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ASSESSMENT OF SLOPE STABILITY AT BUILDING SITES

Worley Consultants LTD

BUILDING RESEARCH ASSOCIATION OF NEW ZEALAND

PREFACE

The Building Ressearch Association of New Zealand commissioned this report to assist in the development of uniform procedures for the assessment of slope stability at building sites. The views represented are not necessarily those of the Association.

ACKNOWLEDGEMENTS

Worley Consultants Ltd wishes to acknowledge the help and support of BRANZ, Mr D. K. Taylor, who was responsible for technical liaison with BRANZ, and Professor M. J. Selby, who carried out a technical audit of the report. Mr Norman Woods, who was responsible for the preparation of this report, is grateful for the assistance of several of his colleagues, particularly Mr David Convery, Dr Bruce Riddolls, and Mr Brian Shakes, who provided valuable advice.

This report is intended mainly to provide local authorities with guidelines by which to judge the adequacy of slope stability assessments submitted with building permit applications. The guidelines are also intended to be of use to those undertaking the technical assessment work.

ASSESSMENT OF SLOPE STABILITY AT BUILDING SITES

BRANZ Study Report SR4 **Worley Consultants Ltd**

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ABSTRACT

Most local authorities require a favourable report on the stability of sloping land before issuing a building permit. A variety of different approaches is used in making slope stability assessments ranging from superficial inspections to detailed investigations involving subsurface investigation, laboratory testing and stability analysis. This report reviews the procedures commonly employed in slope stability assessments in New Zealand at present. The advantages and disadvantages of these procedures are discussed and, where appropriate, recommendations made as to their most suitable application.

Costs for slope stability assessments can be very high and the level of investigation carried out should be appropriate to the nature of the proposed building development and the consequence of any potential slope failure. The avoidance of a potential failure by, for example,

modifying the building layout may preclude the need for costly laboratory testing and stability analyses.

The validity of the results of a stability analysis is governed chiefly by the reliability of the input data. A thorough understanding of the geology of the slope and of the groundwater conditions is particularly important. The factor of safety derived from an analysis should therefore be judged in terms of the equivalence to reality of the assumed conditions used in the analytical model.

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1.1 General

Most local authorities require a favourable report on the stability of sloping land prior to issuing a permit for building. A variety of approaches is used in these stability assessments, from a superficial site inspection to a detailed investigation involving subsurface investigation, laboratory testing and stability analysis. Not only can the approach adopted vary, depending on the person or organisation undertaking the assessment, but also the scope of the work may vary according to the ability or willingness of the client to pay for the level of assessment actually required. In addition, different local authorities may have different criteria for judging the acceptability of a slope stability assessment. Clearly, there is a need for more uniformity than exists as present.

1.2 Objectives

The primary objective of this report is to help establish a uniform approach to the assessment of slope stability. The investigatory and assessment procedures normally employed are discussed, their advantages and limitations examined and recommendations made as to their most appropriate application. It is intended that coverage of the subject in this manner will provide the person scrutinising a slope stability assessment report, e.g. the local authority engineer, with guidelines by which to judge its adequacy and validity.

Throughout the report reference is made to the relative cost of different assessment procedures. Also, in Appendix A, typical costs for various levels of assessment are given, based on rates charged by consultants in 1986.

The inclusion of this information is to highlight the considerable expense that the developer or property owner may be faced with. The public should be aware of these costs, and all parties should recognise that any attempt to minimise costs by reducing the scope of investigations may significantly impair the validity of the assessment. In this respect, it is essential that the investigator clearly states in his report the limitations of his investigations, and suitably qualifies the conclusions reached.

1.3 Scope

The overall approach and individual procedures discussed in this document are relevant to the stability assessment of fill slopes, cut slopes and natural slopes. Greater coverage is given to 'soil' slopes, in view of their predominance in connection with most building works and recognition of the higher incidence of slope failures in these materials. However, procedures that are primarily intended for 'rock' slopes are also discussed in view of the significant implications that rock slope failures may have locally.

Only the technical aspects of a slope stability assessment are dealt with; such matters as liability or insurance are not covered. This document is not intended to fulfil the role of a text book; each topic or procedure is only covered in broad terms except where a more detailed discussion is considered desirable in order to highlight the importance of a particular topic. For the reader who requires elaboration on a particular subject, a list of references is provided.

The appendices provide additional information to expand on certain topics covered in the main text. Appendix A provides typical costs for various 'levels' of assessment, based on rates charged by consultants in 1986. Appendix B provides background information regarding the types and causes of instability, and Appendix C briefly outlines common remedial and preventive measures.

2. QUALIFICATIONS OF ASSESSMENT PERSONNEL

2.1 The Geologist and Civil Engineer

The subject of slope stability falls within the fields of both geology and civil engineering. However, that is not to say that all geologists and civil engineers will necessarily have all the required skills and experience for carrying out a slope stability assessment. A geologist who specialises in mineral resource assessment may be no more appropriate to the task than the civil engineer who has spent most of his working life designing roads.

The following extract from a keynote address by Professor R B Peck (1977) highlights the limitations of the 'pure' geologist and civil engineer with respect to slope stability assessment :

> "Geologists are good at recognition of landslides and landslide topography and at understanding geological structure and stratigraphy, slide processes and slide features, groundwater and hydrogeology. Geologists are poor at quantifying properties of earth materials, seepage pressures and pore pressures and in performing equilibrium calculations. They are overzealous in classifying slides with little regard to fundamental causes. On the other hand engineers tend to look at a landslide as just another structure but are good at equilibrium calculations and at estimating seepage pressures and pore pressures and at carrying out quantitative studies of remedial measures. Engineers are, however, poor at visualising the anatomy of slides and tend to over-idealise slide masses. They are also poor at picking most probable slide surfaces which are often governed by geological details. They are usually poor at picking out differences from one site to another and at interpretations of subsurface conditions. They tend to interpret slides on a mechanistic rather than geological basis".

2.2 The Engineering Geologist and Geotechnical Engineer

Specialisation in both disciplines in recent years has led to the development of engineering geology and geotechnical engineering and it is the practitioners in these fields who are most suited to carrying out a slope stability assessment. The designations 'Engineering Geologist' and 'Geotechnical Engineer' were defined by Professor P.W. Taylor in a submission to the Commission of Inquiry into the Abbotsford Landslip Disaster, as follows:

> "The engineering geologist has a thorough knowledge of geology, and also some knowledge, acquired by academic training or through experience or both, of the methods of engineering analysis as applied to geotechnical problems. Instead of the 'purely scientific' approach of the traditional geologist, he is trained to apply his knowledge in assisting in the design and construction of civil engineering works. He is capable of understanding the problems faced by engineers and of communicating with them in a way which is of value in making engineering decisions".

> "Amongst civil engineers, some specialise in geotechnical engineering. Either by post-graduate university studies, or by practical experience and private study, such engineers have specialist knowledge of soil mechanics, foundation engineering and possibly rock mechanics".

Notwithstanding the general suitability of the engineering geologist and geotechnical engineer for carrying out slope stability assessments, the limitations of both should be recognised; there are few who have a thorough understanding of both geology and engineering. Consequently, particularly for sites with complex geology or those involving less conventional building structures, interaction between the engineering geologist and geotechnical engineer is important.

In the following text, the person undertaking the assessment is referred to as the 'investigator'. However, where there is a clear preference for either a geotechnical engineer or an engineering geologist to undertake or supervise any of the tasks discussed in this document, such preference has been stated in the relevant section.

2.3 Local Authority Requirements

The Code of Practice for Urban Land Subdivision (NZS 4404:1981) requires that slope stability assessments carried out in connection with subdivision developments should be undertaken by a 'soils engineer'. A soils engineer is defined in the standard as ...." a person who is currently entitled to practice as a registered engineer and has experience in soils engineering acceptable to the Council; or such other person as the Council may specifically approve as being competent." In the case of building permit applications, there is no general requirement with respect to the credentials of the person carrying out the stability assessment. However, many local authorities use a standard form entitled "Statement of Professional Opinion as to suitability of land for Building Construction". This form must be signed by a person defined as "....a registered engineer experienced in the field of soils engineering and more particularly land slope and foundation stability (as applicable) ...." Unfortunately, this requirement does not recognise the fundamental importance of geological expertise in slope stability assessment. Even in the case of a slope stability analysis, normally performed by the geotechnical engineer, an engineering geologist should be involved to assess the validity of the assumptions made, particularly with respect to the proposed failure surfaces, and to assess the rationality of the results.

3. SITE INVESTIGATION

3.1 General

The aims of the site investigation for a slope stability assessment are to determine the nature and distribution of the slope forming materials, to determine the groundwater conditions within the slope and to examine any existing or possible future external influences on the stability of the slope.

In this section the procedures normally employed for collecting this information are outlined. They are grouped according to the type of investigatory activity and presented in the order that they should preferably be carried out, i.e. desk study first, followed by a field inspection and finally, a subsurface investigation.

3.2 Desk Study

3.2.1 Existing Documented Information

Prior to visiting the site, and certainly before carrying out any detailed and costly subsurface investigation work, it is recommended that the investigator spend time examining what background information is available for the site and the surrounding area. This information may not only prove useful in ascertaining the relative stability of the subject slope but may also provide an indication of any specific stability problem which can then be addressed at an early stage in the investigation. This may result in a considerable saving in time and effort.

Relevant sources of background information may include :

(a) Prevous site investigation reports covering the subject site, prepared at Scheme Plan Stage,

(b) Previous site investigation reports for the subject site if, for example, the proposed building development involves redevelopment of a site that has already been built on,

(c) Previous site investigation reports for neighbouring sites if already developed or intended for development,

(d) Regional stability or land use studies carried out for a local authority, catchment board, etc.,

(e) Geological maps; the 1:25000 Industrial Series geological maps produced by the New Zealand Geological Survey, currently available for the majority of the Auckland Metropolitan Area, as well as for Hamilton, are particularly useful as a source of engineering geological data,

(f) New Zealand Land Inventory Maps (NZMS 290)

(g) Technical papers in journals, such as the New Zealand Journal of Geology and Geophysics,

(h) N.Z. Soil Bureau maps, and

(i) University theses.

3.2.2 Aerial Photograph Interpretation

In addition to an examination of existing documented information, it is recommended that aerial photograph interpretation be carried out for every site studied. This technique, and its value for slope stability assessments, is briefly outlined below.

Aerial photograph interpretation involves the examination and interpretation of the three dimensional image perceived when a pair of aerial photographs is viewed through a stereoscope. In order for a stereoscopic image to be seen, the two photographs must have a degree of overlap (normally 60%) in terms of the area covered by each photograph. Aerial photographs are classed as either oblique or vertical depending on the angle of the optical axis of the camera at the time of exposure. Both oblique and vertical aerial photographs have their uses although the latter are more widely available as they form the basis for modern topographic map production.

When viewed stereoscopically, aerial photographs provide the investigator with a three dimensional overview of the subject site and its surroundings. The value of putting the site into its wider topographical setting using this technique cannot be overemphasised. It is quite possible that, within the limits of the site itself, there may be no evidence of instability; however, the site cannot be assessed in isolation - it may be, for example, that the site is part of a larger landslide mass or that the area upslope may be potentially unstable and could affect the subject site. Such situations should be relatively easy to identify from aerial photographs whereas, on the ground, the investigator is faced with many distractions and possible difficulties in reaching a vantage point from which to gain an overview of the site.

The aerial photograph examination should, where appropriate, include a comparison of the subject slope with others in the area. The incidence of instability on slopes which are similar to the subject slope in terms of, for example, slope angle, aspect, general morphology, etc., may forewarn of possible stability problems on the subject slope. This assumes however that ground conditions in the slopes being compared are essentially the same - this may not be the case. Consequently, the results of comparative studies, based on aerial photograph interpretation, cannot be considered conclusive unless supported by thorough field checking (possibly including subsurface investigation).

Aerial photographs are also useful in identifying, either directly or indirectly, many other geological and geomorphological features besides instability, e.g. rock type, structural discontinuities, superficial deposits (colluvium and fill), etc. Also, for developed areas which are to be redeveloped, old aerial photographs taken prior to initial development may reveal the presence of former topographic features, e.g. pre-existing stream courses and areas of instability, that may have been obscured or eradicated during the course of development.

Complete black and white aerial photographic coverage of New Zealand is available and this is periodically updated (at least every ten years for any given area) in terms of the rate of development and change taking place in various parts of the country. There is an ongoing programme of coverage at 1:25000 and 1:50000 and there are numerous special surveys at a variety of larger scales. The latter include surveys for state highways, rail routes and development projects such as irrigation and power schemes.

For most parts of New Zealand, photography is available dating back to the 1940's. The Photo Library of the Lands and Survey Head Office in Wellington holds a comprehensive collection (approximately 1/2 million photos) of all Crown copyright and some private copyright aerial photographs covering the whole of New Zealand. In addition, the twelve district Lands and Survey offices also maintain their own collections of photographs for their local districts.

Aerial photographs may be examined by the public at these offices and, if required, copies can be purchased either through the Lands and Survey Offices or directly from New Zealand Aerial Mapping Limited, Hastings.

3.3 Field Inspection

3.3.1 General

The object of the field inspection is to acquaint the investigator with actual site conditions, to put the building development proposals into perspective and to assess the likely existence and magnitude of any stability problem. On the basis of the field inspection, the investigator must be able to decide the need for and most appropriate form of subsurface investigation required to establish ground conditions beneath the site.

The field inspection requires, above all, geological judgement in interpreting landforms and making predictions regarding geological conditions on the site. Accordingly, the inspection should be carried out by an engineering geologist and, preferably, one acquainted with local conditions and stability problems. The engineering geologist may however need to seek advice from an engineer with respect to the engineering aspects of the proposed building development, particularly where the building layout and structure is complex.

Providing that a desk study has already been undertaken, as is recommended, the investigator should already be aware, in general terms at least, of the topographical and geological conditions of the site.

Also, he should be in possession of a site plan and have been briefed such that the proposed building layout can be clearly defined on site. The plan should show not only the position and size of buildings but also any proposed earthworks and proposed locations for services, and effluent disposal systems if required.

It may be useful, particularly for sites where the proposed building layout is complex, for the client to meet the investigator on site at the time of the field inspection. This allows any uncertainties regarding the building proposals to be resolved before large sums of money are committed for detailed investigations. Any erroneous assumptions made at the outset may lead to embarrassment and misunderstanding later. There have been cases for example where a subsurface investigation was carried out on the wrong section because the wrong site plan was sent to the investigator.

An additional advantage of client and investigator meeting on site at the outset of the investigation is that any obvious problems can be brought to the attention of the client and, where possible, can be avoided by modifying the building layout. Also, any possible disagreements with regard to the final cost of the assessment can be avoided if the client is briefed on site regarding the type of investigation required and its likely cost. Should the client feel that the level of investigation proposed is excessive or unwarranted he could then choose to seek advice elsewhere. The only costs that would have been incurred at this stage would have been a few hours of the engineering geologist's time in carrying out a desk study and making the field inspection.

