used for visual inspection of failed zones or potential failure zones in areas where cut slopes are intended.

Down hole techniques available for mechanical testing, such as pressuremeters and bore hole hydraulic jacks, directly measure deformation properties and within certain limitations may also be used to determine strength parameters. In situ permeability tests may also be useful in some situations.

Piezometer and slope inclinometer installations are necessary when investigating an existing landslide, and desirable for monitoring the performance of a cut slope.

2.3 ROCK MASS PROPERTIES

The validity of using laboratory testing data in a slope stability analysis is dependent on a number of factors. Rock properties, particularly those associated with failure, are not fundamental material constants but experimental values, and there is an obvious danger in testing a small number of representative samples and

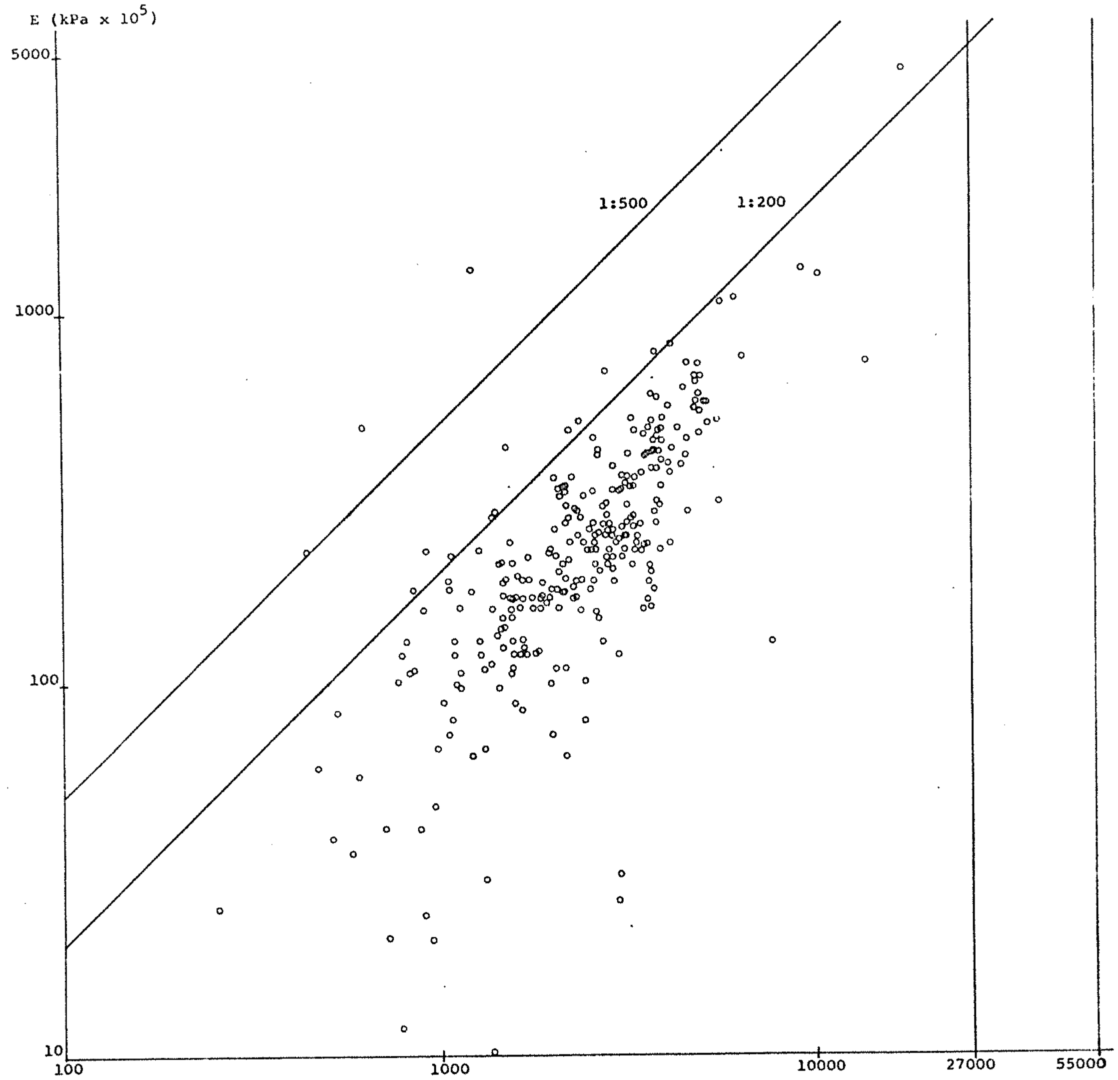

FIGURE 2 Rock Classification, Waitemata Group (Auckland)

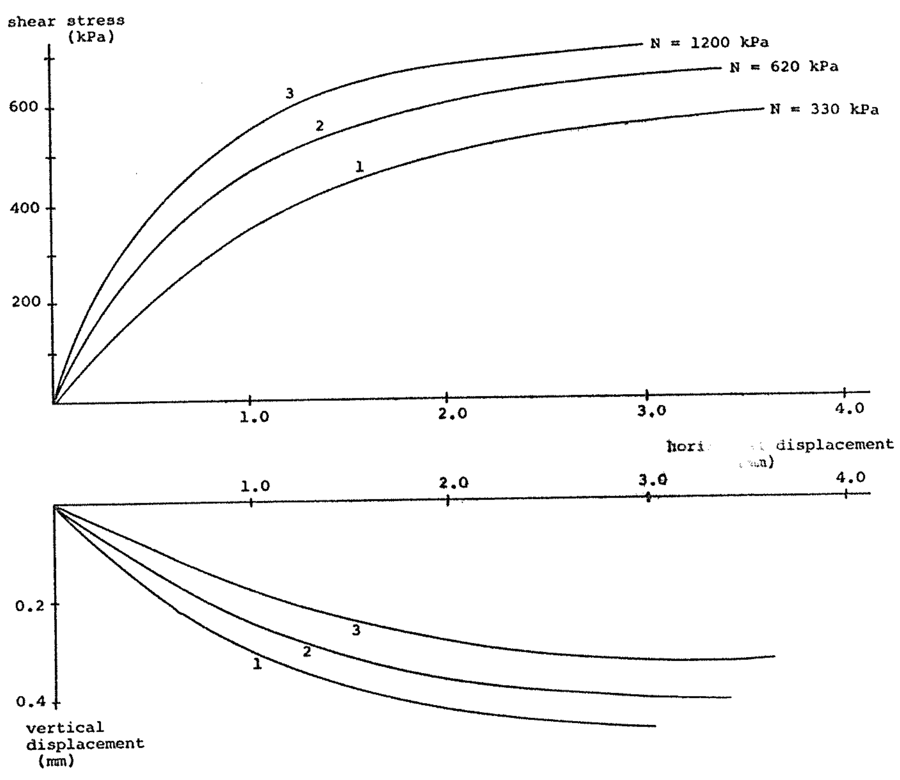

FIGURE 3 Laboratory Shear Test Results - Siltstones

designing a slope on the basis of these tests.

An engineering geological assessment of an area is an essential part of a slope stability study. Detailed mapping to show lithology and structure (bedding, folds, faults, joints) may indicate likely modes of failure. It is necessary to determine the origin of discontinuities; those formed in shear are likely to be at residual strength while those that are tension features will have higher in situ strength.

Figure 4 shows a detailed engineering geological map of Karaka Bay, Auckland. Aerial photographic interpretation delineated areas of past landslide movement within an interbedded sandstone/siltstone sequence. Sliding seems to be controlled by both the gentle dip of bedding towards the slope, and to a lesser extent by faults with an east-west trend.

3.0 GEOLOGICAL INVESTIGATIONS OF LANDSLIDES IN TERTIARY SEDIMENTARY ROCK - A REVIEW

Benson (1940, 1946), one of the first geologists in New Zealand to consider the problems of landslides and their effect on engineering structures, described landslides in both Tertiary sedimentary and volcanic rocks of the Dunedin area.

An engineering geological study of large scale landslides in the Kilmog Hill - Seacliff area north of Dunedin was accompanied by a map showing the widespread presence of ancient and recent landslides within the Burnside and Abbotsford Mudstones (Stout 1971a). Small landslides occur within the debris of larger landslides, and it was suggested that an area of about 49 km^2 had been involved in a massive failure. Montmorillonite "occurs in abundance within both the Abbotsford and Burnside Mudstones" and it was inferred that the presence of montmorillonite has influenced landslide development. Crozier (1969) also reported "significant amounts" of montmorillonite within the Abbotsford Mudstone which has a "high capacity for volume change relating to variations in its temperature and moisture content".

Young (1968) briefly described how stability problems in severely crushed beds of calcareous siltstone influenced the construction of the Paringa-Haast section of State Highway 6.

The stability of the hillside at Tahunanui was reviewed by Falconer (1963). This is the site of an old deep landslide with intermittent, local creeping movements related to periods of intense rainfall. The area cted is 800 m long and extends 540 m from hill crest to the sea. It contains about 40 hectares of residential development. The ground surface has a succession of terraces and scarps considered to be characteristic of rotational landslides. Falconer advanced the hypothesis that the process of internal weathering along fissures causes expansion of the material into the fissure itself. Once further entry of water into the fissure has been blocked off, the strength of the weak fissured material may rise again "because the available locked up water is being spread in a thickening layer". Hence the intermittent movements.