3.3.2 Observations

The time spent inspecting a site will depend on its size and complexity. The site inspection must of course be thorough; evidence missed at this stage may result in a misdirected subsurface investigation or a misleading assessment.

Table 1, page 49, presents a check list of observations that should be made. One or more of the items listed may not be relevant to each site inspected. However, the use of a check-list is a methodical approach that ensures that observations are not overlooked. The value of the individual observations listed is briefly explained below.

3.3.2.1 Ground profile

Observations should be made of the landforms both within and surrounding the site. Certain landforms are more commonly associated with instability than others; for example, the heads of gullies and other depressions in slopes are particularly susceptible to instability due to the ingress of groundwater seepage and/or surface water.

Irregularities in the ground surface may be evidence of past or continuing instability, whether in the form of a landslide or progressive soil creep. Slope angles should be measured rather than estimated, particularly at critical locations, e.g. on steep ground adjacent to the site of a proposed building. A visual estimation of slope gradient can be very misleading particularly when the slope is viewed from above (a high slope of 35° looks much steeper than it actually is) and, if a stability analysis is carried out, an error of say 5° in slope angle can make a significant difference to the calculated factor of safety. In most cases, the use of a tape and Abney level (or clinometer) should give acceptable accuracy although, for particularly difficult sites, where more precise data is required for a stability analysis, a detailed topographic survey may be warranted.

3.3.2.2 Evidence of instability

In addition to irregularities in the ground profile, there are other signs that may indicate past or continuing instability. Some of the more easily recognisable signs of instability are illustrated in Figure 1, page 50 and are briefly discussed below. "Field Assessment of Slope Instability" (Crozier, [1984]) is also a good guide and has many references to New Zealand examples.

A careful inspection should be made for any cracks in the ground (they may be obscured by vegetation). The position and extent of any such cracks should be recorded on a site plan; they may be related to a nearby topographical feature, e.g. a nearby break in slope. Observations should also be made regarding crack aperture and any vertical displacement of the ground surface across the crack. The existence of ground cracks does not necessarily indicate slope movement however; they may, for example, develop due to shrinkage of certain clayey soils during prolonged periods of hot dry weather. The existence of shrinkage cracks should nevertheless be recorded as they may influence stability by allowing increased infiltration of rainfall. In addition, the presence of clayey soils with high shrinkage and swelling characteristics must be taken into account with respect to the founding depth of shallow strip and pad foundations.

Observations should be made of existing nearby structures which may show evidence of ground movement in the form of cracks in walls and pathways, jamming of doors in buildings, displacement of fencelines, etc. A cautionary note should be made however, that some signs of distress may be due to poor workmanship rather than ground movement.

Trees growing in an area which has been displaced by substantial ground movement may exhibit trunk curvature. Such signs should be noted although it should be understood that a tree which has a non-vertical or irregular trunk does not necessarily imply ground movement; young trees growing in the shade of a larger tree (which may subsequently be removed or die) may tend to grow at an angle in order to reach light. A critical discussion of the relationship between deformed trees and soil creep is made by Phipps (1974).

3.3.2.3 Presence of fill

The presence of fill on a site may or may not be detrimental to stability, depending on the fill materials used, the method of placement and the provision or otherwise of underdrainage. On more recently developed sites, filling should have been carried out in accordance with good engineering practice as outlined in New Zealand Standard 4431:1978, Code of Practice for Earth Fill for Residential Development. This standard requires that the ground be properly prepared prior to filling (including the provision of any necessary drainage measures) and that the fill should be of good quality and be compacted to a predetermined standard. The extent and thickness of fill must also be recorded on 'as-built' plans.

In the case of older properties however, fill may have been loosely placed (e.g. end tipped from a truck) without compaction. Old filling may also contain unsuitable material such as vegetation which may decay with time and leave voids, possibly giving rise to subsidence. In addition, fill may have been placed over an old drainage course without providing adequate subsurface drainage.

The presence of fill may be suggested by an unnatural appearance to the topography or a marked contrast in vegetation type. The detection of fill on newer properties, where substantial earthworks have been carried out, may be much more difficult to detect although this information should be available from the developer or the local authority.

3.3.2.4 Hydrological features

Water is the principal triggering agent leading to slope instability. Consequently, observations regarding hydrological features both within and in the vicinity of a site are an essential part of a comprehensive field inspection. A site visit made directly after (or even during) heavy rainfall can be particularly enlightening; seepage, overland flow, ponding, etc. may only be evident at such times. As indicated in Table 1, page 49, the following observations regarding hydrological features are of particular importance.

The surface covering, both on and above a slope, has a significant effect on the ability of rain to infiltrate the slope and affect groundwater conditions. Vegetation is generally beneficial in this respect as it reduces soil moisture by evapotranspiration and interception. An impermeable artificial surface covering, e.g. concrete paving, above a slope may be wholly effective in preventing infiltration over the paved area providing of course it is not seriously cracked. Its benefit will be wasted however if no drainage is provided and the runoff from the paved area discharges on to the slope below.

The position of existing natural or artificial drainage courses should be noted in relation to the layout of the proposed building development. The size and nature of drainage courses may give an indication of their erosive potential which could lead to undercutting at the toe of a slope.

Springs and other seepage points may provide valuable information for establishing the groundwater profile. The relative rate of flow of water from such features should be recorded and should be complemented by remarks regarding weather conditions at the time of inspection and for the preceding month or so. This information will help to assess any likely fluctuations in groundwater levels.

Areas of existing waterlogging should be noted as well as areas which may be subject to waterlogging following periods of prolonged rainfall.

The positions of all water carrying services and other facilities (such as existing water storage tanks, effluent irrigation fields, soakage pits, etc.) should be recorded. The location of underground services may not be known precisely although their presence and approximate position may be inferred from manhole covers and possibly from linear depressions in the ground surface. The age and condition of service pipes should be assessed and any signs of leakage recorded.

For sites where slope stability is subsequently concluded as being marginal and where serious leakage from services could trigger movement, it may be necessary to relocate or modify existing services, in conjunction with other preventive measures, during site development.

3.3.2.5 Slope forming materials

Providing that a desk study has already been carried out, the investigator should already have a general understanding of the geology in the area of the site and should be able to make predictions regarding the nature of the slope forming materials. Observations made during the field inspection are aimed at confirming (or otherwise) this basic information and elaborating on it.

Exposures of the materials forming the subject slope are obviously the most relevant although useful information can also be gained from an examination of exposures elsewhere in the vicinity of the site. Exposures may be available in stream courses, cut banks, cliff faces, and the like.

A detailed discussion of schemes for the description and classification of slope forming materials is beyond the scope of this document. However, numerous schemes are currently employed, some of the most recent and comprehensive of which are listed in section 5.3 of this report. Unfortunately, these vary not only in the terms used but also in the definition of the same term. It is therefore vitally important that the system or terms used for the description of soil and rock be defined by the investigator in the assessment report.
The key to good description is to work systematically, and a list of suggested headings under which descriptions can be made is as follows :

Material :
(a) colour,
(b) grain size and other textural features,
(c) degree of weathering,
(d) strength,
(e) soil or rock type, and
(f) other characteristics, such as plasticity, moisture content, etc.

Exposure or Outcrop :
(a) geological structure, e.g. distribution of different lithologic types, folding of the bedding and uniformity of materials comprising individual 'layers' e.g. presence of boulders,
(b) spacing, orientation, continuity and aperture of discontinuities.

An example of detailed descriptions of materials forming a cliff face is given on Figure 2, Page 51.

A comprehensive discussion of data collecting techniques for rock slopes is given by Hoek & Bray (1977).

The terms 'soil' and 'rock' will be used in the description of the slope forming materials. There is however some disagreement over the use of these terms. They are firmly established in the vocabulary of the geologist, the engineer, the agriculturalist et al, although unfortunately the terms are used to mean different things. In this report these terms are used in the engineering sense. Terzaghi & Peck (1967) defined soils as "... an aggregate of mineral grains that can be separated by such gentle mechanical means as agitation in water" and rock as "... a natural aggregate of minerals connected by strong and permanent cohesive forces". Terzaghi & Peck qualified these definitions by adding, "Since the terms 'strong' and 'permanent' are subject to different interpretations, the boundary between soil and rock is necessarily an arbitrary one". They might also have added that, not only is the term 'permanent' subject to interpretation but, it can hardly be applied to natural materials which are subject to weathering. In the draft Method of Soil and Rock Description for Engineering Use (New Zealand Geomechanics Society [1985]) the following extract is relevant to the distinction between soil and rock (soft and hard) :

> "Often rocks with unconfined compressive strength values > 50 MPa are referred to collectively as 'hard' rocks and those < 25 MPa (especially < 10 MPa) are collectively referred to as 'soft' rocks. The boundary between soils and rocks is often arbitrarily taken at the boundary between very low and extremely low strengths, i.e. 1 MPa ...".

The strength based distinction between soil and rock as given above is adopted for use in this report, although it is emphasised that these strengths relate to the intact material whereas it is the mass characteristics, particularly the structural defects, of the slope forming materials that generally govern stability. This applies to both soil and rock.

3.3.3 Other Local Sources of Information

Local residents may be able to provide valuable site information. They may provide clues as to the cause of a nearby landslide or point out the existence of some site feature which has been hidden by subsequent regrowth of vegetation and which might otherwise have been overlooked. Such information should however be treated with some caution as there is a tendency for people to exaggerate events such as landslides, particularly with respect to their magnitude. Also, information regarding timing may be unreliable.

3.4 Subsurface Investigation

3.4.1 General

The subsurface investigation may be the most costly component of the stability assessment. To gain the maximum amount of information it should be designed on a geological basis preferably by the engineering geologist who carried out the field inspection and who should have the clearest idea of likely ground conditions and possible stability problems. The investigation must also be designed in the context of the proposed building development as there will be other considerations besides slope stability e.g. foundation design and possibly ground percolation potential for effluent disposal.

The design of the subsurface investigation may be influenced by the apparent stability of the slope as determined during the field inspection. For slopes which show signs of past or recent instability the investigation will be aimed primarily at defining the configuration and nature of the failure surface or zone. Also, the possible need for carrying out ground movement monitoring should be considered. For slopes which are apparently presently stable the task is less specific and hence, may be more difficult.

It is essential that the description of materials encountered in the boreholes or investigation pits is carried out by a person with geological training. Ideally, this work should be undertaken by the engineering geologist who made the field inspection and who will be able to relate the materials encountered in the boreholes or pits to those examined during the inspection. He would be in a position to modify the subsurface investigation on site if found necessary.

It is desirable that, for all types of subsurface investigation, samples of the materials encountered are collected. These may be of value for reference during report preparation and are essential where laboratory testing is required later. If samples are not collected and are subsequently found to be necessary the only option would be to return to the site and carry out additional drilling or excavation - thus adding considerably to the overall cost of the assessment.

The subsurface investigation techniques discussed here are limited to those that are normally carried out in New Zealand for slope stability investigations. They include hand auger boreholes, investigation pits, machine drilling and investigation shafts.

3.4.2 Hand Augering

This technique involves the use of a lightweight, easily portable, hand operated auger in which the auger head provides the cutting action for penetration and also enables disturbed samples to be collected for examination and, if required, for basic classification tests. Different size auger heads can be used to give boreholes ranging from 50 mm to 300 mm in diameter.

The depth of penetration that can be achieved varies depending on the ground conditions encountered, as well as the physical strength of the operator. Generally however, hand auger boreholes do not penetrate much more than a depth of 7 metres. Considerable difficulty may be encountered in augering below the water table due to collapse of the hole (casing is rarely used to support the hole) and the difficulty in recovering samples. The occurrence of rock or other hard fragments of only relatively modest size may also prevent penetration.

The main advantage of hand augering is that it is relatively inexpensive and invloves lightweight wquipment which can be easily manhandled. Consequently, it may be used on steep or thickly vegetated sites which would otherwise be inaccessible to a heavier drilling rig or excavation plant (unless of course the formation of access tracks was permitted).

During the course of hand augering the structure and fabric of the soil is virtually completely destroyed by the action of the auger. However, the disturbed material collected in the auger head may be described in terms of soil type, colour, plasticity, moisture condition, etc., and this information provides a reasonable idea of changes in material type throughout the hole.

Soil descriptions are generally supplemented by hand shear vane tests which give soil strengths in terms of undrained shear strength. The size of the vane is relatively small, however, and the presence of small gravel fragments may give misleading results. In addition, it may not be easy to take readings at the most appropriate levels in the auger-hole; thin bands of softer material which may be particularly significant to the stability of the slope are easily missed. Nevertheless, if tests are properly carried out (especially with respect to rate of rotation of the shear vane) at closely spaced depth intervals, e.g. every 300 mm, they provide a very useful profile of relative strength of the slope forming materials.

Thin walled metal tubes may be fixed to the end of the auger rods (in place of the auger head) and driven or pushed into the base of the hole to collect relatively undisturbed samples. Strength tests may be carried out on these samples to determine soil strength parameters although it should be noted that the use of small diameter tube samples may result in considerable sample disturbance due to wall friction, particularly in the case of sensitive soils.

The main advantage of hand augering is that, with the exception of the occasional tube sample, the material collected by the auger is in a disturbed condition and may comprise a mixture of soil from adjacent levels in the hole. Hence, the structure of the soil is obscured, changes in soil type cannot be precisely defined and, more importantly, 'soft' or 'weak' zones or layers, which may represent potential failure planes, and which may be very thin, may be missed. Despite these serious shortcomings, hand augering remains the most common shallow subsurface investigation technique in New Zealand, even on sites where slope stability is a critical factor. An alternative low cost investigation technique that provides continuous core recovery is required to replace the traditional method of hand augering.

3.4.3 Investigation Pits

Investigation pits may be hand dug or excavated with a back-hoe or bulldozer. Hand digging is less common because of the time involved and depth limitations. However, for sites which are inaccessible for a mechanical excavator and/or where only a shallow depth is required, hand digging may be appropriate.

Pits can be excavated to depths of between 4 m and 7 m depending on the type of plant used. Larger excavators not only provide deeper pits but are also generally track mounted and may therefore be able to reach pit locations which would be inaccessible to a lighter wheeled excavator. The cost of hire for a heavier machine would of course be greater and would probably include the use of a transporter.

In addition to the limitations imposed by the type of plant, the occurrence of rock and of significant water inflow also restricts the depth of investigation pits. In addition, it is unsafe to enter deep pits that are not properly supported; the safety aspects of trench excavation are covered in a booklet published by the Department of Labour (Safety in Construction No.5, Code of Practice for Excavation on Construction Work under the Construction Act 1959). The need to provide shoring will substantially reduce the amount of investigation work that can be achieved in a given time.

Despite the practical difficulties, investigation pits are greatly preferred to other techniques for shallow investigations. They allow the investigator to examine in situ materials closely to make observations regarding not only the types of materials but also their structure. Any weak layers can be identified, closely examined and their disposition measured. Also, of equal importance, direct observations can be made of groundwater flow - location, direction and rate. In situ tests, e.g. hand shear vane tests, can be carried out in the walls or base of the pits and undisturbed samples can be taken by forcing thin walled tubes into the base of the pit or by carefully excavating block samples.

Investigation pits are rarely backfilled properly; normally, the pit is refilled with the excavated material using only the force of the back-hoe bucket for compaction. In addition, the ground directly surrounding the pit may suffer considerable disturbance due to the excavation. As a result, problems of differential settlement may arise where shallow foundations are constructed on or adjacent to the site of an investigation pit. Investigation pits should therefore be sited with due consideration to the layout of the proposed building(s). An investigation pit that is not satisfactorily backfilled may also promote infiltration of surface runoff into the ground.

3.4.4 Machine Drilling

The two techniques discussed above will, in many circumstances, be adequate to define subsurface conditions at the site. However, there will be many cases where a deeper subsurface investigation is required and, under these circumstances, machine drilling is normally carried out.

There are many different machine drilling techniques but, for the purposes of site investigations for slope stability studies, the only satisfactory method is one which allows continuous core sampling throughout the depth of the borehole. This is normally achieved using machine drilling rig which provides a rotaty action combined with a downward thrust (normally by hydraulic feed). Such a rig should be capable of rotary core drilling of both soil and rock, performing in situ tests and of taking undisturbed soil samples. The diameter of the holes drilled will depend on the type of tests, if any, which are to be carried out on the recovered core samples.

Soil strength materials are normally recovered using a steel tube, called an open barrel, which may be pushed into the ground under hydraulic pressure or other means. The soil sample is retained in the barrel by friction and, if a non-return valve is provided, by suction. Retention of the sample may be assisted by use of a spring type core catcher although this may cause damage to the sample.

Rock strength materials are normally recovered using a triple-tube core barrel which is advanced through the ground by combined high speed rotation of the barrel and hydraulic pressure. A cutting action is provided by the bit at the end of the barrel which may be impregnated with diamonds or fitted with teeth depending on the hardness of the material being drilled. Cuttings are flushed to the surface by water under pressure which is emitted from ports in the drilling bit and passes up the outside of the drill string. The most suitable core barrel for obtaining good quality core is the triple - tube barrel which has an inner split liner into which the core is fed during drilling. The retractor core barrel is a specialised form of the triple-tube barrel; for particularly weak materials the liner is advanced beyond the end of the bit thereby protecting the core from washing by the flushing water. The use of drilling mud or foam, as an alternative flushing medium, may also improve the quality of core samples. Foam is particularly appropriate for coring materials which may swell in the presence of water.

Machine drilling is expensive; the hire of a machine drilling rig for one day may, at present rates, cost about $1,200. Moreover, the amount of drilling that can be achieved within that period of time may seem disproportionate to the cost. Machine drilling is therefore normally restricted to investigations for larger building developments or other sites of high value.

3.4.5 Investigation Shafts

As with investigation pits, large diameter shafts also provide the investigator with an opportunity to examine materials in situ. In this respect, they are preferred to machine boreholes.

Investigation shafts are generally excavated using the same plant used for constructing bored piles, e.g. bucket-auger, continuous flight auger or cable-tool boring plant. Such plant however is as costly to hire as a machine drilling rig and, being somewhat specialised, may not be readily available in the area in which the investigation is being carried out. Hence, additional transportation costs may be incurred. A further practical disadvantage of this technique is that the plant is generally heavy and cumbersome and therefore difficulties may arise on steep sites or sites with restricted working space with respect to access and subsequent manoeuvrabitlity.

As with investigation pits, the safety of the investigator is of prime importance and casing should be installed progressively as the shaft is deepened. It is common practice for the walls of the shaft to be examined in 1 metre stages. Except where ground conditions are favourable, it is considered dangerous for shaft inspection to be carried out, aided by pumping, below the water table because of the danger of the shaft floor bursting.

3.4.6 Groundwater Monitoring

The importance of water as a destabilising agent has already been emphasised and any slope stability investigation would be seriously lacking without consideration of groundwater conditions. The comments made below refer specifically to observations in boreholes (whether drilled with a hand auger or a machine drilling rig). However, they may be equally applicable to investigation pits or other forms of has been completed.

During the course of drilling, the water level in a borehole may not be a true reflection of the groundwater level; water may be removed from the hole during the drilling operation or, in the case of machine drilling, may be introduced for flushing drill cuttings. Depending on the permeability of the ground, it may take some time for the water level to stablise. This may happen minutes or days after the drilling has been completed.

Ideally therefore, any change in water level should be monitored. Also, the ground water level may fluctuate considerably depending on weather conditions; during periods of heavy, prolonged rainfall the level may rise significantly. If a stability analysis is to be carried out, accurate prediction of the likely highest groundwater level will have a significant effect on the reliability of the analysis.

Unfortunately, extended water level monitoring is often not possible, because of the urgency with which the assessment is required. However, where time is available it is recommended that monitoring is carried out.

Monitoring water levels in boreholes normally requires the installation of some form of 'casing' to allow water level measurements to be made even if the hole collapses. Usually, PVC pipes are inserted into the hole immediately after drilling has been completed. The pipe is perforated for a certain length at the base to allow water to enter. In order to prevent the entry of silt, the perforated section of the pipe should be screened with graded granular filter material or sheathed with filter fabric. The borehole should be capped at the ground surface to prevent the entry of rainfall runoff which could affect the water level in the hole. Although probably not as significant as with investigation pits, the entry of rainfall runoff into open boreholes could be detrimental to stability.

Post-construction groundwater monitoring should be considered an essential activity where prescribed remedial or preventive works involve drainage measures (refer Section C.2 of Appendix C). This will enable the effectiveness of the measures to be checked.

3.5 Ground Movement Monitoring

If the field inspection reveals evidence of current or recent movement on a slope there may be a need to monitor the rate and direction of movement. Such data may be used to help define the nature of the movement, particularly in relation to rainfall events. Ground movement monitoring may also be used to help define the depth and geometry of the failure surface of zone, as discussed by Carter & Bently (1984). As with monitoring needs to be carried out over a long period of time in order to be meaningful.

A large variety of methods are employed, ranging from conventional surveying to sophisticated, highly sensitive devices that are installed in boreholes within the unstable area. A discussion of the various types of instrumentation available for ground movement monitoring is beyond the scope of this document. However, many texts on this subject are available; reference can be made to Chapter 5 in Hanna (1985) or Chapter 5 by Franklin in Brunsden and Prior (1984).

3.6 Sampling

Samples of the materials encountered during the site investigation are useful for reference during report preparation and are obviously essential if laboratory testing is to be carried out. At the site investigation planning stage it may be difficult to predict whether laboratory testing will be required. It is good practice to collect samples as a matter of course; if this is not done, additional costs will be incurred in drilling additional holes or excavating further pits later, simply to collect samples.

The main purpose of samples in connection with slope stability studies is for determination of shear strength parameters for use in stability analyses. Conventional tube and block sampling techniques have been mentioned in the foregoing text. Whatever sampling technique is used, emphasis should be placed on minimising disturbance to the sample. Sample disturbance can be caused by the relief of in situ stresses and mechanical interference during sampling, by clumsy handling during transportation and by disturbance during preparation of the sample for laboratory testing. In general, the effect of sample disturbance will be a reduction in the strength parameters measured from the testing.

The sealing of samples to preserve their in situ moisture content is as important as minimising sample disturbance. Sample desiccation, caused by inadequate sealing, or subsequent effect on test results. It is therefore considered good practice to test samples as soon as conveniently possible following sampling. This will also minimise any other possible time dependent changes in the sample.

3.7 Photography

The adage, 'A picture is worth a thousand words' is particularly relevant to site investigations. Good quality colour photographs of the site, surface exposures of slope forming materials and of the materials recovered from boreholes and test pits are valuable for reference during report preparation. They may also be essential in resolving disputes associated with the subsequent performance of the slope and of any remedial or preventive works.

4. ASSESSMENT PROCEDURES

4.1 Introduction

The information collected during the site investigation forms the basis for the stability assessment. The more detailed the investigation, the more thoroughly the slope will be understood and hence, the more reliable should be the assessment of its stability. Restrictions on time and funds may however place a constraint on the amount of investigation that can be undertaken. In reaching conclusions in the stability assessment the investigator must be conscious of any limitations in the knowledge of the slope, whether in terms of its geological composition or of the groundwater conditions. Any such limitations should be brought to the attention of the client and clearly stated in the assessment report.

Clients who are unable or unwilling to pay for the necessary, more costly, level of investigation must appreciate the limitations imposed on the investigator in carrying out the stability assessment and must accept the consequent higher risk.

Whatever level of site investigation has been carried out, the investigator must address the following questions in making the assessment :

a) Is there evidence of past or active instability on the slope ?
b) Where there is evidence of instability, is such instability likely to either continue or recur? The investigator must be able to define the nature of instability in order to resolve this question.
c) Where there is no evidence of instability, can a potential form of instability be recognised? Also, what is the likelihood of that form of instability taking place.
d) In all cases, will the proposed building development, including associated site formation works, be beneficial or detrimental to stability?
e) In all cases, will the proposed building development be affected in the event of slope instability?

In the following Sections, 4.2 to 4.4, the factors involved in answering these questions are discussed. The initial assessment is of an empirical nature, involving an examination of all the observations made during the site investigation. The experience of the investigator, in terms of his knowledge of instability problems in the locality and of the stability characteristics of the slope forming materials, is relied on in making an empirical assessment. Although the conclusions reached may be perfectly valid it is often necessary to subsequently undertake an analytical form of assessment as discussed in Section 4.5.

4.2 Potential for Instability

The first step in the stability assessment is to determine whether there is potential for instability given the ground conditions as defined by the site investigation.

In a geological sense, all slopes have potential for instability as a result of the natural processes of denudation. However, in the context of a stability assessment for a proposed building development, the investigator must establish the likelihood of that potential being realised during the 'lifetime' of the building. The lifetime of a building is normally taken to be 100 years although, in reality, the period may vary considerably from place to place depending on such factors as demand for land, changes in living style, etc. Although most natural processes which bring about changes in slope profiles are generally extremely slow and can be discounted in connection with building developments, there may be exceptions. Coastal erosion for example can be quite rapid in certain locations and an estimation of the rate of erosion should be carried out for all coastal sites. Also, the normally slow process of weathering can be greatly accelerated, with a resulting reduction in the strength of slope forming materials, by exposing subsurface materials through excavation work carried out during site formation works.

In determining the potential for instability the investigator seeks to define a possible failure mechanism within the framework of the slope model (the slope model is the form and composition of the slope - the topographic profile and the nature and distribution of the slope forming materials as established from the site investigation). This step in the assessment process requires, above all, an understanding of geological principles and more specifically of mass movement processes in establishing the slope geology and assessing possible failure mechanisms. Consequently, it should be undertaken by an engineering geologist and preferably one who is familiar with local geological conditions and stability problems.

A brief summary of the more common types of slope movement and the ground conditions with which they are normally associated is provided in Section B.1, Appendix B, of this report. The reader is also directed to the Reference section (pages 43 - 48) and particularly to those texts on the subject of slope stability problems in this country. For example, Appendix II of 'Slope Stability in Urban Development', compiled by Taylor, Hawley and Riddolls (1977), provides a valuable discussion of stability problems peculiar to selected areas of New Zealand. In the discussion of the Auckland region, the common occurrence of shallow landslides in the residual soil mantle, overlying bedrock of the Waitemata Group, is highlighted. The section on the Christchurch area on the other hand discusses the dispersive nature of the loess soils occurring in that area and the influence of this property on tunnel erosion and slope instability.

The investigator not only seeks to recognise a potential failure mechanism but also to assess the severity of the hazard that it may represent. Again, this is an empirical assessment relying on the judgement and experience of the investigator and his knowledge of the factors which influence stability. Background information regarding causes of instability is provided in Section B.2 of Appendix B.

An examination of other slopes in the area may be helpful in making this assessment. Evidence of instability on slopes which are similar in terms of geological, hydrological and topographic features, may indicate that the slope being investigated could also be potentially unstable. It should be emphasised however that instability elsewhere may be due to specific localised ground conditions.

Careful field checking (possibly involving subsurface investigation) is needed before definite conclusions can be drawn from a comparative study of this kind.

The assessment of slopes which show evidence of past or active instability is in some respects simpler than for those that show no signs of past movement. The surface evidence of movement provides an indication of not only the type of movement but also gives clues as to the depth and configuration of the failure surface or zone. It is expected that the subsurface investigation will have been designed to confirm this. Where time or financial constraints have not allowed an adequate understanding of the type and nature of slope movement it is wise to be conservative in making an assessment of the risk of continuing or recurring movement.

On the basis of the initial empirical assessment, the slope may be put into one of the following categories :

a) Slopes which show evidence of past or active instability
b) Slopes in which a potential failure mechanism can be recognised
c) Slopes of uncertain stability in which a potential failure mechanism cannot be positively identified, and
d) Slopes with no potential for instability.

This categorisation is based on the assessment of the slope in its existing condition. However, as discussed in the following Sections, the stability of the slope must also be examined in the context of the proposed building development, and the consequence of any future instability must also be taken into account.

4.3 Influence of Building Development

The previous section examined the stability assessment of slopes in their existing condition. The investigator must also consider the effects of the modifications to the land due to the building construction and associated site formation works.

The following modifications should all, to a greater or lesser extent, be beneficial to stability:

a) The construction of retaining walls,
b) A reduction in slope height or slope angle by excavation during site formation works,
c) The pinning effects of piled foundations,
d) The covering of part of the site by house construction or the formation of driveways and other paved surfaces, and
e) An improvement to natural drainage by collecting surface water in pipes or lined trenches and discharging it away from the slope.

Conversely, the modifications listed below may be detrimental to stability:

a) The formation of unsupported slopes,
b) The surcharging of slopes with fill or structures,
c) The discharge of water onto or into a slope by irrigation or effluent disposal in poorly sited soak holes, and
d) The removal of vegetation.

The client should be advised if the building proposals are likely to result in a reduction in the stability of the slope. If this is the case, appropriate modifications to the proposal should be made. If this is not possible, a considerable increase in building costs may arise due to the need for preventive measures.

Conversely, the building proposals may result in a considerable improvement in stability; if the building works eliminate any risk of instability, as for example the removal of a slope by earthworks, then building development could be allowed to proceed without the need for further investigation or assessment.

4.4 Consequence and Avoidance of Instability

The risk of slope instability on a site may not necessarily pose a threat to the proposed building development or to other existing buildings, either on or in the vicinity of the site. Clearly, if there were a possibility of a small localised slope failure remote from the proposed building site, this would not influence the feasibility and subsequent safety of the proposed building.

In assessing the consequence of instability, allowance must be made for any uncertainty regarding the likely extent of the land that would be affected by possible future movements. In cases where the instability may take place downslope of the building site, the possibility of retrogressive movements, following the initial movement, should be taken into account. On the other hand, where there is a possibility of a failure upslope of the building site, the likely travel distance of the slide debris should be considered. Slope failures elsewhere in similar materials may give a good guide to the likely travel distance of slide debris.