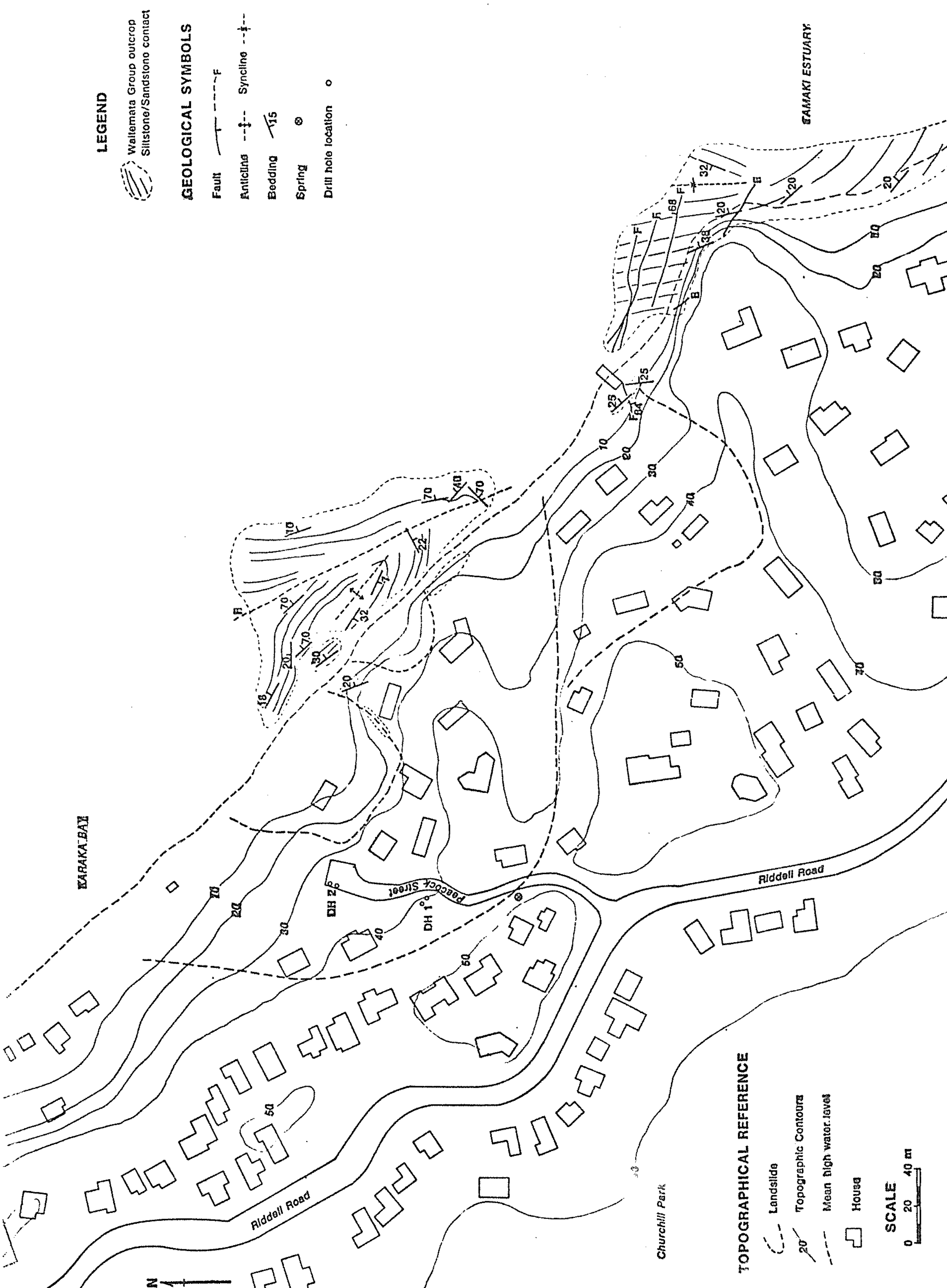

FIGURE 4 Karaka Bay Slope Stability Study Engineering Geological Map

About 1.5 km south of Utiku, both the North Island Main Trunk Railway and State Highway No. 1 have been affected by movement of an extensive landslide (Ker 1970). Surface movement and rainfall have been measured over a period of four years. Utiku Sandstone conformably overlies Taihape Siltstone with an apparant gradational contact. Geological evidence within the district shows that large-scale landslides develop along the strike of this impermeable contact. During long dry periods, non-indurated siltstones and sandstones undergo some shrinkage and physical breakdown. Where this process is facilitated at depth by the presence of open fractures, slip surfaces develop. Fracturing is considered to be more intensely developed adjacent to areas of movement.

Stout (1971b) discovered a clay layer, "a few millimeters thick at the most", parallel to bedding in the Utiku landslide. He considered that the clay layer "has definitely been a controlling parameter in location and movement of the Utiku slip". Joint patterns seem to be related to the marginal areas adjacent to ancient landslides, rather than regional stress release by erosional removal. X-ray analyses of a clay sample taken from a slip surface showed relatively high concentrations of montmorill--onite. From interpretation of drill hole and movement data, the basal surface of movement seems to be a composite surface, planar on the eastern two thirds and rotational on the western third. At least part of the mass consists of relatively undisturbed siltstone. The firm blocks in the slide mass are probably not saturated except in joints or cracks, reflecting the very tight, relatively impermeable siltstone. Material between these blocks is broken, crushed, and saturated siltstone, and in some places pug and organic debris are present near the surface cracks. Piezometer measurements have been correlated with rates of landslide movement. Groundwater movement through the slide mass is believed to be largely through interconnecting cracks and broken zones.

The North Island Main Trunk Railway is also affected by landsliding north of Waimiha (Riddolls 1972), where a number of landslides have caused displacements of the track. The main lithology is grey mudstone commonly alternating with "silt or sand layers" up to 150 mm thick. In the upper slopes sandstone is more abundant and forms beds up to 3.5 m thick. The mudstone is moderately weak where unweathered, but near the surface is very weak and closely fractured by irregular joints. The beds in the nearby stable ground dip in the opposite direction to the landslides. It has been shown from recent investigations that the landslides are formed mainly in debris derived from the bedrock (B.W. Riddolls, pers. comm.).

Large landslides in the Lower Wanganui valley seem to be tectonically controlled, occurring roughly parallel to swarms or zones of active fault traces (Ker 1973). In the thick massive siltstone sequences faults may be indicated by joint swarms.

The problems of landsliding on the east coast of the North Island have been largely studies by organisations concerned with soil conservation although some geological work has been associated with the soil conservation programmes. Bishop (1968) prepared a lithostratigraphic map of a small area at Waerengaokuri, near Gisborne, showing siltstones, sandstones, limestones and highly plastic blue-grey bentonitic mudstones. At one place the bentonitic mudstone is diapirically extruded along a fault trace. Montmorillonite is present in Tertiary mudstone in the Waipoa River catchment (Claridge 1960). Movement of landslides in these beds causes grinding of the montmorillonite rich mudstone, the ground material then fills cracks and, on wetting, swells and "lubricates" the rock mass. Claridge referred to these rocks as crushed, perhaps implying some tectonic control of mass movement. O'Byrne (1967) confirmed that the erodable Tertiary rocks in this area are either unconsolidated or finely jointed.

Stout (1971c, 1971d), in a brief discussion of roading problems along State Highway 35 north of Gisborne and various soil conservation measures around Gisborne, concluded that an understanding of the clay content and clay mineralogy is critical in an evaluation of landsliding in this area. Montmorillonite was present in all samples analysed. Kaolinite was found in two landslides, although only on sheared surfaces and not in the undisturbed mudstone.

In the Tertiary Waitemata Group around Auckland, Gilmour (1963) reported landslides at Northcote, Castor Bay, Clifton Road, Seacliffe Avenue, Glendowie, Point Chevalier and Hillsborough. The generally low angle of dip of the Waitemata Group "makes these beds very stable, but in certain localities where warping and faulting has taken place, slipping and slumping is very common on steep slopes and cliff faces".

At Muriwai on August 27, 1965 after 230 mm of rainfall in 3 days, a large land--slide resulted in the death of two people. Field observations of later movements "indicated that the material responded to the disturbance almost instantaneously by spontaneous liquifaction". A pronounced scarp was reported "partly lithologically controlled, being associated with the sandstone/siltstone junction".

4.0 METHODS OF SLOPE STABILITY ANALYSIS

There are three ways of designing slopes for soft rock

- the limit equilibrium approach based on the strength characteristics of the rock mass
- the stress analysis approach based on both deformation and strength characteristics of the rock mass
- the empirical approach based entirely on experience.

4.1 LIMIT EQUILIBRIUM METHODS

In the limit equilibrium approach, geological investigations are followed by an evaluation of the strength characteristics of both intact rock and discontinuities which may be involved in failure. In soft rocks the most critical failure mode may be shear or tensile failure in the intact mass. If the shear strength of the rock is low and not controlled by discontinuities, conventional soil mechanics methods may be used to determine the location and shape of the most critical rupture surface.

When the shear strength is wholly controlled by discontinuities, it is most likely that the failure surfaces will pass along the discontinuities, the orientations of which are determined by geological investigations. The stability analysis then becomes the same as that for hard rocks, a complex vector problem that may be solved by the use of stereographic projection techniques (John 1968; Londe _et al_. 1968).

In unjointed Tertiary sedimentary rock with gentle dip of bedding towards the slope surface, the failure surface may be best approximated by a planar surface coincident with bedding planes along part of the length, the remaining part in the form of a logarithmic spiral passing through intact rock.

4.2 STRESS ANALYSIS METHODS

Because of the limitations of limit equilibrium methods in the design of large excavations in rock, stress analysis methods were developed. These include the field measurement of residual rock stresses, and field or laboratory tests to determine the stress-strain and strength characteristics of intact rock and discontinuities. Stress-strain behaviour is likely to be non-linear even in the range in which rock is loaded by an excavation.

Finite element methods may be used to perform slope stability analysis. Methods of measurement of residual stress are still being developed and there is considerable difficulty in obtaining reliable data. It is dangerous to use analytical results for determination of residual stresses as they are closely related to the tectonic history of an area, and such results may be several orders of magnitude in error.

4.3 EMPIRICAL METHODS

With an empirical approach to slope stability, engineering geological information is used to classify the problem in a behavioural grouping. This then allows the selection of a design based on appropriate previous experience.

4.4 DYNAMIC ANALYSIS

It is likely that some of the large landslides in Tertiary sedimentary rock occurred during earthquakes such as that at Inangahua in 1968.

High pore water pressures can develop in fine grained materials during earthquakes and may act as a triggering mechanism for slope failure. Methods of dynamic analysis are available but should be used with due consideration of the assumptions involved.

4.5 COMMENTS ON DESIGN METHODS

The selection of design methods is related to the scale of the slope stability problem. Empirical methods are by nature low cost and may be acceptable in an area where the consequences of slope failure are not great. A limit equilibrium analysis with varying complexity, related to the size of the problem is most commonly used. Stress analysis methods require expensive site investigation techniques and are likely to be used only for the design of large excavations. Stability analyses are at best a two dimensional approximation of a three dimensional problem.

Factors of safety may be expressed in terms of the balance of overturning and resisting forces. In some cases the ratio of actual strength to developed stress may be used. In view of the indeterminancy of a number of the factors used in a slope stability analysis, combinations of variables that would cause failure are assessed leading to a probability assessment of failure (McMahon 1971).

Checks that the behaviour of rock conforms to original expectations should continue through construction, and design modifications carried out as required.

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<u>Chairman:</u> *Mr. J.P. Blakeley*

<u>Section 7:</u> ROCK SLOPES

<u>PRESENTATION AND DISCUSSION</u>

<u>*Dr. Martin*</u> introduced the paper, "The Stability of Slopes in Weathered and Jointed Rock", by commenting that it was more specifically related to the situation encountered, for example, in Wellington where there were deep and highly jointed weathered rock profiles associated with the greywackes and argillites. The situation was typified by the Shell Gully cutting. Such weathering profiles were bounded by completely weathered rock and residual soils near the surface and slightly weathered or fresh rock at depth. For visual classification purposes the scheme shown in Table 1, seemed to be gaining wide acceptance and Dr. Martin felt that people should be familiar with the various classifications set out in the table. Dr. Martin referred specifically to moderately and highly weathered rock where the problem of the transition in behaviour from a soil to a rock made quantitative stability analyses very difficult. For such rock both the strength of the rock mass and the mechanism of failure could be governed by both the nature and strength of joint patterns and the strength of the intact rock fragments.

The first stage of a stability assessment was a general characterisation of the rock mass of the whole, such as visual classification, description of the rock structure, joint patterns etc. The extent of the investigation depended on the economic consideration and the nature of the problem being considered. Having characterised the rock mass, the design problem then arose. The engineer had to make a decision as to the most suitable profile for proposed cuttings and the development. He referred particularly to the paper by Deere and Patton, and broadly outlined the sections on Design as set out in his own paper.

Dr. Martin said that the problems that were inherent in highly and moderately weathered rock slopes had motivated research in the University of Auckland in the last few years sponsored by the National Roads Board. The first phase of the project was the report on Rock Slope Stability Problems, presented by Richards, Millar and Martin, (noted in the reference list) and the development of rock joint testing equipment. The next stage would be to look more closely at specific case histories in the field to try to understand the various mechanisms of failure that occurred in the highly weathered jointed rocks.

<u>*Mr. Millar*</u> discussed briefly methods of determining joint strength characteristics of rock and described the nature of joint strength variation with weathering. In slope stability problems, direct shear testing was considered the most useful method for the determination of strength parameters in that there was an enforced shear plane similar to that which would occur in the slope. A shear box constructed at Auckland University was described. Samples are set in plaster and fitted into a mould. A normal load is applied by a vertical ram and shearing force applied by horizontal rams. The shearing action may be reversed to enable large displacements and after a number of reversals a residual strength may be obtained. Features of the machine include a maximum sample area of approximately 150 mm^2, loads up to 200 kN, high stiffness, manual or automatic controls, displacement controlled horizontal shearing and measurement of pressures and displacements using transducers and recording equipment. To date tests had been conducted on silt stones from Mangaweka, papa from Poro-O-Tarao Auckland Waitemata sandstone and greywackes and argillite samples from Shell Gully in Wellington. Rock joints had been tested using a normal load stage testing technique. The rock was sheared at a low normal stress and the test was stopped immediately peak strength was obtained. The sample was then returned to its original position and the normal load was approximately doubled and the test repeated. Theory indicated that the strength obtained at the next increment of normal stress should not be greatly affected by the previous test. This was because at low normal stresses only the steep weak and narrowest irregularities on the surface were sheared off and these do not significantly influence the shear strength at higher normal loads. Mr. Millar showed slides of an argillite sample after failure had occurred and a grade 3 greywacke rock sample; a typical result from a joint strength test; typical results for grade 4 rock; failure envelopes for different weathering grades of greywacke.

He said that the overlap from grades 2 and 3 could be explained by the fact that the difference between the two weathering grades was mainly in the depth of weathering that had occurred within a block of rock. The strength along the joint surface was not markedly affected during this stage of weathering. Also there were marked differences in the irregularities of different surfaces of samples which resulted in a wide band of results.

He then referred to design problems, which he illustrated with reference to the Shell Gully cutting. Two methods of analysis were noted. The first was the one-dimensional analysis presented in the reference by Hoek. The general failure equations for this analysis were in terms of the ratio $\gamma H/c$, the slope angle and the angle of the joint failure plane. Charts were available in which these results could be inserted and included modifications for drainage and tension cracks. The second method was the wedge failure analysis, using a stereographic technique. Such analyses were conducted for small bench failures which had already occurred on the Shell Gully slopes. They showed that for vertical benches assuming a drained slope a minimum angle of frictional resistance of 45-52° would have been required for the benches to be stable. In an undrained slope the minimum angle of frictional resistance necessary was 64°. Test results had indicated joint friction angles at approximately 50° of low normal stresses. An alternative design of a 41° slope with a limited number of benches to prevent scouring was suggested. This would have greatly reduced the number of small scale failures which have occurred. There was a lot of controversy over the use of benches. The factors in favour of benches were that they caught debris coming down the slope, restricted failures to small failures, intercepted drainage and stopped gullying. Disadvantages included the non-uniform stress distribution that was generated in the slope promoting stress relief fractures, ravelling and allowing water to enter the slope and promoting many of the small scale failures that they were designed to restrict.

Mr. Riley said his paper, "Stability of Hard Rock Slopes", gave some analytic methods which might be used to analyse a small rock slope where the shear strength along the discontinuities, i.e. the joints, shears, faults, could play a major part in the failure. The paper was largely a précise of lectures at the Engineers Rock Mechanics Course at the Imperial College in London. The procedures were more fully described in a book by Hoek and Bray called "Rock Slope Engineering". From the type of analysis described, decisions could be made on what could be done with the slope, whether it was flattened or stabilised by drainage, rock bolts or cables. Alternatively, the risk of failure could be accepted and the cost of cleaning up afterwards worked out. Mr. Riley said the collection of geological data was vital to any slope analysis and for a rock slope analysis particular attention should be paid to faults in shear zones as these were very extensive laterally and often contained gouge materials which had very low strength properties. Joints were important, and in general had higher friction angles, so failures on them tended to be smaller. Bedding planes were also very important and these were laterally very extensive. The most important information was often very difficult to find as weak zones in the rock face weathered more readily and became hidden by the product of the weathering. For this reason, a freshly cut rock slope was much better for the collection of data than a weathered face. The determination of shear strength was an important factor. Shear box testing could be used to produce a failure envelope for any particular rock. For a rock slope design a simplification was usually adopted and this was to assume that the failure envelope was linear over the range of stresses encountered beneath the slope.

Mr. Riley said the only really satisfactory method of finding an applicable apparent cohesion was to back-analyse an existing failure in similar material on a similar discontinuity set. This would give a range of c and ϕ values which could be applied to the slope at the factor of safety of 1 which would have occurred at failure. It was suggested that the friction value angles obtained from the shear box test and modified by the roughness of the plane be used and the corresponding cohesion obtained from the back analysis to give design parameters for the slope. Analysis of a failed slope was an extremely useful tool as factors were taken into account which could not be measured in laboratory tests.

It was often difficult to arrive at the mode of failure. It might be obvious from the data collected that a plane failure could occur; otherwise a detailed analysis might be needed. Many mechanisms were possible and sometimes very complex ones were suggested. However, in many cases they could be simplified to one of the methods used in his paper. An analysis of which type of failure could occur would be aided by stereographic projections. These allowed the collection of data to be looked at systematically. With regard to groundwater conditions, water was the most common trigger causing slope failures. It had the effect of applying pressure on the failure surface which reduced the normal stress and reduced the frictional resistance. It increased the driving force through water pressure acting in the tension cracks. Mr. Riley said some work had been done on tension cracks in homogeneous rock slopes to give a theoretical depth and the formulae were shown in his paper. They were arrived at by differentiating the factor of safety equation with respect to the tension crack depth and equating the result to zero for maximum and minimum values. Further theoretical work had been done on the position a tension crack would occupy when formed in wet weather when the slope was saturated which showed that it would occur at the crest of the slope. In fact, tension cracks often occurred behind the crest of the slope and this was the reason for the assumption of tension cracks forming in dry weather. These cracks could be extremely large. Some he had seen in Cornwall, England, were 5' across x 30' deep up to 30' behind the crest of a 65° slope, 250' high. This slope had stood in this condition with the tension cracks for 30 years. Its eventual failure was inevitable, as measurements across the crack showed that creep was continual, and speeded up during rainfall. A slow acceleration could be found in the rate of creep. The only method for earthquake analysis in a rock slope was to provide a horizontal acceleration at the chosen value. Mr. Riley said that wedge failures on two intersecting discontinuities commonly occurred on a small scale. They were much rarer on a large scale, but they did occur and their effects could be very severe. The wedge could extend well back behind the crest of the slope. They were only likely to occur in a Wellington greywacke where two intersecting faults were in a slope. This was relatively unusual as the faults usually tended to follow a fairly dominant structural pattern. Circular failures could occur in small rock slopes where weathering or jointing had produced material properties which did not vary much in direction. Prepared charts allowed quick analysis of these. Where the slope was very large circular failure could sometimes occur.