If the proposed building is likely to be affected during its lifetime, the least costly solution would be to avoid the potential hazard. Avoidance involves siting the building a safe distance from any area that may be affected by movement or the debris from a movement. The investigator may stipulate in his report that a building line restriction should be imposed whereby, for example, the building may not be sited closer than a certain distance to the crest or toe of a slope. The avoidance solution will however only be possible where the stability problem is localised (does not cover the whole or a substantial part of the site) and where the size of the property provides scope to modify the building layout and avoid the problem area. Figure 3, page 52, illustrates a simple example of using the avoidance solution in overcoming potential instability hazards.

In cases where the instability hazard is unlikely to affect the proposed building or where the building layout has been modified to avoid the hazard, building development may be allowed to proceed. It must be recognised however that the stability problem or potential problem still exists and, as such, may place a severe constraint on any future building development plans e.g. an extension to the initial building or construction of an additional building. The cost of remedial/preventive measures, required later to accommodate future development, could be substantially higher than at the present. Also, the presence of the original building could create access difficulties for the plant used to carry out the measures. In addition, despite the fact that a future slope failure may in no way jeopardize the building itself, it will undoubtedly have an unfavourable psychological effect on the residents and could also adversely affect the value of the property.

4.5 Analytical Assessment

4.5.1 General

Discussion so far has centred on the use of observation, judgement and experience in defining the most likely form of instability and assessing the likelihood of that instability occurring. The conclusions reached from this empirical assessment, although they may be perfectly valid, can only be expressed in qualitative terms. An analytical assessment on the other hand allows the relative stability of a slope to be expressed numerically in terms of a factor of safety. The factor of safety of a slope may be defined as the ratio of the available shear strength of the material along the critical failure surface to the shear stresses acting on that surface. Put another way, the factor of safety measures the factor by which the shear strength would have to be reduced in order to bring the slope to the point of imminent failure.

Stability analyses may be carried out using a 'total' or 'effective' stress approach. In the total stress approach it is assumed that, during shearing of the materials along the failure surface, no drainage of pore water occurs. This approach is applicable to the assessment of short term stability whereas, in the context of this report, it is the long term stability of slopes that is of prime importance. For this purpose, the effective stress approach, which allows for the inclusion of groundwater conditions in the analysis, is most applicable.

Although, ideally, an effective stress analysis should be carried out, the costs of determining the required input data may be very high. The effective stress strength parameters can only be determined by expensive laboratory testing and, in order to accurately define groundwater conditions in the slope, long term monitoring of piezometers is required. In practice, where the budget does not allow for laboratory testing, effective stress strength parameters are often assumed, using values which appear to be appropriate for the slope forming materials. If this is done it is recommended that a sensitivity analysis be undertaken. This involves carrying out several stability analyses using a range of c' and Ø' values. The investigator must then examine the values needed to achieve a suitable factor of safety and assess how realistic these are. An accurate understanding of groundwater conditions is difficult to achieve unless provision has been made for long term monitoring of piezometers installed in the slope. This is frequently not possible and consequently, assumptions have to be made with respect to the likely worst groundwater conditions that the slope may experience. Again, a sensitivity analysis may be carried out to gauge the effects of variations in groundwater conditions on the factor of safety.

Despite the less appropriate nature of total stress stability analyses for determination of long term stability, the input data is much simpler to determine. A total stress analysis does not require determination of groundwater conditions and strength parameters can be measured by simple, inexpensive shear vane tests.

4.5.2 Methods of Stability Analysis

The methods of stability analysis most commonly used at present are based on the concept of limiting equilibrium in which the stability of a slope is analysed in terms of the forces operating in and on the slope at the point of failure (when the factor of safety is equal to one). Finite element methods of stability analysis, which involve examination of the deformations that occur throughout a slope as it tends towards failure, are not discussed here. Such methods are very time consuming (and hence costly) and are rarely warranted for slope stability studies in connection with building developments. The commonly used methods of stability analysis are briefly discussed below.

4.5.2.1 Infinite slope analysis

An infinite slope analysis is really only suitable for long slopes where the potential failure surface, which is assumed to be planar, runs approximately parallel to the ground surface. In addition, the depth to length ratio should be small, i.e. the failure surface should be at a relatively shallow depth. It is therefore most suitable for shallow translational slide movements involving, for example, the movement of a thin soil mantle over underlying rock or other harder material or, conversely, the movement of a crust of relatively stiff soils over underlying soft soils.

This method may be applied to both cohesive and cohesionless soils and has the advantage that it can be carried out rapidly by simple hand calculation. The analytical procedure is explained by Lambe & Whitman (1969), and Chowdhury (1978). The chief disadvantage of the method is that it is limited to the geological situations outlined above.

4.5.2.2 Sliding block analysis

A sliding block analysis is also suitable for slopes where there is a well defined potential planar failure surface, e.g. an interface with underlying substantially stronger material. However, in this method, the failure surface need not be parallel to the ground surface and the results are not sensitive to the depth/length ratio.

The sliding mass is divided into two or more blocks and the equilibrium of each block is considered independently using interblock forces. The analytical procedure is explained by Chowdhury (1978).

4.5.2.3 Method of slices

The Bishop Method of Slices assumes a circular failure surface. The mass overlying the failure surface is divided into a number of slices and the force equilibrium for each slice is considered. The rigorous method assumes values for the vertical forces on the sides of each slice until all equations are satisfied. In the simplified method however the resultant of the vertical forces on each slice is assumed to be zero. Numerical errors can occur in the application of the simplified method when the inclination of the base of a slice is negative (as may occur near the toe of a deep-seated slip surface) and where high pore pressures are assumed. However, providing that the analyst is aware of these possible sources of error, the Simplified

Bishop Method has been found to give comparable results to more rigorous methods.

In addition to the Bishop Method, the Ordinary Method of Slices (otherwise known as the Swedish Circle Method or Fellenius Method) also caters for failure surfaces of circular shape. This method has however been found to give very conservative results for deep failure surfaces and also where high pore-water pressures are assumed.

In choosing a method which assumes a circular failure surface the investigator should satisfy himself that it is a realistic failure mode; circular rotational movements are generally confined to truly homogeneous materials.

The Janbu Method (Janbu 1954, 1957) allows for <u>failure surfaces of arbitrary shape</u>. As with the Bishop Method, the potential failure mass is divided into slices. In the Rigorous Janbu Method the force equilibrium on each slice is considered. However, assumptions must be made regarding the line of action of interslice forces. In the Simplified Janbu Method interslice forces are not included and the calculated factor of safety has to be corrected to allow for these forces.

The rigorous Janbu Method is one of the most commonly used methods of stability analysis. It is particularly useful as it provides a rigorous solution for an arbitrary failure surface in a non-homogeneous soil. Morgenstern and Price (1965) and Sarma (1979) have presented methods of analysis also applicable to failure surfaces of arbitrary shape. However, both methods, although possibly more accurate than the Rigorous Janbu Method, are very detailed and time consuming. Generally, the reliability of the input data available does not warrant the use of such methods.

4.5.2.4 Stability charts

A variety of stability charts are available and are useful for providing a first approximation of stability. Generally, stability charts assume the following conditions:-

a) The slope surface is planar and the ground above the crest and below the toe is horizontal,

b) The soil is homogeneous, and

c) The critical failure surface is circular.

From a comparison of various charts, it has been found that those developed by Cousins in Chowdhury (1978) are preferable in terms of reliability of the solution given, their simplicity of use and the wide range of slope angles and pore pressures that may be used. The charts are presented by Chowdhury (1978) together with a discussion of their use. This text also explains the various other stability charts available.

4.5.2.5 Rock slope stability analysis

Appropriate methods of analysis for various modes of failure in rock slopes are given by Hoek & Bray (1977).

4.5.3 Input Data

The following information is required for carrying out most methods of stability analysis. The accuracy with which the data needs to be defined will depend on the requirements of the particular type of method used.

4.5.3.1 Slope profile

A stability analysis should be carried out on the critical section(s) on the slope. The critical section is that part of the slope which appears to have the highest risk of instability in combination with the worst consequence of failure. The slope profile should be measured to the highest degree of accuracy commensurate with the accuracy of the other input data.

4.5.3.2 Groundwater conditions

Relatively minor variations in the position of the groundwater table can have a significant effect on the calculated factor of safety. For high risk slopes it is recommended that the slope be analysed under the likely worst groundwater conditions. In addition, it may be necessary to take account of the possible development of perched groundwater.

A sensitivity analysis may be carried out to determine the effects of variations in the groundwater table on the factor of safety. This is particularly useful for examining the effectiveness of drainage measures to improve the stability of a slope.

4.5.3.3 Distribution of slope forming materials

Provided that an adequate subsurface investigation has been carried out, sufficient information should be available to construct a realistic 'geological model' for the slope (the distribution and nature of the slope forming materials) and to establish the most likely form or forms of failure that could occur within the model thus defined.

4.5.3.4 Physical and mechanical properties of the slope forming materials

The only properties which need to be defined for a slope stability analysis are unit weight and shear strength. Other properties such as plasticity, particle size distribution and permeability may be required for classification purposes, correlation, etc., but are not essential input data for a stability analysis.

Unit weight can be measured by in situ tests or laboratory tests on undisturbed tube samples. Methods of determining shear strength parameters are outlined in the following section.

4.5.4 Shear Strength Determination

The most common type of laboratory test for the determination of effective stress strength parameters is the triaxial test. This test allows the stresses applied to the sample to be controlled and also permits measurement of the pore water pressure developed in the sample during shearing. The triaxial test is described in detail by Bishop and Henkel (1976). There are two types of triaxial test that are appropriate for determining the shear strength of soils in terms of effective stresses, these being:-

(a) consolidated undrained test with pore water pressure measurement and

(b) consolidated drained test.

The type of test should be chosen to suit the soil type being tested. For example, in the drained test, no build-up of pore water pressure is permitted and therefore, for soils of low permeability, such tests could take several days to reach failure. For such soils, the undrained tests would therefore be more convenient.

Normally, three sub-samples of the same material are tested at different stress levels and the results plotted together to define the shear strength parameters, c' and Ø'. However, multistage tests can be carried out in which the deformations occurring in a single sample are measured at increasingly higher levels of stress. Great care is required however to ensure that the sample is not overstrained at the lower stress levels. Multistage tests are particularly useful where limited samples are available or where the soil is heterogeneous.

Direct shear tests may also be used to determine shear strength parameters in terms of effective stresses, providing they are carried out at a sufficiently slow rate to avoid a build-up of pore water pressure (i.e. to simulate drained conditions). They are particularly useful for measuring the shear strength on a predetermined plane in a material by trimming samples at the correct orientation. Although it may not be possible to achieve full saturation of the sample in this test, a high degree of saturation can be achieved by soaking the sample for a sufficiently long period prior to testing.

For a total stress analysis, shear strength is generally determined from in situ shear vane tests which are both quick and inexpensive to perform. However, great care should be exercised in carrying out these tests, the results of which can be subject to error due to the presence of large size fragments in the soil being tested or through careless testing by the operator. Total stress strength parameters may also be determined in the laboratory by unconfined compression (uniaxial) tests or undrained triaxial tests.

All laboratory testing should be carried out in a laboratory registered by the Testing Laboratory Registration Council of New Zealand (TELARC) and which is accredited for carrying out the required tests in accordance with its terms of registration.

4.5.5. Results of Stability Analysis

4.5.5.1 Limitations of results

The contention that a factor of safety quoted for a particular slope is a true expression of the stability of that slope is a serious misconception. A single factor of safety is applicable only to the failure surface analysed and for the specific conditions assumed in the analysis. Hence, by varying the configuration of the failure surface, the soil strength parameters and the groundwater conditions, an individual slope can be represented by a whole range of factors of safety. In addition, for one specific set of input data the calculated factor of safety may vary considerably depending on the method of analysis used.

There is generally some uncertainty attached to all of the various input data used in an analysis. The soil strength parameters are the major source of uncertainty, particularly where the budget does not provide for laboratory testing and, as a result, strength parameters have to be assumed. Even where laboratory testing is carried out, errors in measuring these important properties may arise due to sample disturbance, testing errors or inappropriate test conditions.

The groundwater conditions assumed in the analysis are another major source of uncertainty. Unless groundwater levels have been monitored over a long period of time (which is not commonly done), the position of the groundwater table adopted for the analysis will have to be based on the water levels measured at the time of the subsurface investigation, with some adjustment to account for the expected rise in the water table under the likely worst rainfall conditions.

In some cases, the likely failure mode and the position of the corresponding failure surface can be predicted with reasonable accuracy, e.g. where there is a well defined soil/rock interface and it is anticipated that any failure would involve translational movement along that surface. In many cases however, it may not be possible to define a specific potential failure surface and, under these circumstances, a critical failure surface will be chosen from the analysis of several trial surfaces. The critical surface is the trial failure surface that yields the lowest factor of safety. The question of whether this critical surface represents a realistic failure mode introduces another element of uncertainty regarding the results of the analysis.

It will be evident from the above that considerable judgement is required in selecting a factor of safety appropriate for a given slope. Accordingly, an analytical assessment is in many ways no less subjective than an empirical assessment. The investigator should clearly state in the report all uncertainties and assumptions regarding the input data used in the analysis and thus, the limitations of the quoted factor of safety.

4.5.5.2 Acceptability of factor of safety

In view of the generally uncertain validity of the input data used in a stability analysis, and the fact that different methods of analysis commonly yield different factors of safety using the same input data, it is doubtful that a factor of safety truly represents the relative stability of a slope. Factors of safety of less than unity are frequently derived from the analysis of stable slopes, despite the rigorous investigation and testing procedures used to define the input data. Conversely, failures have occurred on slopes which according to analysis, using input data based on equally rigorous investigation and testing procedures, have yielded factors of safety substantially greater than unity.

Notwithstanding the above, the factor of safety is a traditional and very useful basis for decision making and design in engineering activity and is especially useful as a comparative tool for defining the beneficial effects on slope stability of remedial or preventive measures. In addition, the use of a stability analysis provides the investigator with a means of examining the level of understanding achieved with respect to the subject slope. In this sense it is a useful quality assurance procedure.

It is considered undesirable to impose a rigid system whereby compliance with a minimum factor of safety is required in order to gain approval for a building permit. As has been found in other countries, such an approach can result in manipulation of the input data in order to achieve the required factor of safety. Such manipulation is not considered good engineering practice except for cases where the investigator, mindful of the uncertainties inherent in the input data, seeks to express numerically the adequate stability of a slope which his judgement tells him is quite satisfactory.

It is considered preferable to examine the results of a stability analysis on a case by case basis and allow the investigator to use his judgement in assessing the validity of the results in terms of the equivalence of the assumed analytical model to reality.

5. REFERENCES

5.1 Other Relevant New Zealand Documents

The reader's attention is drawn to two other documents on the subject of slope stability which have been published in New Zealand and which cater for New Zealand conditions. Unlike this report, which is particularly concerned with the assessment of slope stability, these publications provide general guidance on slope stability matters.