Mr. Brown in dealing with his paper "The Stability of Slopes in Tertiary Sedimentary Rocks of New Zealand", showed a map of N.Z. giving the distribution of tertiary sedimentary rock. These rocks were found in most parts of the country and were particularly abundant in the central and eastern part of the North Island. Many geotechnical problems could arise in such areas associated with residual soils, such as soil shrinkage under house foundations in the Auckland area, slope stability in residual soils, slope movement in colluvium soil erosion in east coast areas of the North Island giving rise to serious soil conservation problems and large scale landsliding. He asked why should one look at tertiary sedimentary rocks as a separate topic. There was a danger that some engineers had grown to expect tertiary rocks to behave like indurated material when it came to the design of cut slopes and tunnel linings. It was important to remember, he said, that most tertiary rocks including most in N.Z. were very low strength when compared with other sedimentary rocks. For rock material, strength, either in tension or compression, was mainly due to diagenetic bonds or crystal interlocking. With tertiary rock the bonds had not been quite so well developed. The strength of intact rock material was of importance in stability problems as part of the failure surface may be through intact rock, as joint strengths tended to approach intact rock strength. Testing such rocks to produce meaningful results may be quite difficult. Mr. Brown said the samples he had plotted had all been described as unweathered Waitemata group sandstone and siltstone from the Auckland area and it had been difficult to predict such a wide variation in strength by a visual assessment. If three samples had been taken for triaxial testing from the group of aamples with such widely varying unconfined compressive strengths, one would have probably expected to get an inaccurate value of ϕ as a result. One way of overcoming this was a stage testing method, but in practice there was difficulty in controlling the te[illegible] that the sample was not deformed too extensively and if the failure was irregular or it occurred along a number of planes one was not very sure what strength was being measured. There was also the problem that all sedimentary rocks were anisotropic. With core samples one was limited to testing the strength of rock at a direction round 30-40° to the axis of the core. Shear box testing was more flexible when testing anisotropic samples. The sample may be mounted for shearing in the direction of bedding. There had been problems with tertiary sedimentary rocks drying out as the cement mould set. Strength was quite closely related to the moisture content.

With regard to an engineering geological assessment for a slope stability study, Mr. Brown said that tertiary rocks could be difficult to map. Surface exposures may be limited and bedding attitudes difficult to determine, especially when at a low angle. Conventional core drilling may not be of much use in the low strength materials and generally the few percent of core that was lost was quite critical, therefore consideration should be given to a drilled shaft for in situ logging and sampling.

In connection to areas where landslides had occurred, Mr. Brown said it was hard to generalise on the reasons for failure but obviously the excess pore-water pressures were very important. There had been a tendency for geologists working in that sort of material to find montmorillonite and then assume this was what had led to failure. He felt this was only part of the answer and it was necessary to look at the detailed geology of the areas as the local structural details controlled the mode of movement. The type of stability analysis undertaken would depend on the individual details and it was unlikely that the problem in any two areas would be the same.

Dr. Pender opened discussion on Mr. Riley's paper. He said that in looking at slopes around Wellington in the unweathered, slightly weathered or moderately weathered greywacke one was inclined to wonder about the mechanism responsible for the evident stability of the slopes. Some were very steep, thus suggesting that the material had some cohesive properties and yet exposed faces of the rock were so intensely jointed that cohesion was unlikely. The material was so closely and tightly jointed in a more or less irregular manner that it might exhibit similar properties to the low-porosity aggregate described by Rosengren and Jaeger (1). This material had no cohesion as such, but because the low-porosity made interparticle movement very difficult the failure envelope was strongly curved at low normal stresses. The form of the failure envelope could be expressed as follows:-

$$\tau_f = k\,\sigma_n^m \tag{1}$$

where τ_f is the shear stress at failure
σ_n the normal stress on the failure plane
k and m material parameters

Dr. Pender's purpose was to present the results of some preliminary calculations into the effect of this type of failure envelope on the stability of a rock slope. At this stage only rather crude calculations could be presented.

The case of planar failure along AB was investigated as shown in Fig. 1. The calculation of the stability was dependent on the distribution of normal stress along the failure plane AB in contrast to the usual analysis of planar failure in a c, ϕ material. At this stage he had made the simplest assumption possible, namely that the normal stress at any point along AB was determined by the weight of overburden.

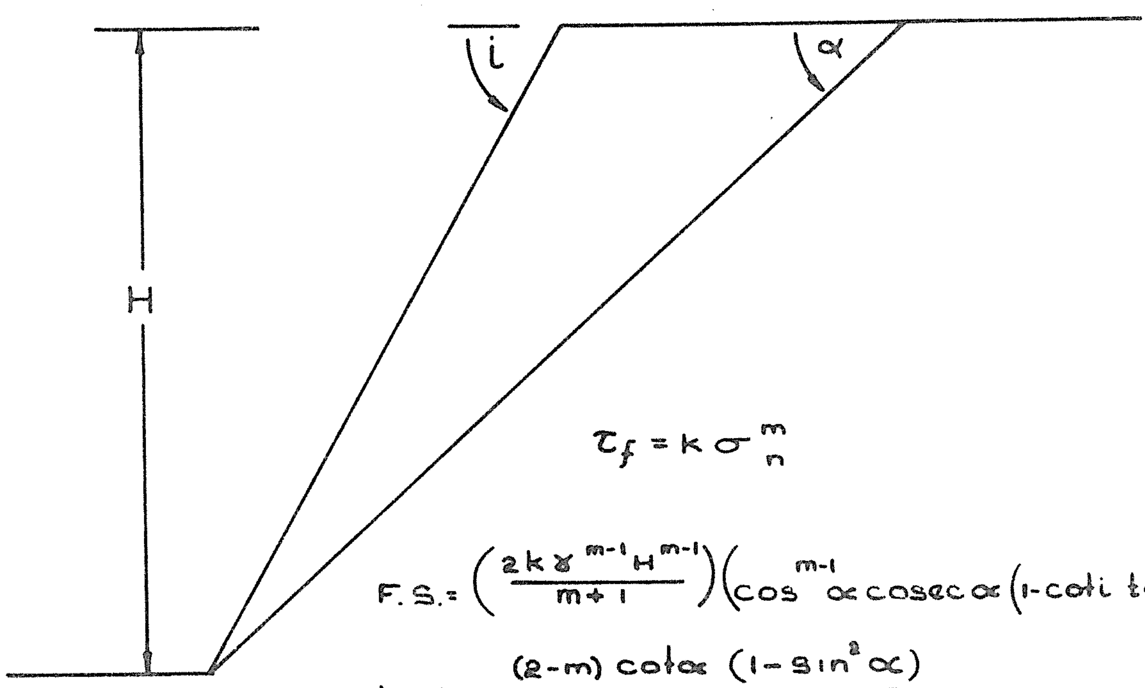
A straightforward analysis yielded the following expression for the factor of safety:

$$F.S. = \left(\frac{2k\,\gamma^{m-1}\,H^{m-1}}{m+1}\right) \quad (\cos^{m-1}\alpha \,\mathrm{cosec}\,\alpha\,(1-\cot i \tan\alpha)^{m-1} \tag{2}$$

The next step was to minimise this with respect to α. Once again straightforward, but slightly tedious work yielded:

$$\tan i = \frac{(2-m)\cot\ (1-\sin^2\alpha)}{(1+(m-2)\ \sin^2\alpha)} \tag{3}$$

Jaeger (2) in his 1971 Rankine Lecutre discussed planar failure in a material with a power law failure envelope. However, his calculations were based on the average normal stress on the failure plane, and although this did not involve any assumption about the distribution of normal stress along AB it did not fully exploit the essential feature, the non-linearity, of the failure envelope given in equation (1).



$$\tau_f = k\,\sigma_n^{\,m}$$

$$F.S. = \left(\frac{2k\gamma^{m-1}H^{m-1}}{m+1}\right)\left(\cos^{m-1}\alpha\,\operatorname{cosec}\alpha\,(1-\cot i\,\tan\alpha)\right)^{m-}$$

$$\tan i = \frac{(2-m)\cot\alpha\,(1-\sin^2\alpha)}{(1+(m-2)\sin^2\alpha)}$$

FIG 1

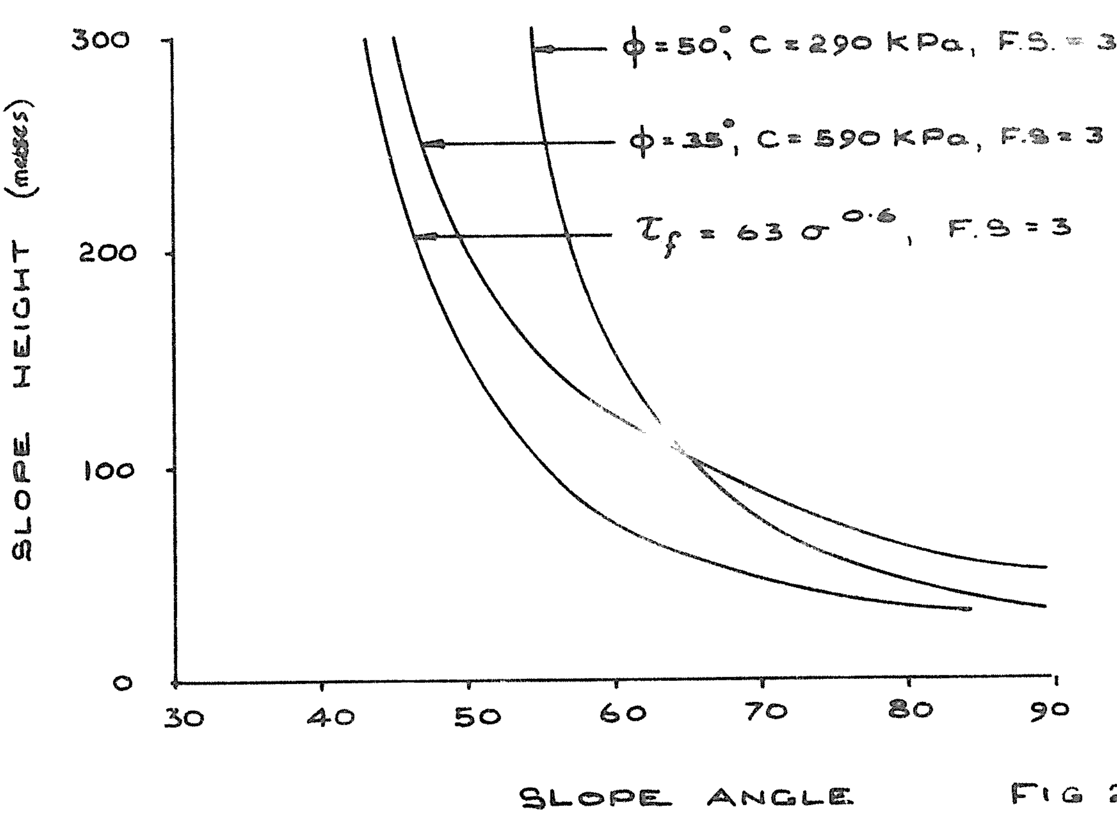


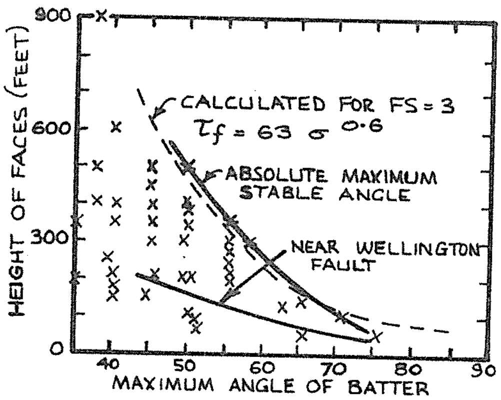
FIG 2

Results of calculations of slope height versus slope angle for a nominal factor of safety of 3 were plotted in Fig. 2 for various values of k and m. Also included were similar curves for comparison with a conventional c, ϕ material.

Grant-Taylor (3) had examined many of the slopes around Wellington and his plot of slope angle versus slope height was reproduced in Fig. 3 along with a calculated curve from equation (2) for a nominal factor of safety of 3. It was clear that the model presented gave a more satisfactory representation of the relation between slope height and angle than the c, ϕ approach .

FIGURE 3

Relationship Between Maximum Stable Angle and Height of Batter in Wellington Greywacke.



These calculations were of a very crude and preliminary nature. The distribution of normal stress along the failure surface had been assumed rather than derived and other shapes of failure surfaces not considered. Nevertheless the results indicated that the model had considerable promise and merited further investigation. Clearly the big difficulty was the determination of the parameters k and m but no more so than for the parameters in a c, ϕ analysis.

References supplied by Dr. Pender:

1. ROSENGREN, K.J. and JAEGER, J.C. "The Mechanical Properties of an Interlocked Low-Porosity Aggregate", Geotechnique Vol. 18, 1968. P. 317-328.

2. JAEGER, J.C. "Friction of Rocks and the Stability of Rock Slopes". Geotechnique Vol. 21, 1971. p. 97-134.

3. GRANT-TAYLOR, T.L. "Stable Angles in Wellington Greywacke". N.Z. Engineering, Vol. 19, 1964. p. 129-130.

Dr. Martin siad it was important to recognise that failure envelopes for jointed rock were curved and particularly if making use of analysis based on apparent linear parameters the stress parameters used for the analysis must be appropriate for the range of stresses of the problem. He felt there would be a new generation of slope stability charts coming out based on non-linear failure envelopes for weathered rock. Some research had been done in England and had characterised the curved failure envelopes by two parameters which were obtained for the rock. One of the parameters was the Schmidt hardness to characterise the extent of weathering of the joint surface and the other parameter was the extent of roughness of the joint surface. These parameters were then inserted in an analytical expression which had given them a characteristic non-linear failure envelope which could then be used for a more generalised stability analysis ...lving plane and failure surfaces.

Mr. Mitchell referred to the comment that it had been stated by an engineer that the Karaka Bay area was safe for construction. He said that much of it probably was not but he asked where Mr. Brown would place a line between a section he would build on and one he would not. This was the type of information that would be required in any land use zoning regulation.

Mr. Brown said the problem had looked after itself because the area had been built on. Where houses had been demolished was where it was proposed to re-build and he felt it would be quite unwise to do so.

Mr. Carryer said that stereographic projections were a most useful method of presenting data. Their use and the interpretation of the data must be tempered by field interpretation of the importance of the partings present in the rock, particularly in jointed rock. For example, expanding on what had been said by Mr. Riley, in any fault system a large number of parallel joints may be developed at an angle to the fault. A stereographic projection would, because of the large number of joints present, tend to hide the representation of the fault that formed them. Very often such a fault would have characteristics which made it a stability problem much larger in scale than the problems related to the jointing. Thus, unless field interpretations were considered of primary importance, very often a misleading conclusion could be reached from stereographic projections.

Mr. Riley said that faults had dominating influence and should be noted. When the data was plotted on the stereographic projection, if a plane was found which looked bad for the slope, then the initial readings could be referred to and the actual characteristics of that joint more closely defined.

Mr. Depledge on the question of terminology, said he would call these tertiary soft rocks, 'soils' in geotechnical language. He said that more importantly there were two papers, one which referred to 'weathered and jointed rock' and the other to 'hard rock'. This might give the impression that hard rock had no joints in it and he had not seen a hard rock without joints. He endorsed Mr. Miller's denunciation of small scale benches which he felt allowed infiltration of water which added to the problem of slope instability. He found one of their major uses was in showing in small scale what could happen in large scale.

Mr. Miller said the answer to the question about hard rock was that Mr. Riley's paper did show that most of the problems of hard rock were in the joints and discontinuities. Generally with increased weathering one tended to get increased jointing because then the rock was generally nearer the surface and more fractured.

Mr. Shaw said that Mr. Riley mentioned that in the compiling of data, one required the orientation of core as it was a major problem. He said this was a fairly common practice but was a costly business and he wondered whether the problem was one of cost or difficulty of operation.

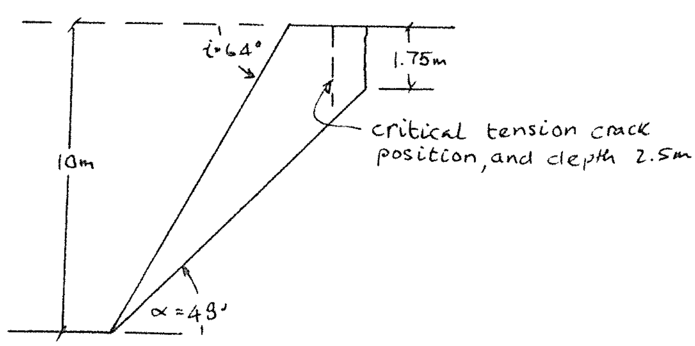
Mr. Riley said it was probably a bit of both. He said there was a core orienter in use but it involved putting a probe down a bore hole with fingers out at the end which matched up with the end of the core stub left in the end of the hole. After the core run, the particular core is brought out and the fingers are matched on to the end of the core again and so orient the core in that way. This required to be done with each core run. Should the rock be poor quality or any rotation occur in the core barrel then the results were meaningless. There were other methods, one developed at Mt. Isa, in defining the rock mechanics and geological structure of the ore body there where they used an inclined drill hole and sent a small phial of paint down on the end of the drill rods and broke it against the end of the core stub. The paint had run down in a vertical direction and when the core was brought up, that was the line. Bore hole T.V. cameras had been tried but most people who had tried them had found that their success rate was 30% or less. They gave a lot of trouble. Water down the holes did not help the cameras. If they worked they would be ideal. The only real answer, and which was reasonably economic and easy, was to drill a hole of 2' diameter to inspect orientation. This had been done at Shell Gully.

Mr. Hancox in referring to orientation of core said that during the investigations for the Tongariro power development a simple device consisting of a cylinder containing gelatine and a ball-bearing had been used with some success to orientate angle drill holes (see Geomechanics News, June 1974).

A participant stated, with regard to the slide at Shell Gully, with its benches or no benches, there was a programme of planting. He would like some comment on this. In response to an enquiry about planting on Shell Gully slopes, *Mr. Millar* said the intention was to put topsoil on the benches and plant them but he felt the slope was going to even out at 41° before they had a chance to do it.

Mr. Evans said that root pressures could be enormous and vegetation was really not desirable in rock slopes. The answer might be in the selection of planting.

Dr. Pender then described some work on which he and Mr. Riddolls had been engaged. This was the back analysis of a two dimensional wedge failure in a zone of highly/completely weathered greywacke in Lower Hutt. There was a quantity of data available on strength parameters for these materials derived from laboratory triaxial testing on undisturbed core. Back analysis of observed failures provided a useful means of checking whether the laboratory strength parameters represented field behaviour. The particular failure analysed was chosen because there was no obvious influence from any geological feature or defect. Thus it was sufficient to analyse the stability using procedures for a homogeneous soil. The profile of the slope is shown below:



In elevation the width of the failed area was approximately equal to the height of the slope.

In a homogeneous 1/2 to 1 slope, the most critical failure surface was usually circular. However, the location of this critical circle, Hoek (1), cut the top surface of the slope some distance from the scarp of the present failure surface. Thus the failure was back analysed as planar rather than circular. From the geometry of the failed surface it was possible to determine the angle of the failure plane, in this case 48°. For a planar failure in a homogeneous soil the slope angle, failure plane angle and friction angle were related by:

$$\alpha = \frac{1}{2}(i + \phi) \qquad (1)$$

where i and α are defined in the figure and ϕ is the friction angle.
Substitution of the values for i and α gave a friction angle of 32°. This was not an unreasonable value for greywacke at the transition from grade IV to grade V, although values several degrees higher would be equally acceptable. The appropriate value of the cohesion was a little more difficult to decide. One line of attack was to consider the depth of the tension crack. Terzaghi (2) showed that for an active state of stress in the top of a slope the maximum depth of tensile stress is given by:

$$Z_o = \frac{2c}{\gamma H} \tan\left(45 + \frac{\phi}{2}\right) \qquad (2)$$

Use of equation (2) gave a lower limit on c because there was no certainty that the failure plane passed through the bottom of the tensile stress region. [illegible]tion of scarp at the top of the failure plane suggested that most of the tension crack was along joint surfaces. Thus the depth of the tension crack was not considered as a reliable indication of the probable cohesion mobilised on the failure surface. The attempt at estimating cohesion was thus based on the critical depth of tension crack for a slope with the present geometry, Hoek and Bray (3). This yielded a value of c/γ 0.69 m, for typical bulk mass density figures (1.84 t/m^3 for partly saturated to 2.16 t/m^3 for saturated grade IV - V) this gave cohesion ranging from 12.4 to 14.6 kPa. These values are acceptable but once again at the low end of the range one expects for such material.

The factors of safety calculated for various conditions with the above values of c and ϕ are tabulated below:

Condition	(t/m^3)	c (kPa)	F.S.
Intact dry slope	1.84	12.4	0.86
Intact slope, saturated by capillary tension	2.16	14.6	1.17
Saturated slope with tension crack	2.16	14.6	1.09
Saturated slope with tension crack full of water	2.16	14.6	0.99
Saturated slope with tension crack full of water and percolation of water along failure plane	2.16	14.6	0.85

Thus the values of c and ϕ adopted are satisfactory to explain the failure under fairly modest assumptions about water in the slope. The case of steady-state seepage was not included because the failed slope forms the middle section of a rather higher benched slope and also because the failure occurred two years ago and well before the present extremely wet winter.