'Slope Stability in Urban Development', compiled by Taylor, Hawley and Riddolls (1977) and published by the DSIR, examines slope stability primarily in connection with urban development and particularly subdivision development. It covers, in general terms, a number of the topics discussed in this document including causes and evidence of slope instability. It also discusses the legal and insurance aspects of slope instability. The appendices to the handbook, which cover local stability problems in the major urban areas, are of particular value. The second reference is a small booklet, prepared and published by the New Zealand Geomechanics Society for the Earthquake and War Damages Commission, entitled 'Stability of House Sites and Foundations; Advice to Prospective House and Section Owners'. This booklet discusses, in simple terms, the various ground related problems that may affect private dwellings. With respect to slope stability, it very briefly examines the signs to be looked for on sloping and cliff-top sites which may indicate instability. It also makes cautionary remarks about site development and maintenance of stability.

5.2 General List of References

Given below is a full list of those references cited in both the main text and appendices of this report together with a few additional references, relevant to the subject of slope stability, which are considered of particular value.

Bishop, A.W. and Henkel, D.J. (1976). The Measurement of Soil Properties in the Triaxial Test. Edward Arnold, London, 227p.

Brunsden, D. and Prior, D.B. (1984) Slope Instability. Wiley, Chichester, 620p.

Carter, M. and Bentley, S.P. (1984). The Geometry of Slip Surface Beneath Landslides: Predictions from Surface Measurements. Can. Geotech. J. Vol. 22, pp 234-238 (1985).

Chowdhury, R.N., (1978). Slope Analysis. Elsevier Scientific Publishing Company, Amsterdam, 423p.

Crozier, M.J. (1984). Field Assessment of Slope Instability. p.103-142 in Brunsden D. and Prior, D.B. (eds). Slope Instability. Wiley, Chichester, 620p.

Department of Labour, New Zealand (1975). Code of Practice for Excavation on Construction Work under the Construction Act 1959. (No.5 in Series, 'Safety in Construction'). Department of Labour, Wellington, 95 p.

East, G.R.W. (1974). Inclined Plane Slope Failures in the Auckland Waitemata Soils; Three Case Histories with Different Remedial Measures. pp 5.17 - 5.24 in Proc. Symposium on Stability of Slopes in Natural Ground, New Zealand Institution of Engineers, Wellington

Geotechnical Control Office (1984). Geotechnical Manual for Slopes. Hong Kong Government Printer, 295p.

Gray, D.H. and Leiser, A.T. (1982). Biotechnical Slope Protection and Erosion Control. Van Nostrand Reinhold, New York, 271p.

Hancox, G.T., (1974). Geological Aspects of Slope Stability. Unpublished Internal report, Engineering Geology Section, New Zealand Geological Survey, Department of Scientific and Industrial Research.

Hanna, T.H. (1985). Field Instrumentation in Geotechnical Engineering. Trans Tech Publications, Clausthal-Zellerfeld, West Germany, 843p.

Hoek, E. and Bray, J.W. (1977). Rock Slope Engineering (Revised 2nd Ed.). The Institution of Mining and Metallurgy, London. 402 p.

Janbu, N. (1954), Application of Composite Slip Surface for Stability Analysis. European Conference on Stability of Earth Slopes, Stockholm.

Janbu, N. (1957), Earth Pressure and Bearing Capacity Calculations by Generalized Procedure of Slices, Proc. 4th Int. Conf. Soil. Mech. Found. Eng., London, 2; 207-212.

Joyce, D. Michael (1982). Site Investigation Practice. E & F.N. Spon Ltd, London, New York 369 p.

Johnston, M.R. (1979). Geology of the Nelson Urban Area. New Zealand Geological Survey, DSIR, Wellington, 52 p.

Lambe, T.W. and Whitman, R.V. (1969). Soil Mechanics. John Wiley and Sons Inc., New York, 553 p.

Morgenstern, N.R. and Price, V.E. (1965). The Analysis of the Stability of General Slip Surfaces. Geotechnique, 15, 1; pp 79-93.

Peck, R.B. (1977) Keynote address to Penrose Conference (Geol. Soc. of Am. and ASCE joint meeting) Vail, Colorado, Oct.9, 1977. Unpublished.

Phipps, Richard L. (1974). The Soil Creep - Curved Tree Fallacy. Journal Research U.S. Geological Survey, Vol 2, No.3.

Riddolls, B.W. (1977). Slope Stability of Western Hills, Hutt Valley : An Aid to Town Planning. Unpublished report by the Engineering Geology Section, New Zealand Geological Survey, Department of Scientific and Industrial Research.

Sarma, S.K. (1979). Stability Analysis of Embankments and Slopes. Geotechnique, 23; pp 423-433.

Schuster, R.L. and Krizek, R.J. (1978). Landslides: Analysis and Control. Special Report 176. Transportation Research Board, National Academy of Sciences, Washington D.C., 234 p.

Selby, M.J. (1976). Selected Annotated Bibliography of Mass Wasting in New Zealand to 1975. Occasional Report No.1, Department of Earth Sciences, University of Waikato, New Zealand, 38 p.

Selby, M.J. (1982). Hillslope Materials and Processes. Oxford University Press, Oxford, 264p.

Sowers, G.B. and Sowers, G.F. (1970). Introductory Soil Mechanics and Foundations. (3rd Ed). Macmillan, New York, 556 p.

Standards Association of New Zealand (1978). Code of Practice for Earth Fill for Residential Developments (NZS4431:1978). Standards Association of New Zealand.

Taylor, D.K., Hawley, J.G., Riddolls, B.W. (1977). Slope Stability in Urban Development (DSIR Information Series No.122), DSIR, Wellington, 71p.

Terzaghi, K and Peck, R.B. (1967). Soil Mechanics in Engineering Practice (Second Edition). John Wiley & Sons, New York, 729p.

Transportation Research Board, U.S.A. (1978). Landslides : Analysis and Control. National Academy of Sciences, Washington D.C., 234 p.

The New Zealand Geomechanics Society (1980). Stability of House Sites and Foundations, Advice to Prospective House and Section Owners. Published for the Earthquake and War Damage Commission by the New Zealand Geomechanics Society, Institution of Professional Engineers New Zealand, Wellington, 19 p.

The Institution of Professional Engineers New Zealand (1981). Proceedings of the Symposium on 'Geomechanics in Urban Planning' held jointly by the New Zealand Geomechanics Society and New Zealand Planning Institute. The Institution of Professional Engineers New Zealand, Wellington.

The New Zealand Institution of Engineers (1974). Proceedings of the Symposium on Stability of Slopes in Natural Ground, held by the New Zealand Geomechanics Society. The New Zealand Institution of Engineers. Wellington.

Varnes, D.J. (1978). Slope Movement and Types and Processes. In Landslides: Analysis and Control, Special Report 176. Transportation Research Board, National Academy of Sciences, Washington, D.C., 234p.

Varnes, D.J. (1984). Landslide Hazard Zonation: a Review of Principles and Practice. Natural Hazards 3, UNESCO, Paris, 63p.

A series of four audio-tapes on the subject of geotechnical engineering is available from the Centre for Continuing Education, University of Auckland. These tapes, sponsored by the Institution of Professional Engineers New Zealand, were produced by an experienced consulting geotechnical engineer, Dave Hollands. His discussion of the topics of soil strength, groundwater effects and slope stability, on the second tape in the series, is particularly relevant to the subject of this report. In addition, some important basic aspects regarding the choice and design of retaining walls are also covered.

5.3 References for Description of Soils and Rocks

A manual of soil and rock description for engineering use is under preparation by the New Zealand Geomechanics Society. A draft of the 'method' is available to all members of the Society. It is hoped that, when finally published, the methods advocated will be used throughout the engineering profession in New Zealand and thereby achieve uniformity in the description and classification of soils and rocks.

The methods proposed are based on various internationally recognised schemes, including the following :

British Standards Institution (1981). Code of Practice for Site Investigation, (BS 5930:1981). British Standards Institution, London, 147p.

Geological Society of London (1977). The description of rock masses for engineering purposes: Geological Society Engineering Group Working Party Report. Quarterly Journal of Engineering Geology, Vol. 10, pp 355-389.

International Association of Engineering Geology (1981). Rock and soil description and classification for engineering geological mapping. Report by the IAEG Commission on Engineering Geological Mapping. Bulletin of the International Association of Engineering Geology, No. 24, pp 186-226.

International Society for Rock Mechanics (1978). Suggested methods for the quantitative description of discontinuities in rock masses. International Journal of Rock Mechanics and Mining Sciences, and Geomechanics Abstracts, Vol 15, pp 319-368.

International Society for Rock Mechanics (1981). Basic geotechnical description of rock masses. International Journal of Rock Mechanics and Mining Sciences, and Geomechanics Abstracts, Vol. 18, pp 85-100.

TOPOGRAPHY

T.1 Landform

Briefly explain the geomorphological setting of the subject slope.

T.2 Slope profile

Describe the profile of the slope including observations regarding - slope height, slope gradient, slope shape (convex/concave), irregularities in slope profile, etc. Observations should extend beyond the property boundaries, where relevant to the assessment of the subject slope.

T.3 Retaining/support structures

Record locations and type of any retaining walls or other supporting structures which would influence the stability of the subject slope. Observations should be made with respect to construction materials and condition of the structure.

GEOLOGY

G.1 Geology - subject slope

Are exposures of the materials forming the subject slope available for inspection? If not this fact should be stated. If they are, the location of the exposure(s) should be described and also, shown on an accompanying sketch/site plan. The exposed materials should be described according to a suitable classification/description scheme; the scheme used should be stated. In addition, the distribution/extent of different materials within the exposure should be described and, where necessary for the sake of clarity, illustrated by an accompanying sketch.

G.2 Geology - local

Are exposures of materials, which are expected to be of a similar type of those forming the subject slope, available in the vicinity of the subject slope. If so, record all relevant observations as required for G.1.

HYDROLOGY

H.1 Vegetation

Record distribution and extent of vegetation both on the site and, where relevant, beyond site boundaries. Record vegetation type (e.g. grass, shrubs and/or trees) and vegetation cover (bare, sparse cover, moderate cover or dense cover). A particular note should be made regarding the presence of vegetation species chararacteristic of wet ground as well as those with a high water abstraction capability e.g. eucalyptus cinerea. Also, any evidence of past modifications to vegetation cover should be recorded.

H.2 Artificial surfacing

Record the location of any areas where the natural ground surface has an artificial covering e.g. buildings, paved surfaces etc. Also record adequacy of drainage of surface runoff from these areas and the effectiveness of surface protection e.g. any holes or cracks.

H.3 Services

Record position, approximate size and condition of pipes, water tanks, septic tanks, reservoirs etc, either on or above the subject slope. A careful scrutiny should be made for any evidence of leakage. Where underground services are inferred, evidence should be stated.

H.4 Natural surface drainage

Record position of any natural drainage courses, either on site or, where considered relevant, beyond site boundaries. Observations should be made with respect to water flow and level. Special attention should be paid to any evidence of bed or bank erosion. Observations regarding subsurface water courses may also be made in this section.

H.5 Subsurface erosion

Record any evidence of subsurface erosion such as piping or solution cavities (particularly prevalent in loess soils, pumice soils and limestone).

H.6 Seepage

Record position of springs and other points of seepage. Where active, record flow (substantial, slight) and where inactive state evidence e.g. staining.

H.7 Miscellaneous

Record location of any ponds, areas of waterlogging etc.

H.8 Weather

Record weather conditions at time of inspection and for preceding 1 month.

INSTABILITY

I.1 Ground profile

Record any local irregularities in the ground profile e.g. hummocky surface, local oversteepening, unnatural depressions etc. The location of any such irregularities should be described and highlighted on an accompanying sketch/site plan.

I.2 Ground cracks

Record the location, aperture and persistence of any cracks in the ground surface.

I.3 Structural distress

Record the location and nature of any cracks in existing buildings, walls, pathways or other paved surfaces. Other forms of distress may include bulging of retaining walls, subsidence of paved surfaces, tilting of originally vertical power poles etc.

I.4 Miscellaneous

Other possible direct or indirect evidence of past or present instability to be recorded including curvature or leaning of tree trunks, anomalous water seepage, areas of strongly contrasting vegetation, etc.

Table 1 Check List of Field Observations for Slope Stability Assessments.