Although it was possible to find c and ϕ values that could account for the slope failure these were at the low end of the range for the material. Also the sensitivity of the calculations to the cohesion value is very evident and it is this parameter which tends to be more variable than ϕ.

References supplied by Dr. Pender:

HOEK, E. "Estimating the Stability of Excavated Slopes in Opencast Mines". Trans. Sect. A. IMM Vol. 79, 1970, P. A109 - A132.

TERZAGHI, K. "Theoretical Soil Mechanics" Wiley, 1943, p. 37.

HOEK, E. and BRAY, J.W. "Rock Slope Engineering". Inst. Mining and Metallurgy , 1974.

Mr. Webley referred to the question of grouting of sedimentary rocks. He asked for comments on its effectiveness as a stabilisation measure and whether a theory could be produced to explain the use of such a practice.

Mr. Riley said it could be quite dangerous in some cases to carry out grouting because of the high pressures induced into the rock, perhaps in the places where they were not wanted. He did not feel grouting could do much good.

Mr. Smith said that grouting of sedimentary soils had been used extensively and successfully overseas. It had been developed originally by British Railways. N.Z. Railways had done considerable grouting north of Auckland which had been successful. It had been used privately in Whangarei. Nobody seemed to know why it worked. Insofar as grouting of rocks was concerned, the pressures could cause bigger problems, in breaking the rock and causing failure.

Mr. Bhula said the Wellington City Council had experience of one place in Karori where there was a high retaining wall holding up a large bank. A tension crack had developed at the rear of the wall some 8-10' away and it had been found that during the winter months the wall tended to move forwards. After using pressure grouting it had been found, by continual inspection of the wall that the movement had been eliminated altogether.

Mr. Yorkat said lime had been used extensively in runways and roadways in the form of a slurry placed in boreholes which then migrated laterally and stabilised the soil. He wondered if there may be some application to stabilise some rock slopes with this method.

Mr. Smith said he had not seen lime used in connection with stabilisation of pavements, but certainly it had been successful with slopes. Reverting to the question of grouting, he said that

D.J. Ayres had written several papers on cement grouting of slips, for the Institution of Civil Engineers.

Dr. Hawley said that since he had produced a paper on terminology, a subtle but serious difference in usage of terms had arisen between the soil mechanicians and the geologists. For the soil mechanician, a term such as 'land-slide' referred to an event; to the geologist a landslide appeared to be a place. This was the stuff of which expensive court cases were made. He said that Table 1 in his paper was a list of types of event, not places, not types of material, not situations, so they could not list such things as dangerous ground or fossil landslides.

The Chairman thanked the four speakers of the session for the contribution they had made and all those who had taken part in the discussion.

<u>Chairman:</u> *Mr. D.K. Taylor*

<u>Section 8:</u> <u>SUMMARY AND CLOSURE</u>

At the final session of the Symposium, the Chairman asked the audience to indicate by show of hands their answers to the questions he had posed at the opening session:-

(a) Was the cost of landslips to the community such that there was a serious problem.
(Two-thirds thought there was).

<u>Mr. Toynbee</u> said he was certain the answer was 'yes' and it was clear that it was a serious problem, not only in Nelson, but elsewhere. Nelson City had had the problem for many years because it was an elongated city with some relatively unstable hills limiting development. The alternative was to go on to the hills and perpetuate the problem. This must be attempted realistically otherwise the City would spread out on to good farm land and Nelson Province had very little available. Commenting on Mr. Gill's suggestion, that there could be some local fund to help people who had troubles with slips not covered by the Earthquake and War Damage Commission, he felt if this eventuated it could well be administered locally although he would prefer not to see the local body actually doing the work. Mr. Toynbee also raised the question whether this suggested fund should not be subsidised by the Earthquake and War Damage Commission. If this were the case it would ensure a responsible local attitude as well as give some financial assistance for major problems. The local contribution would have to come, not only from subdividers, but from others also. There should not be too many tags to the subsidy and it should be controlled locally by as few people as possible. The short answer was that he was sure there was a stability problem and it was not only here and he would like to see Mr. Gill's suggestion taken further.

The Chairman then asked the next question:-

(b) Were we considering the possibility of landsliding at the right time in relation to the commencement of planning and construction?
(Nobody thought we were considering it soon enough).

<u>Mr. Riddolls</u> put the question another way by asking if everybody was satisfied with the present situation. There was no doubt that today there was a much greater awareness of the problems that could arise through instability. Under the Town and Country Planning Act as it now stood, district scheme reviews required that unstable areas be notified but this did not extend to the cutting of natural slopes where no apparent instability existed. If people were not satisfied with this, what should be done? The general view was that in many instances, but especially in urban development, the problem of landsliding <u>was</u> tackled too late - usually at the stage of picking up the debris. It therefore seemed desirable that planning controls should be modified so that appropriate investigations were commenced before any hillside development, perhaps starting with a landslide susceptibility assessment as already described in the symposium, followed by surveillance throughout development so that modifications could be made where unfavourable conditions arose. If it was good enough to pay large sums to build a house - often with a large fee for an architect to design it - surely there was some justification for also paying for attempting to ensure its permanence for as long as possible.

<u>Mr. Taylor</u> said the third qestion was rather more a[illegible]se:

(c) Was there a balanced view of the importance of landslip in relation to the consequences in different circumstances? Was too much effort and attention spent in those circumstances where one could afford to clean up the consequences and perhaps too little attention to those cases where the results would be disastrous?
(Only a few agreed there was a balanced view).

<u>Mr. Russell</u> said that if this question was taken in its full context the answer obviously must be 'no'. When it came to the national scene, for investigational work on large power stations, rail-

ways, major roading works - construction works that involved considerable public funds, corporation funds or company funds and/or involved the safety of a large number of people, obviously very good work was being done, and soil conservation engineers and geologists were on top of the problem and should be given every encouragement to carry on in that manner. To the average citizen, however, Mr. Russell said that these areas were very remote and generally speaking, very little was heard of them. Most of the problems that arose in the city and in the urban areas, generally arose through ignorance where sections were hacked into by bulldozers and it was likely that at this time in Wellington there was some bulldozer driver cutting a vertical bank in clay and rock up to 10' deep within one foot of the neighbour's property. The only person who would win, was the lawyer. It was quite impossible to legislate for or police all works that were done over the years on suburban sites, but at least the element of risk could be substantially reduced by ensuring that the lands and excavations were reasonably safe and stable at the time a house was built. Mr. Russell said that before anything constructive could be done, the will to do it had to be there and in many cases this was not evident. The suggestion that legislative powers were required in itself seemed to suggest lack of interest and passing of the buck. The human angle, which was of more importance to the citizen, suggested the issuing of a code of practice in very simple terms, to cover the control of excavations within urban limits. Authority to cover the procedures required could be obtained, if it was not already there, by an adjustment in the building by-laws and/or an adjustment in the Town Planning Codes of Ordinances. There was a need to protect the citizen from the folly or greed of others. Mr. Russell felt it was up to Engineers to lead in this matter and it should be shown that they had the interest of the community in the forefront.

The Chairman then presented the next question:-

(d) Were we using the best available techniques which we could afford to forecast the likelihood of landslip?
(Over half felt 'no').

Professor P.W. Taylor said that when this was considered on the small scale, he would agree that the best available techniques were not being used. The builder, the bulldozer driver or the house-proud owner were doing things which even common sense should tell them they should not be doing. This arose from ignorance and possibly the Geomechanics Society had some duty to educate the public to some extent on those matters. The Society in Auckland produced a booklet some years ago on house foundations which covered this topic and perhaps this approach should be extended. On the large scale, Professor Taylor felt they were doing the best they could afford. Particularly for the intermediate sized slopes associated with subdivisions, the analytical techniques used were seldom the most sophisticated available, but engineers just could not afford to use them and such techniques must be left to the large non-profit organisations such as the Ministry of Works and possibly the universities in their research projects - institutions which could freely spend the taxpayers' money. For any problem of medium size, it was the expense which must control the situation and whereas the standards of practice varied throughout the country, the best standards of practice were pretty good. Most engineers tried to do a conscientious job within the limitations of the money they were allowed to spend and the knowledge they had at their fingertips. It must be admitted that soil mechanics was a very complex subject - the more one learned about it, the more complex it appeared to be. It was perhaps misleading to say that a certain slope had a factor of safety of 1.1 under the worst possible conditions and therefore it would never slip. All one could really say was that there was a fair probability that it would remain stable. This was not generally realised by non-engineers, who believed that the engineer could calculate anything and everything in a precise manner. Generally, the standards of engineering practices were reasonably high in New Zealand. There was the matter of lag in information dissemination. To a large extent - with some exceptions - the best available techniques were being used, that could be afforded.

Mr. Taylor

(e) Were new or modified administrative or legislative processes necessary to reduce the incidence of damage to property?
(About 80% of participants considered they were necessary).

Mr. Gillespie said that in his position it was very easy to become emotional over the recent large number of landslips in Wellington. He believed, however, it was necessary to try to maintain a rational view of this type of experience because Wellington had received its worst winter on record. Civil engineers did not design their drains, culverts etc. for an infinite flood condition, they designed on a rational return period of 50 or 100 years and probably in the broadest sense landslips might have to be considered in the same way. The main factors which he saw as necessary, either by better policing of present local authority laws, or by the introduction of new legislation, included better control of storm water from areas on or above any areas of development, particularly simple problems - discharge from yards, roofs, driveways etc. This was the basic triggering mechanism of the majority of the landslips in Wellington. He understood that this was already covered by Common Law which decreed "Thou shalt not discharge concentrated flows of water on your downhill neighbour". It was also covered by the Soil Conservation and River Control Council legislation but these things were very poorly policed.

Mr. Gillespie said there was also a need for the better control of excavation both in mass earthworks for residential development in the broad scale and in the small scale where house platforms were being cut by the builder. Town and Country Planning Regulations required Local Authorities to have regard to landslip, amongst other things, in their deliberations. Thus in their normal review of scheme plans submitted prior to any development, the Local Authorities could and should use any local knowledge they have to determine the feasibility of cut slopes etc. in a proposed subdivision. Alternatively, consideration could be given by the Local Authority to demanding that a proposed development be supported by the production of sound geotechnical evidence. Considerable assistance on the small scale problem could be given by local authorities demanding that building permit applications show what types of cutting and filling were required on a building site when application was made for a permit. He referred to two recent Provisional Codes of Practice now out by the Standards Association. Both of these Codes of Practice specified that cut batter was being formed for building platforms and were not to exceed 8' high and they gave quite conservative slopes for batters that were formed, in sound unfractured rock not steeper than 1/2 to 1 and in fractured rock, not steeper than 1 to 1. The use of that type of criteria could be of great assistance, he believed. Current legislation to some extent already provided for minimising some problems. Mr. Gillespie, said a gread deal could be achieved by more energetic policing of current legislation combined with a programme of public education showing how dangerous steep unretained cuts were, how bad their poorly built cribs commonly were and just what the effect of lack of control of storm-water was. Nevertheless, the one factor he did see as dominant, was the need for some Code of Practice in respect of the development of cut ground.

The Chairman said the questions had turned out to be rather more provocative than he had hoped they could be. He asked the audience to show their appreciation to the five people who had spoken.

This concluded the Symposium.

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APPENDIX

GEOLOGY AND ENGINEERING INFORMATION NELSON CITY AREA

The following notes were prepared for the information of symposium participants on a bus tour of the Nelson City Area:-

(1) GEOLOGY OF THE NELSON CITY AREA - *M.R. Johnston, N.Z. Geological Survey, Nelson.*

The geology of the Nelson City area is varied with rocks, ranging in age from Permian to Recent, divided into four major units:

Unit 1 - basement rocks,
Unit 2 - Lower Tertiary rocks,
Unit 3 - Upper Tertiary and Lower Pleistocene terrestrial gravels, and
Unit 4 - Upper Pleistocene and Recent marine and terrestrial deposits.

UNIT 1

The basement rocks are indurated and well jointed and consist of basic volcanics (Brook Street Volcanics) of Lower Permian age, marine sediments (Maitai Group) of Upper Permian age, and acidic volcanogenic sandstone and siltstone (Richmond Group) of Triassic age. They are steeply dipping and are commonly overturned. Folding, about north-east to ENE axes, is widespread in the Maitai and Richmond Groups but no folds have been recognised in the Brook Street Volcanics. They underlie steep sided hills, with slopes of up to 40° in the east of the city.

UNIT 2

The Lower Tertiary rocks are consolidated and comprise basal coalmeasures overlain by a transgressive marine sequence consisting of shallow water siltstone and conglomerate, with local barnacle plate limestone, in the east and deepwater graded beds in the west. The coalmeasures are only exposed in the east as a faulted strip on the west side of the Waimea Fault. The marine sediments crop out in Nelson South and on the west side of the Port Hills. The Lower Tertiary rocks are steeply dipping and in Nelson South and adjacent to the Waimea Fault they are commonly overturned. Except locally, they are poorly exposed, commonly deeply weathered, and underlie gentle to moderately steep topography.

UNIT 3

The terrestrial gravels are widespread and consist of well rounded pebbles, cobbles and boulders in a silty clay matrix. They are moderately to deeply weathered and two distinct units, Port Hills Gravel and Moutere Gravel, are recognised. The older, (Upper Tertiary) Port Hills Gravel contains material largely derived from the area adjacent to Nelson city. It outcrops extensively in the Port Hills, Atawhai, and south-east of Stoke and is moderately to steeply folded about north trending axes. The younger (Lower Pleistocene) Moutere Gravel crops out extensively to the south-west of the city and contains greywacke debris derived from the Spenser Mountains at the northern end of the Southern Alps. Because of the high clay content in the matrix, the gravels can form very steep faces with little risk of initial collapse. However, as weathering proceeds slips become increasingly more numerous on such slopes.

UNIT 4

The Upper Pleistocene and recent deposits include marine storm benches at Tahunanui, the Boulder Bank forming Nelson Haven, and alluvium deposited by rivers draining the area. Extensive areas of reclamation exist in the southern end of the haven.

Instability in the Nelson area is widespread and includes superficial slipping on moderate to steep slopes, large scale mass movements, such as earthflows and rotational slumps, and collapse of oversteepened faces, such as the seacliffs surrounding Tasman Bay.

LEGEND

AGE ($\times 10^6$ years)

UNIT 1 - RECENT AND UPPER PLEISTOCENE

		AGE ($\times 10^6$ years)
	Reclaimed land, boulder banks, river alluvium, marine sand and gravels, scree and fan deposits	< 0.5

REGIONAL UNCONFORMITY

UNIT 2 - LOWER PLEISTOCENE

		AGE ($\times 10^6$ years)
	Clay bound gravel consisting of well rounded cobbles and scattered boulders of greywacke, deeply weathered (Moutere Gravel).	1

REGIONAL UNCONFORMITY

UPPER TERTIARY

		AGE ($\times 10^6$ years)
M	Clay bound gravel consisting of well rounded boulders and cobbles of locally derived rocks. Granite boulders common at base in west (Port Hills Gravel).	8

REGIONAL UNCONFORMITY

UNIT 3 - LOWER TERTIARY

		AGE ($\times 10^6$ years)
L	Graded sandstone and siltstone beds in the west and more massive siltstone in the east. Impure, locally conglomeratic, barnacle limestone at base in east (Oligocene). Dark grey to brown sandstone and siltstone; conglomeratic near top (Eocene).	25

FAULT CONTACT

		AGE ($\times 10^6$ years)
L	Bedded carbonaceous sandstone, siltstone, mudstone with scattered conglomerate lenses and minor coal seams (Eocene)	45

FAULT CONTACT

UNIT 4 - BASEMENT ROCKS

RICHMOND GROUP (TRIASSIC) — 220

T	Grey, bedded volcanogenic sandstone, siltstone and mudstone Coarse granite derived conglomerate lenses.

FAULT CONTACT

BROOK STREET VOLCANICS (LOWER? PERMIAN) — 270

V4	Fine grained green tuff (Groom Creek Formation).
V3	Coarse, green augite tuff (Kaka Formation)
V2	Find grained, grey well bedded clacareous tuff (Grampian Formation)
V1	Grey, green, purple tuff and breccias, grey tuffaceous siltstone (Botanical Hill Formation)

FAULT CONTACT

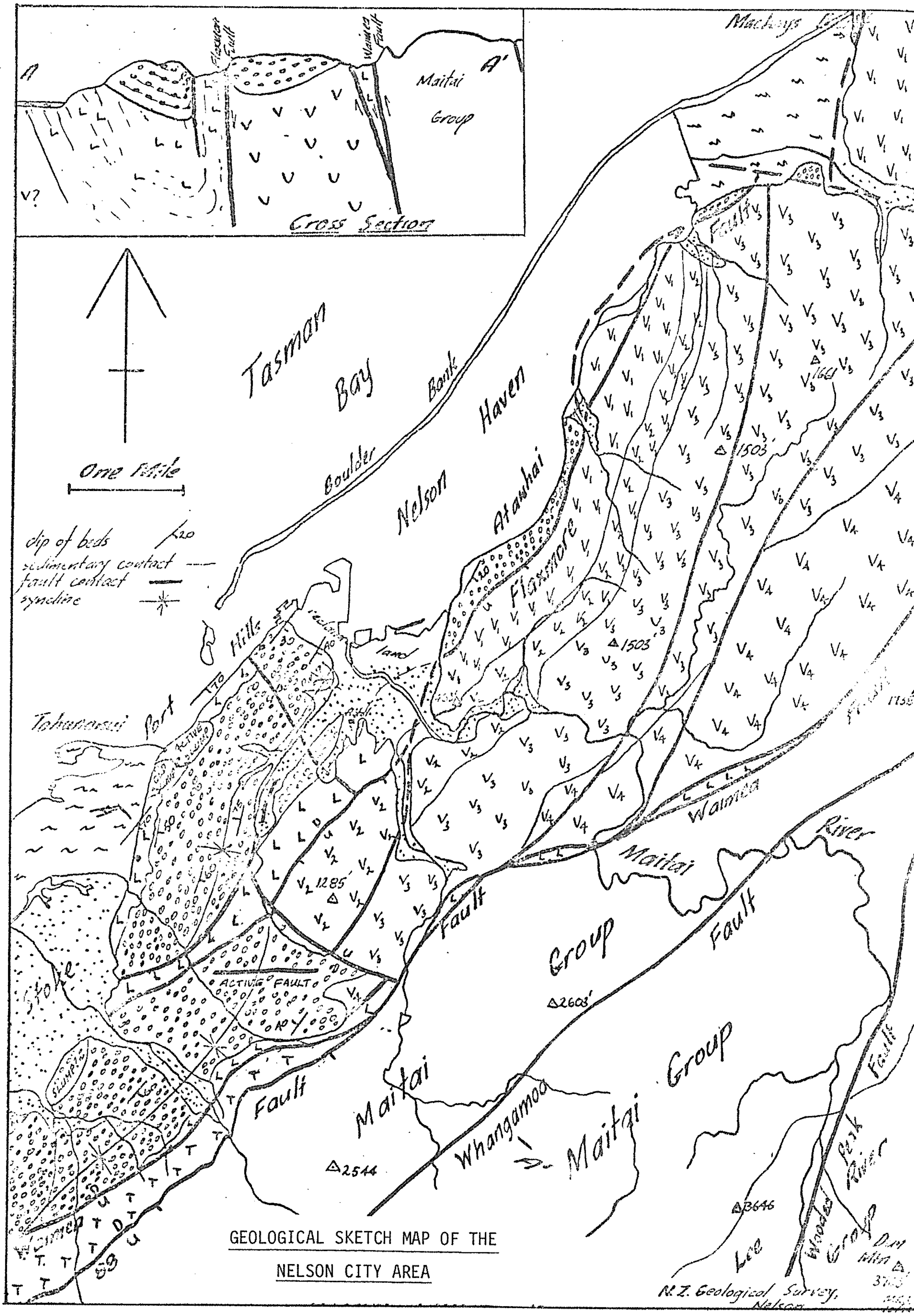
MAITAI GROUP (UPPER PERMIAN)

	Green, grey sandstone, conglomerate, thin limestone lenses (Stephens Formation). Well bedded and laminated green sandstone and grey and red siltstone-mudstone (Waiua Formation). Well bedded and laminated sandstone-siltstone (Greville Formation) Grey and green well bedded sandstone (Tramway Sandstone) Well bedded grey limestone with minor green, grey and pink calcareous sandstone beds (Wooded Peak Limestone).