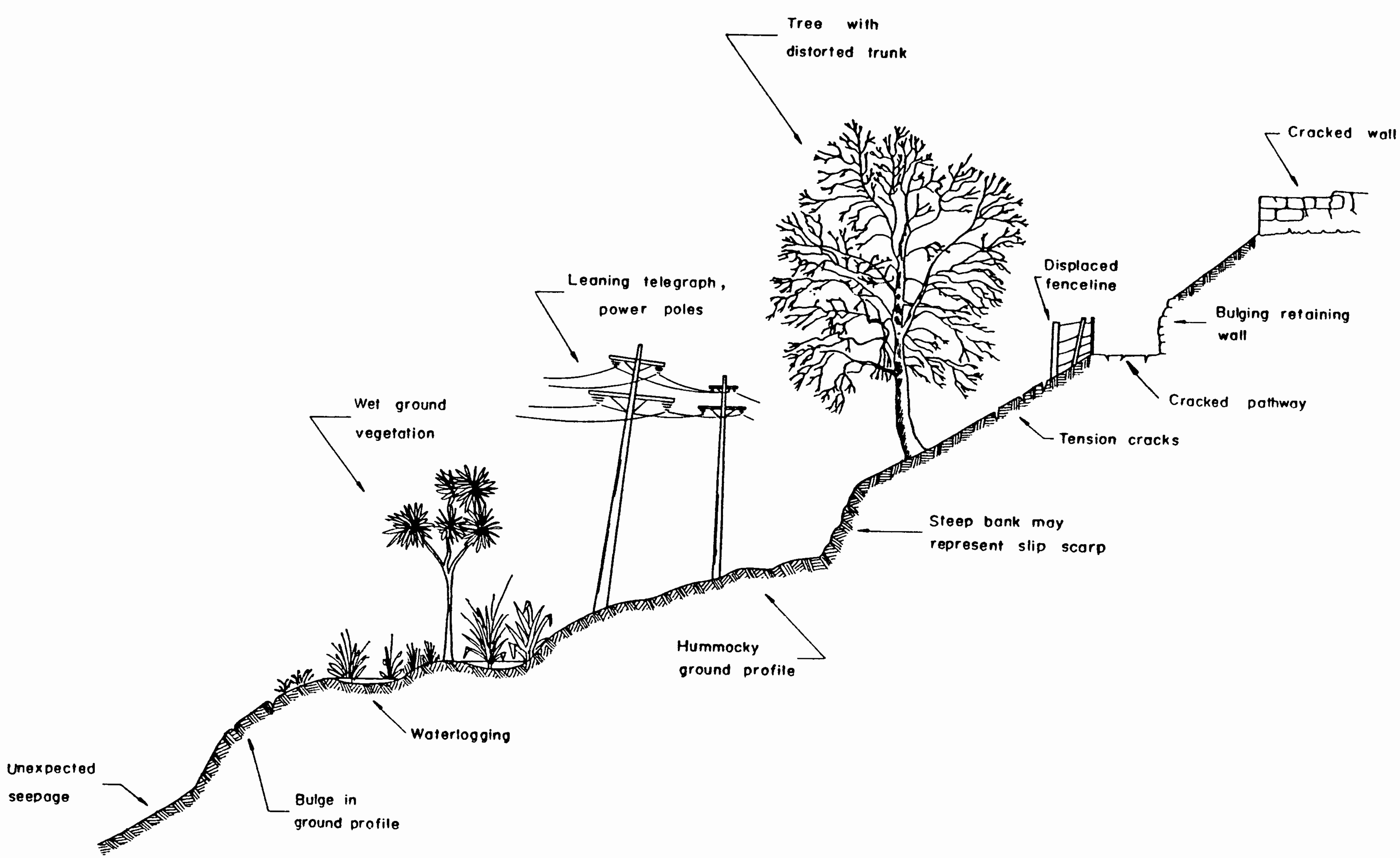

FIGURE 1 : Surface indications of possible slope instability

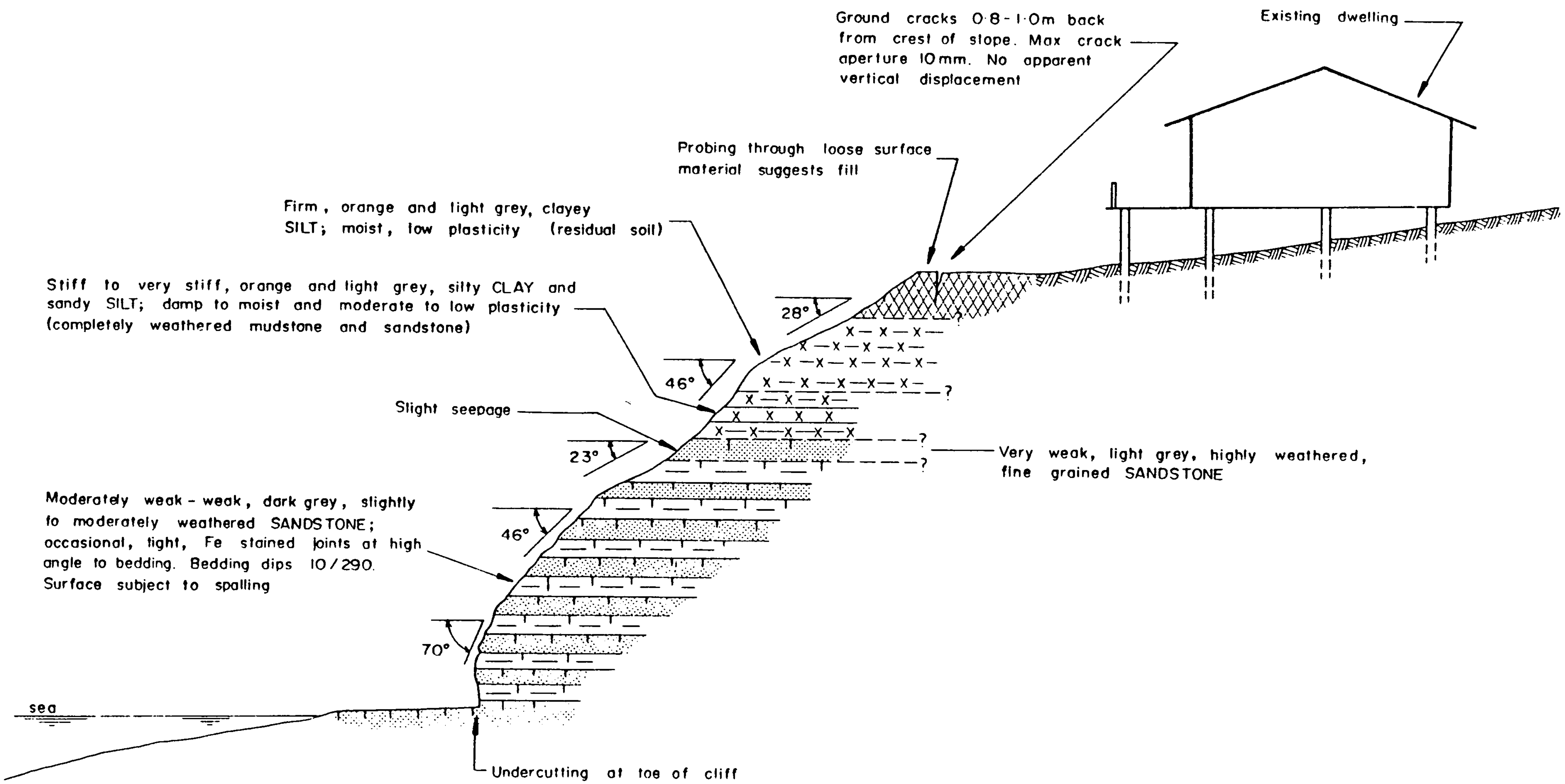

FIGURE 2 : Example of engineering geology profile based on surface observations

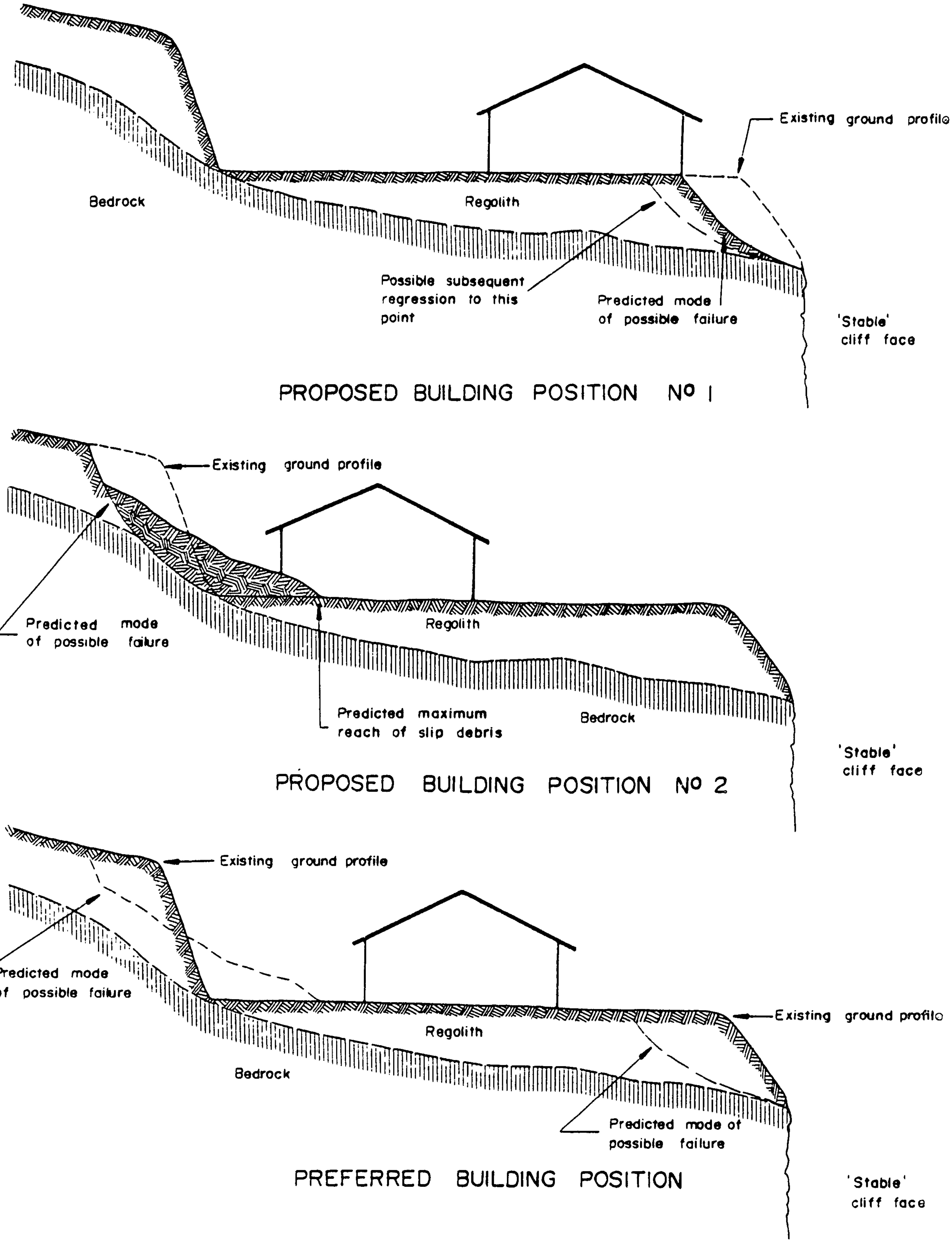

FIGURE 3 : Avoidance solution to potential instability hazards

APPENDIX A

Assessment Costs

APPENDIX A ASSESSMENT COSTS

A.1 General

This section gives an indication of typical costs for different levels of assessment. The figures given reflect costs charged by consultants in 1986. For this type of work, charges are normally on a time basis, plus disbursements at cost, in accordance with the Association of Consulting Engineers New Zealand document 'Conditions of Engagement for Consulting Engineers, Document C, September 1985'. A copy of this document is given on Pages 56 to 58.

A.2 Desk Study Only

The investigator may only be required to examine existing information regarding the stability of a building site when, for example, a client requires a second opinion on a previous assessment. The work would involve scrutiny of the previous report(s), examination of any other existing documented information covering the site and interpretation of aerial photographs. The comments would probably be in the form of a short letter report. The cost for a study of this type could be of the order of \$200 - \$400 depending on the volume of existing data to be reviewed.

A.3 Desk Study Plus Field Inspection

In the review situation mentioned above, the investigator has to rely on observations made by others and does not have the opportunity to gain first hand knowledge of the site. It may be however that conditions on site have changed significantly since the previous report was compiled or that certain features were previously overlooked. Hence, it is considered desirable that the investigator inspect the site himself. In addition to the review situation, the 'desk study plus field inspection' level of assessment may be appropriate for giving advice to a prospective purchaser. The cost of this level of assessment, which, in addition to comments derived from existing documented information and aerial photographs, would also include observations made during the field inspection, is likely to be up to \$500.

A.4 Desk Study and Field Inspection Plus Sub-surface Investigation

The majority of slope stability assessments require a more positive indication of ground conditions than can be gained from a superficial inspection of the site and geological judgement. The cost of the different subsurface investigation techniques varies considerably. Typical costs using various techniques are given below.

A.4.1 Hand Augering and Investigation Pits

In a single day, it should normally be possible to drill 4 to 5 hand augerholes to depths of 3 to 4 metres (the remainder of the day might be spent travelling to and from site - not included in the cost given below). The drilling would be supplemented by appropriate in situ tests and samples would be taken if necessary. The cost of the hand augering, together with a desk study and field inspection, is likely to be up to about $1,200. The assessment report would include detailed logs of the boreholes.

For a similar cost, it should be possible to excavate at least 5 investigation pits to depths of at least 4 metres. In view of the equivalent cost and the technical advantages of investigation pits as outlined in Section 3.4.3, this is considered the preferred technique for shallow subsurface investigations.

A.4.2 Machine Drilling

Machine drilling, which is most commonly used for deeper subsurface investigation, is substantially more expensive than the techniques discussed above. The cost of drilling two boreholes to a depth of 10 metres, together with a detailed field inspection and desk study, is likely to be in the order of $3,000.

A.4.3 Desk Study, Field Inspection, Subsurface Investigation and Stability Analysis

The additional cost of a stability analysis will depend on the complexity of the problem and whether the shear strength parameters are determined by laboratory testing. The cost of carrying out an analysis for relatively straightforward ground conditions using assumed parameters may be only $200 to $300. However, for more complex conditions where several trial surfaces need to be analysed and where two sets of triaxial tests are carried out to define strength parameters, the cost may rise to up to about $2,000.

Hence, for a stability assessment which involves a desk study, field inspection, a subsurface investigation comprising 4-5 hand augerholes and a straightforward stability analysis using assumed effective stress strength parameters (or alternatively a total stress analysis using in situ shear vane results) the total cost could be up to $1,500. If the above work were supplemented by an additional two machine boreholes, two sets of triaxial sets and additional analysis the total cost could be up to $5,000.

ASSOCIATION OF CONSULTING ENGINEERS NEW ZEALAND

A - 17

SEPTEMBER 1985

PRACTICE NOTE

CONDITIONS OF ENGAGEMENT FOR CONSULTING ENGINEERS

DOCUMENT C (1985) - FOR ROUTINE WORK ON A TIME BASIS

This Document details the standard conditions for a contract between the Consulting Engineer and the Client, for routine services normally provided on a time and expense basis. Other documents are available for more complex engagements. It is recommended that this Document be attached to a letter or standard form, defining the scope of work and any special arrangements, and be confirmed in writing by the Client.

SERVICES PROVIDED

1. The Consulting Engineer shall perform his services in accordance with the reasonable standard of skill, care and diligence generally exercised by the profession in New Zealand subject to any financial, physical, time or other restraints imposed by the Client or necessarily resulting from the nature of the engagement.

2. The Consulting Engineer may be required to inspect works being constructed. Unless otherwise agreed in writing, this service shall be limited to periodic site visits to assist in interpreting the design and to observe whether the works for which the Consulting Engineer is the professional adviser are being carried out in general accordance with the contract documents. Any such observation shall not transfer to the Consulting Engineer any of the responsibilities of a contractor nor shall it in any way limit the responsibilities of a contractor to carry out the works in accordance with his contract.

RELATIONSHIP WITH CLIENT

3. The terms of these conditions of engagement shall be binding on the party for whose ultimate benefit the services are to be performed, (referred to herein as the "Principal") whether or not the Principal is the party by whom the Consulting Engineer is engaged. Where the Consulting Engineer is appointed by an adviser to the Principal or by some other representative acting on behalf of the Principal, then:

   (a) Where the adviser or other representative is acting or purports to act as agent for the Principal, then the Principal shall be the Client and the adviser or other representative acknowledges that he is the duly authorised agent of the Client and accepts these conditions on behalf of the Client.

   (b) Where the adviser or other representative does not act as agent for the Principal, then the adviser or other representative undertakes that he will contract with the Principal, for the benefit of the parties referred to in clause 14, to the effect that the terms of clauses 11 to 14 of these conditions shall apply to any claim by the Principal as if the Principal were the Client, and the adviser or other representative shall be liable to the parties referred to in clause 14 for any failure to obtain the benefit of such a contract.

4. Whether or not the adviser or other representative by whom the Consulting Engineer is appointed is acting as agent for the Principal, the party by whom the Consulting Engineer is appointed shall be responsible for payment of all fees and job costs and undertakes to indemnify the Consulting Engineer for any unpaid fees or job costs.

FEES AND JOB COSTS

5. The Consulting Engineer shall be entitled to monthly progress payments of fees and other job costs.

6. The fees shall be charged on a time basis.

7. The Client shall pay the Consulting Engineer for all other job-related costs, including disbursements and telecommunication, reproduction, testing and travelling expenses. The Consulting Engineer may add a service charge of 10 percent of invoice costs where payment to others has been made on behalf of the Client.

8. Accounts for engineering services shall be due on the 20th of the month following presentation. Where payment is not made within 30 days of due date, the Consulting Engineer shall be entitled to recover interest from due date at the rate of 2.0 percent per month.

COPYRIGHT AND USE OF DOCUMENTS

9. Copyright in all documents, and in the works executed from them, will remain the property of the Consulting Engineer. The Client shall be licensed, on payment of all fees and other job costs due to the Consulting Engineer, to use the documents only for the specific purpose for which they were prepared.

10. The Client shall not enter into any contract with nor make any representations to a third party or third parties which describe the Consulting Engineer's duties and responsibilities in a manner inconsistent with the terms of this agreement.

LIMITATION OF LIABILITY

11. The Consulting Engineer shall not be liable for the commercial performance of the project, or for any loss or damage arising by reason of any delay in completion of the project, or for any loss of profits, or for any indirect or consequential loss of whatever nature.

12. If the Consulting Engineer or any subconsultant shall be found liable to the Client (whether under the express or implied terms of this agreement and whether in negligence or otherwise in common law) for any costs, loss or damage suffered by the Client, however caused and of whatever nature, arising out of or connected with the performance or failure of performance of services by the Consulting Engineer or any subconsultant, then the maximum amount of that liability in total for the aggregate of all such claims shall be $200,000.

13. The liability of the Consulting Engineer or a subconsultant to the Client against loss or damage as aforesaid shall be reduced proportionately to the extent that any acts or ommissions of the Client contributed towards any such loss or damage.

14. For the purposes of clauses 11, 12 and 13 of these conditions:

    (a) the expression "Consulting Engineer" shall include all employees of the Consulting Engineer; and

    (b) the expression "subconsultant" shall include all parties engaged by the Consulting Engineer or by any other subconsultant to perform any part of the services provided for by this engagement, and all employees of the subconsultant.

    The terms of clauses 11, 12 and 13 shall be construed as conferring a benefit on, and being enforceable at the suit of, every such party, whether party to this contract or not.

POSTPONEMENT OF SERVICES AND TERMINATION OF ENGAGEMENT

15. Any agreement between the Consulting Engineer and the Client may be postponed or terminated by either party, on the expiration of reasonable notice given in writing.

16. Upon receipt of such notice from the Client, the Consulting Engineer shall take immediate steps to bring the services to a close and to reduce expenditure to a minimum.

17. Upon postponement of the services or termination of the engagement, the Consulting Engineer shall be entitled to payment of fees and other job costs up to the effective date of postponement or termination and such further fees and costs incidental to the orderly termination of the services.

SETTLMENT OF DISPUTES

18. In the event of any dispute arising between the Consulting Engineer and the Client, the matter in dispute shall be referred to the final decision of a sole arbitrator to be appointed by the parties. If the parties fail to agree, within one month of one party giving notice in writing to the other party of a dispute to be referred to arbitration, then either party may request the President of the Institution of Professional Engineers, New Zealand, to appoint an arbitrator and the arbitrator shall be so appointed.

APPENDIX B

Types and Causes of Slope Instability

APPENDIX B

LIST OF FIGURES

APPENDIX B TYPES AND CAUSES OF SLOPE INSTABILITY

B.1 Types of Slope Instability

B.1.1 General

It is beyond the scope of this report to discuss in detail the great variety of slope movement types that occur. Instead, the major types are outlined. For the reader who requires further elaboration, attention is drawn to the section by D. J. Varnes in the textbook on landslides entitled, 'Landslides : Analysis and Control' (Transportation Research Board, U.S.A., 1978). The subject of slope movements is also discussed by Selby (1982) with frequent reference to New Zealand conditions. Also, Selby (1976) is a useful source document of New Zealand case histories of slope instability.

The classification scheme developed by Varnes categorises slope movements into various types based on the types of movement and the materials involved. The primary types of movement identified include:-

a) falls,
b) topples,
c) rotational slides,
d) translational slides,
e) lateral spreads,
f) flows, and
g) complex movements.

There are no documented cases of lateral spreads in New Zealand and therefore this type of movement is not discussed here. A brief outline of the other types and the geological conditions with which they are commonly associated is given below.

B.1.2 Falls

Falls involve the detachment of a mass of soil or rock of whatever size from a steep slope or cliff face along a surface on which little or no shear displacement takes place. The detached mass descends through the air by free fall, leaping, bounding or rolling.

Falls are generally associated with very steep, vertical or overhanging rock cliffs in which steeply dipping discontinuities act as release surfaces. Rock falls may be triggered by removal of support below (by undercutting caused by erosion for example), by hydrostatic pressure in a water filled fracture behind the unstable block or by vibration caused by an earthquake or by blasting. A typical rock fall situation is illustrated in Figure B1, Page 67.

Falls are not restricted to hard rock; falls in soil and soft rock may occur on steep or vertical banks due to undercutting by erosion, stress relief, etc. Slab falls in steep banks formed of weak siltstones and sandstones are a common phenomenon in the North Island of New Zealand resulting primarily from stress relief and the development of vertical cracks parallel to the slope face.

B.1.3 Topples

Toppling involves the forward rotation of an unstable mass about a lower pivot point under the action of gravity. In the sense that little or no shear displacement takes place along the failure surface(s), toppling is similar to falling. Toppling may also be triggered by hydrostatic pressure in a fracture behind the unstable mass or by vibration caused by an earthquake or by blasting.

Toppling is most commonly associated with steep rock faces in which discontinuities dip very steeply into the slope. It may also occur in steep banks formed of clayey soils in which deep vertical cracks form parallel to the slope face. The subject of toppling and the conditions under which this form of instability may occur is discussed by Hoek & Bray (1977). A toppling situation is illustrated in Figure B1, Page 67.

B.1.4 Slides

Sliding involves the movement of an unstable mass along a single failure surface or a number of surfaces. Shear displacement takes place along most of the failure surface(s) or zone although, at the head of the slide, the ground may be in tension. Slides may occur in either rock or soil. They also vary considerably in size; in a cliff-top situation the sliding of the soil mantle over underlying rock may involve only a few cubic metres of slide debris. At the other end of the scale are such massive slides as the one at Tahunanui in Nelson which covers about 26 hectares (refer Johnston [1979]). Slides are categorised as either rotational or translational depending on the manner in which the unstable mass moves in relation to the surrounding and underlying stable ground.

B.1.4.1 Rotational slides

In this form of sliding the failed mass rotates such that, at the rear, it drops in relation to the intact stable ground above and, at the toe, the failed mass rises and, commonly, over-rides the intact ground below.

Rotational failures are normally restricted to soils although they may also occur in intensely fractured weak rock in which there is no clearly defined discontinuity pattern which would otherwise control the shape of the failure surface. Rotational failures may also take place in completely or residual weathered rock in which the strength of the rock is so reduced that shearing may take place through the body of the rock rather than being controlled by the discontinuity pattern (which, in the case of residual weathered rock, will have been destroyed). In essence therefore, the ability of a slope to fail in a rotational manner is governed by the absence or presence of structural defects. Moreover, even where rotational failure takes place, the shape of the failure surface is rarely perfectly circular as the name tends to imply. A truly circular failure surface is only likely to develop in a truly homogeneous material. The shape of the failure surface may be influenced by even quite minor local variations in the slope forming materials as well as any seemingly insignificant structure defects. Figures B2, Page 68 shows the typical form of a rotational slide.

B.1.4.2 Translational Slides

This form of instability involves movement of a failed mass out, or down and out, of the slope along a planar or gently undulating failure surface. Slides of this type may occur in either soil or rock. Whatever the state of induration of the slope forming material however, a prerequisite for translational movement is the presence of some plane or zone of weakness. However, such a defect need not necessarily dip out of the slope along the full length of the failure surface.
Shallow translational slides are a common form of slope instability in New Zealand, typically involving movement of the weaker soil mantle over underlying stronger materials. Movement is commonly attributed to perching of water above the interface.

The massive landslide which occurred at East Abbotsford in August 1979 was a translational slide in which failure occurred along a very thin layer of clay within the Abbotsford Formation about one metre below the interface with the overlying Green Island Sand. A typical example of a shallow translational slide is shown in Figure B2, Page 68.

Translational slides are common on rock slopes and, depending on the discontinuity pattern, may occur as planar or wedge type failures. These forms of instability are discussed in detail by Hoek & Bray (1977). A typical wedge failure, developed as a result of two intersecting sets of joints is illustrated in Figure B1, Page 67.

B.1.5 Flows

In flow failures, in addition to the primary shear movements at the interface between the unstable body and surrounding and underlying intact ground, movements are also distributed throughout the failed mass. Flows occur primarily in soils, and particularly those that are poorly consolidated, but may also occur in intensely jointed rock masses. Flows are commonly very rapid movements involving highly mobile materials which may travel long distances. The liquefaction of uncompacted fill material results in flow type movements.

Soil creep is a slow form of flow movement. This phenomenon, which occurs on steep slopes throughout New Zealand, involves imperceptible movement of the near surface soils as they 'flow' downslope. Although the primary cause of creep is gravity, the process is assisted by many other agents. Figure B3, Page 69, illustrates some of the common causes of soil creep. It should be noted that the effects of soil creep on a building may be no less disastrous than those resulting from the more catastrophic types of movement discussed previously.

B.1.6 Complex Movements

Complex movements here means a combination of two or more types of movements as discussed above. It is quite common for instance for saturation of the slide debris from a slide type movement to result in a subsequent flow.

B.2 Factors Affecting Slope Stability

B.2.1 General

In order to assess the likelihood of a slope failure, the investigator must weigh up the relative importance of the factors which influence the stability of the slope. A multitude of factors are involved, some of which are detrimental to stability and some of which are beneficial. It is a misconception to attribute slope movement to a single factor although there may be one, perhaps quite trivial, factor which triggered movement. However, this final event cannot be regarded as the cause; in the words of Sowers and Sowers (1970)... "often the final factor is nothing more than a trigger that set in motion an earth mass that was already on the verge of failure. Calling the final factor the cause is like calling the match that lit the fuse that detonated the dynamite that destroyed the building the cause of the disaster".

The factors involved can be expressed in terms of the forces resisting movement and those tending to cause movement. Slope failure occurs when the total disturbing forces, i.e. those components of forces acting in the direction of potential movement, exceed the total resisting forces, i.e. those forces which exist due to the strength of the slope forming materials as well as the buttressing forces. The various resisting and disturbing forces and the ways in which these forces may be modified to the detriment of stability are outlined in the following sections. Reference should also be made to Figure B4, Page 70, which illustrates many of the factors detrimental to stability.

B.2.2 Disturbing Forces

The disturbing forces include the following:

(a) The weight of the slope forming materials and the water in them,
(b) Hydrostatic forces,
(c) Seepage drag forces created by percolating water, and
(d) Lateral pressure caused by water or ice in cracks, and by swelling of hydrating clays.
(e) In addition, an increase in the total disturbing force will result from surcharge of the slope by human agencies, e.g. filling, building, waste or stockpiles and the weight of water due to leakage from services. These additional disturbing forces may be permanent or temporary.
(f) The effect of transient disturbing forces caused by earthquakes may also need to be taken into account in certain parts of New Zealand and particularly in situations where the consequence of slope failure is important. Vibrations from blasting, heavy vibrating machinery and even a nearby slope failure also produce transient disturbing forces.

B.2.3 Resisting Forces

The total force resisting movement is the sum of the forces due to the strength of the slope forming materials as well as buttressing forces. The former are expressed in terms of the shear strength of the material available to resist movement over the total area of the failure surface.

Resisting forces may be reduced by a decrease in the strength of the ground or a reduction in buttressing forces.

B.2.3.1 Strength reduction

Strength reduction of the ground may be caused by one or more of the following:

a) An increase in pore water pressure. This may result from a rise in the groundwater table or the development of a zone of perched water. An increase in pore pressure results in an effective reduction in interparticle friction and hence a reduction in shear strength.

b) An increase in saturation in the unsaturated zone above the water table reduces soil suction i.e. the interparticle pressure due to capillary tension. This also results in a reduction in effective shear strength.

c) Percolating water may remove natural cementing materials bonding adjacent soil particles leading to a reduction in the cohesion component of shear strength.

d) An increase in moisture content leads to the hydration and softening of clay minerals as a function of absorption, swelling and reduction in cohesion. This is particularly significant in clayey soils containing a high proportion of such clay minerals as montmorillonite and/or halloysite.

e) Groundwater flow may lead to erosion along preferred drainage paths within the slope, i.e. cavitation and tunnel erosion.

f) Removal of vegetation and subsequent decay of root systems may directly or indirectly reduce resisting forces in a number of ways.

   Vegetation cover on a slope is clearly beneficial in terms of erosion protection and landscape quality. Also, it generally has a stabilising effect on slopes as a result of the mechanical reinforcement of the soil mass by the root system and the depletion of soil moisture by evapotranspiration and rainfall interception. Many cases have been cited of slope failures attributed to vegetation clearance. Under certain circumstances however, vegetation may be detrimental to stability, e.g. the increase in shear stresses on steep slopes due to tree surcharge and the overthrowing of trees by high winds in cliff-top situations.

g) Vibrations from heavy machinery, blasting activities, etc., may reduce the strength of slope forming materials; in particular, saturated fine sands and silts may liquefy if subjected to intense vibration. Ground vibrations resulting from a seismic shock also increase disturbing forces.

B.2.3.2 Reduction of buttressing forces

A reduction of buttressing forces results from removal of support to the slope. This is normally caused by undercutting of the slope either by human activities or as a result of stream/river erosion or wave action.

B.3 References

Hoek, E. and Bray, J.W. (1977). Rock Slope Engineering. (Revised 2nd Ed.). The Institution of Mining and Metallurgy, London, 402 p.

Johnston, M.R. (1979). Geology of the Nelson Area. New Zealand Geological Survey, DSIR, Wellington, 52 p.

Selby, M.J. (1976). Selected Annotated Bibliography of Mass Wasting in New Zealand to 1975. Occasional Report No.1, Department of Earth Sciences, University of Waikato, New Zealand.

Selby, M.J. (1982). Hillslope Materials and Processes. Oxford University Press, Oxford, 264 p.

Sowers, G.B. and Sowers, G.F. (1970). Introductory Soil Mechanics and Foundations. (3rd Ed.). Macmillan, New York, 556 p.

Transportation Research Board, U.S.A. (1978). Landslides : Analysis and Control. Special Report 176, National Academy of Sciences, Washington D.C., 234 p.

Varnes, D.J. (1978). Slope Movement and Types and Processes. In Landslides : Analysis and Control. Special Report 176, Transportation Research Board, National Academy of Sciences, Washington D.C., 234 p.