UNCONFORMITY

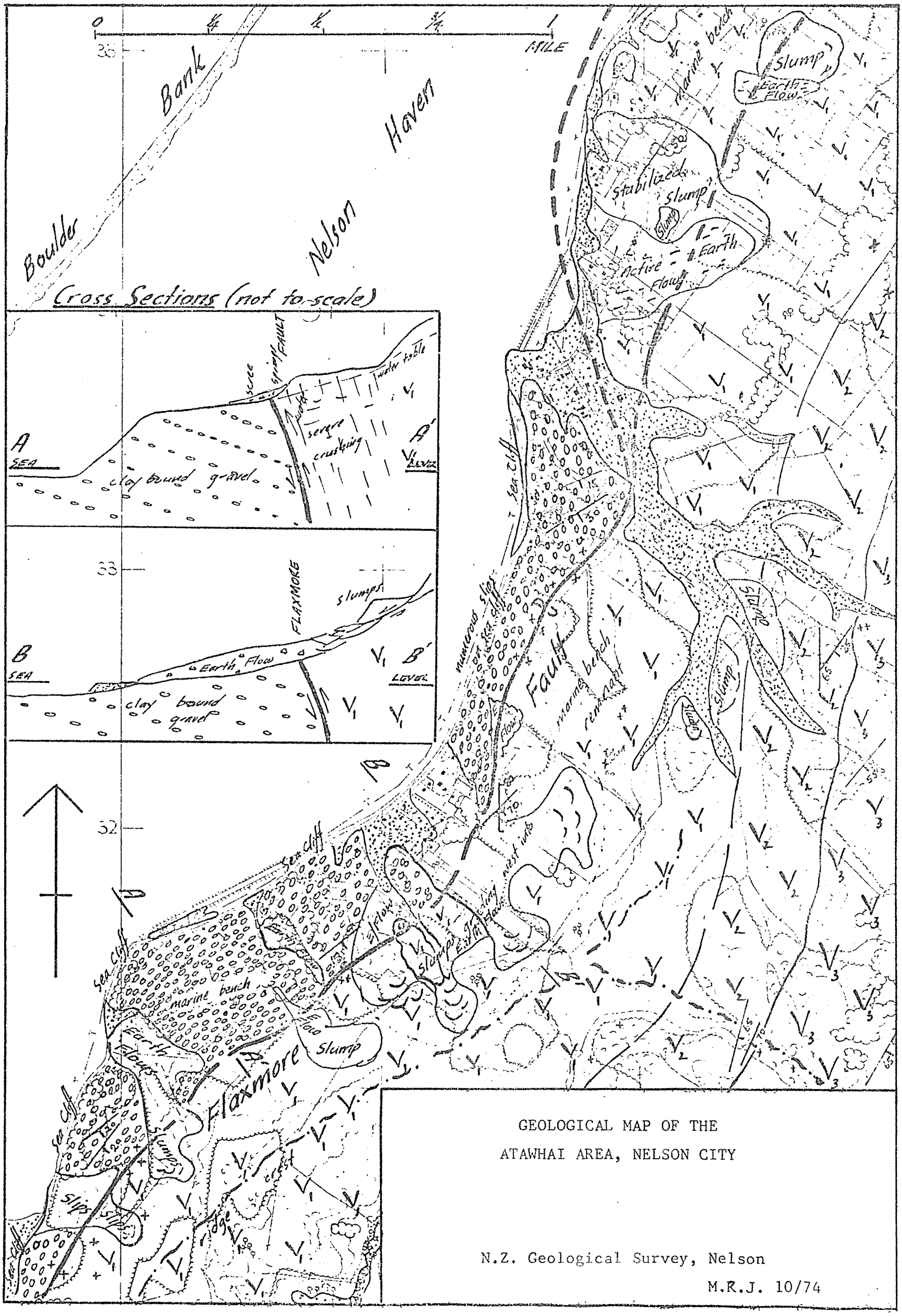
LEE RIVER GROUP (LOWER PERMIAN)

	Green and grey volcanics with volcanic breccia at top and gabbro and dolerite at base. Dunite, pyroxenite and serphentinite



GEOLOGICAL SKETCH MAP OF THE

NELSON CITY AREA



GEOLOGICAL MAP OF THE
ATAWHAI AREA, NELSON CITY

N.Z. Geological Survey, Nelson
M.R.J. 10/74

<u>(2) ENGINEERING INFORMATION</u> - *G.A. Toynbee, N.M. Crampton, City Engineer's Department, Nelson City Council*

Nelson is an attractive area in which to live, mainly because of its benign climate, its central location, its range of scenery, and the fact that it faces north and the sea. The hills behind the city contribute to the scenery, but make Nelson an elongated city with limited flat and gently sloping land. Thus, development of the hills is a logical corollary, and this is made more necessary because of the serious shortage of good farming land in the province. A study of the geologists' paper and maps reveals that the geology of Nelson is complicated and problems of stability abound.
The main areas where there are development and instability are:-

(1) Tahunanui Hill
(2) Waimea Road/Grampians
(3) Atawhai.

(1) <u>The Tahunanui Hill slump</u> has been recognised as such since the European came to Nelson, but it was not until 1962, during the wettest year on record, (61" rainfall compared with a 39" average), together with an earthquake, that its significance was really felt. The scarp, the hummocky nature of the ground, and local ponds, were obvious signs, and these were confirmed by survey discrepancies over the years, but it was a pleasant place to reside, overlooking Tasman Bay and basking in the sun. Houses were built, and little attention was paid to drainage. In the 1929 earthquake, some slipping occurred, and one house was demolished, but problems were soon forgotten and development continued.
In 1962, two houses were demolished, many others suffered serious damage, minor slips occurred, and there was further movement on the major slump, as evidenced by a crack along some of the base of the scarp. In addition, many residents had local movement on their properties, but preferred to ride them out in silence to maintain property values. The Council has subsequently installed a stormwater system and rebuilt roads in much of the area, as well as insisting on all roofwater being piped into the system. Property values have been re-established.
Again in 1970, after some days of intense rain and serious flooding, some local slips (involving the original seacliffs) occurred further south on Tahuna Hill and beyond the main slump. One house was destroyed and one life lost as a result of the partial crushing of a second dwelling.

(2) <u>Waimea Road/Grampians Area</u>. Local problems of stability have occurred along the base of the Grampians from Bishopdale Hill to Brougham Street. Recent slips are evident on Bishopdale Hill, and a close study of much of the area reveals movement in the last century. During heavy rain in October 1974, a small slump, involving at least one house, occurred. This area is particularly susceptible because of the association of a fault line and very sensitive clays.

(3) <u>Atawhai</u>. Atawhai has recently come into the city, and with it more stability problems. The area is traversed by a fault line which has been a contributor to the slumps on the hillsides. Some of these are known to have moved recently, and in one area, remedial work was carried out by the Council on some sections to ensure that the carriageway could remain operational.

Because of the shortage of sections, we are having to consider using land which is suspect, but we must make a study and reasonable assessment of the position. The fact that the Earthquake & War Damage Commission now insures against landslip, by no means implies that we should be less thorough. Finally, after a lot of discussion involving Engineers and Solicitors, it was decided that the developer should pay for the investigation, and the City Council has made it a condition of subdivision that a Registered Engineer's Certificate be obtained. This report is not made public, but the Council's requirements of the subdivider (assuming the subdivision proceeds) are based on it. This system is now working satisfactorily. Certificates must be in one of the two forms, depending on at what stage they are given. The form of these certificates are given on the next page.

ENGINEERS CERTIFICATES FOR SUBDIVISIONS

The following is the layout for a Certificate to be used at the scheme plan stage or at the final plan stage when no engineering works are involved.

"The Town Clerk
NELSON

LAND DESCRIPTION

We have carried out an investigation in accordance with sound engineering principles and practice of the above property, and submit this report in conjunction with ...(Surveyor)....'s Scheme Plan No. .... of Subdivision.

It is of professional opinion, not to be construed as a guarantee, that there (is/will be) a site on each section suitable for the erection of a residential building, providing that:-

(a) ....

(b) ....

(c) ....

etc. as required.

(If there are no suggested conditions, Paragraph 2 would end after the word 'building').

This Certificate is furnished to the Nelson City Council alone, on the express condition that it will not be communicated to or relied upon by any other person.

Yours faithfully,

REGISTERED ENGINEER"

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ENGINEERS CERTIFICATES FOR SUBDIVISIONS AT FINAL PLAN STAGE

APPROVED LAYOUT

Approved by the Nelson City Council on 13th September 1973.

"The Town Clerk,
NELSON

Dear Sir,

We forward herewith the "As Built" drawings and the specifications relating to the above subdivision and certify as follows:-

(1) That the drawings and specifications have been prepared in accordance with sound engineering principles and practice and have been approved by the Nelson City Council. The work has been carried out by XYZ Contractors and we enclose their Certificate that they have carried out the work in full accordance with the Contract requirements. We have observed the work, in terms of our engagement to the extent necessary to ascertain whether the design was being interpreted correctly, and whether the works were being carried out generally in accordance with the contract documents, and one of our staff was engaged as Clerk of Works during the stages of the Work. We enclose our report on Earthfills in terms of the N.Z. Standard 'Code of Practice for Earthfills for Residential Development No. 4431P'. On the basis of these certificates, observations and reports, we are of the opinion that the works have been carried out generally in accordance with the approved drawings and specifications.

(2) As a result of this work, it is our professional opinion, not to be construed as a guarantee, that there is a site on each section suitable for erecting a residential building, providing that:-

(a) ....

(b) ....

(c) ....

etc. as required.

(If there are no suggested conditions, Paragraph 2 would end after the word 'building'.)

(3) This certificate is furnished to the Nelson City Council alone and on the express condition that it will not be communicated to or relied upon by any other person.

Yours faithfully,

REGISTERED ENGINEER"