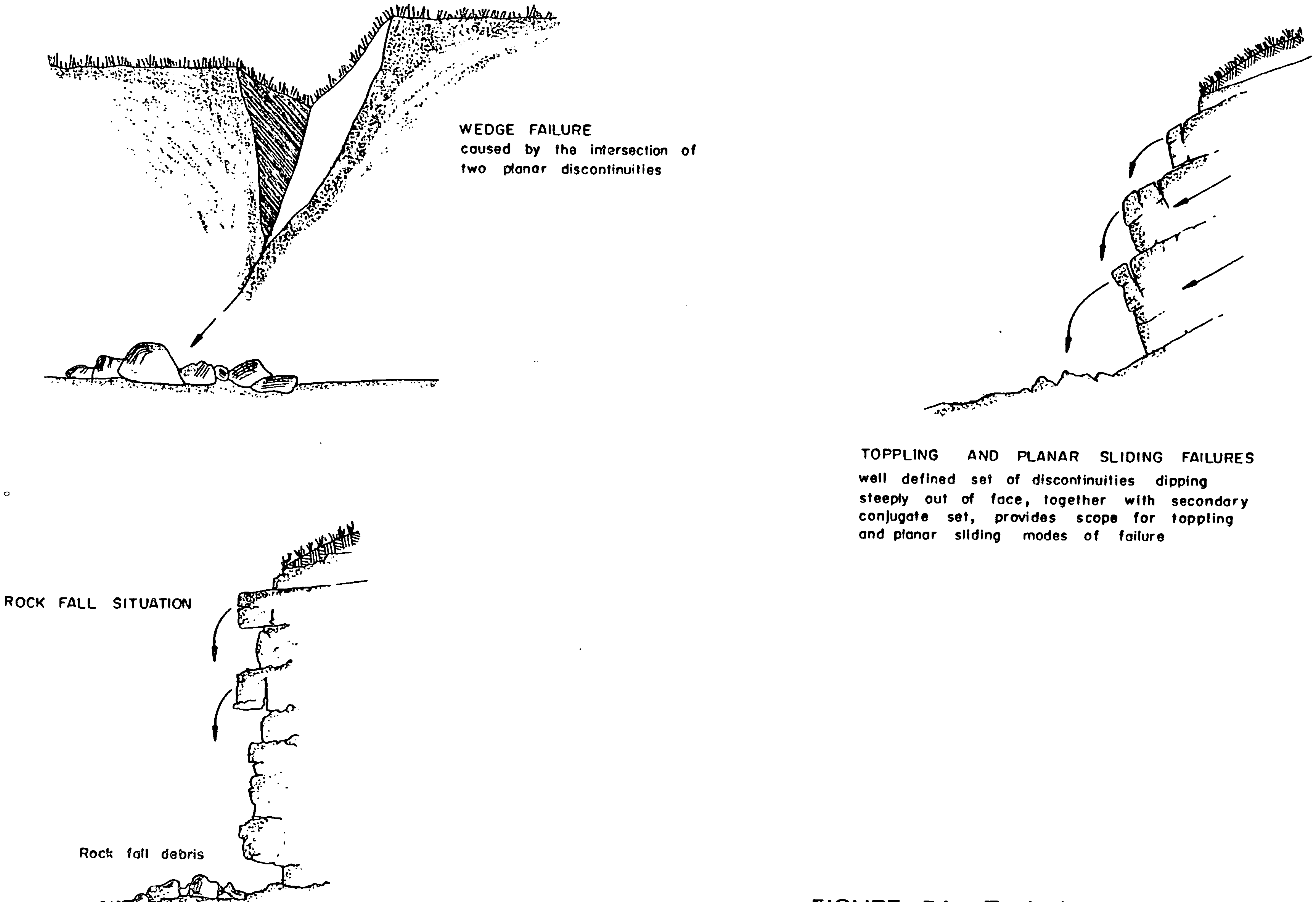

FIGURE B1 : Typical rock slope failure modes

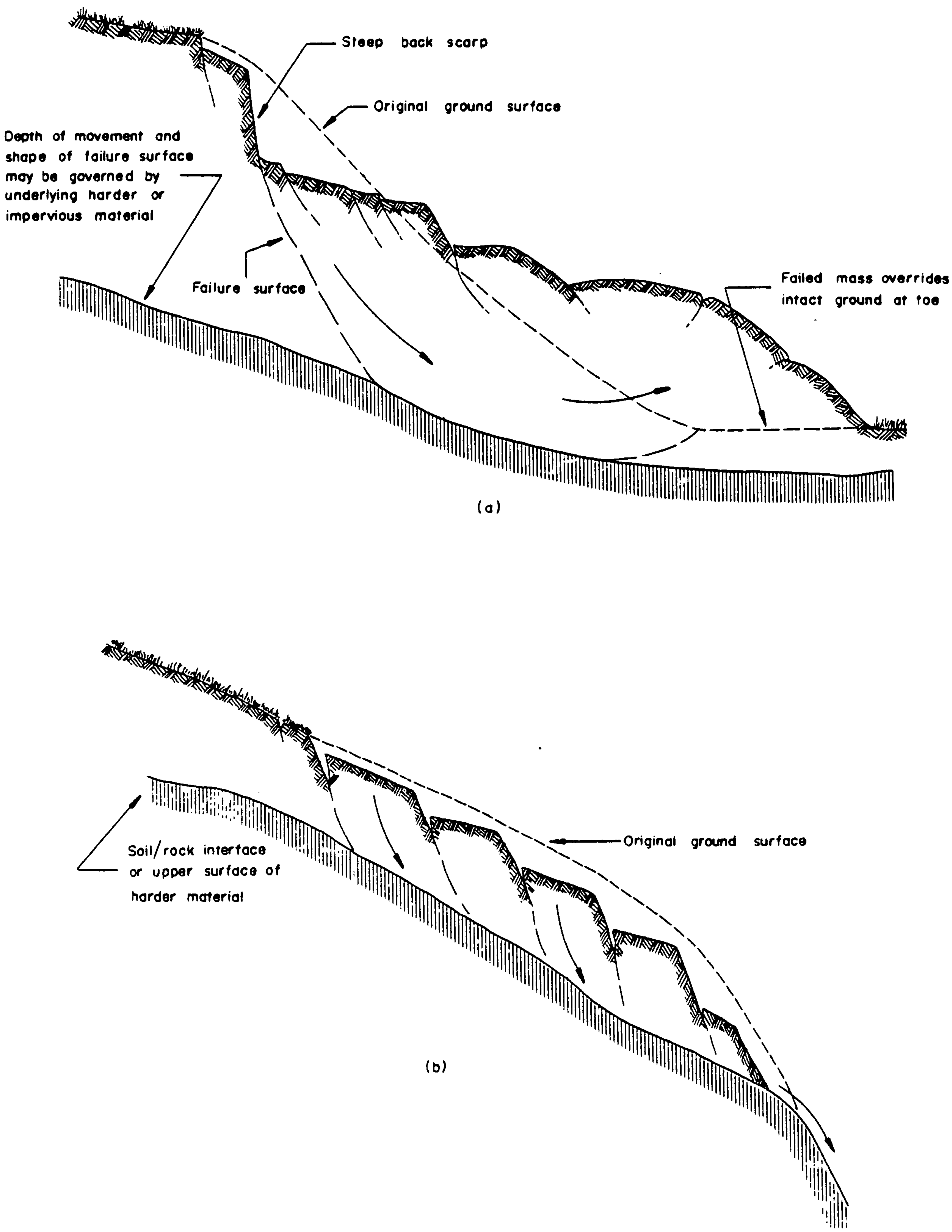

FIGURE B2: Examples of (a) rotational and (b) translational slide failures

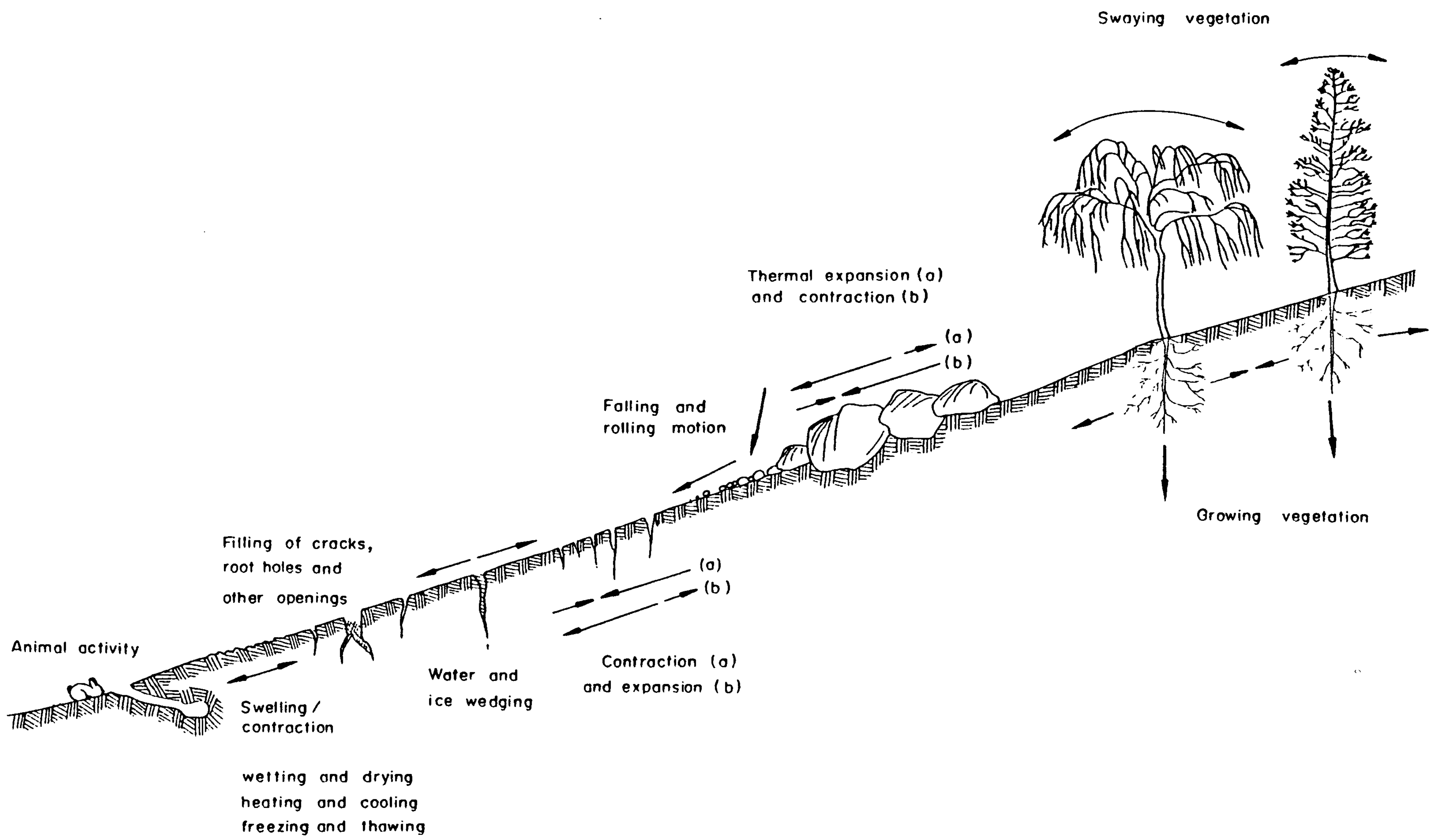

FIGURE B3 : Common causes of soil creep

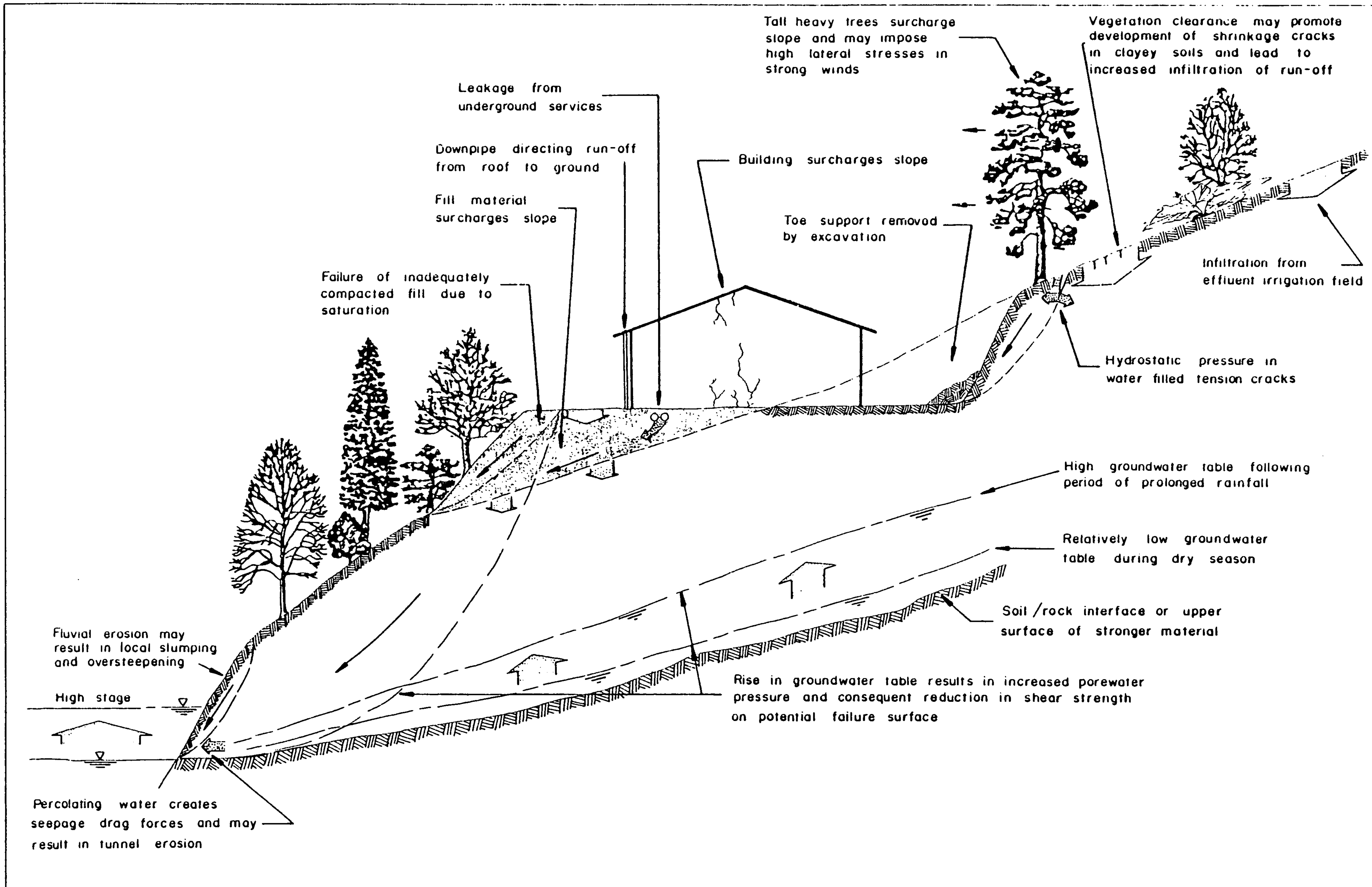

FIGURE B4 : Factors detrimental to stability

APPENDIX C

Remedial/Preventive Measures

APPENDIX C REMEDIAL/PREVENTIVE MEASURES

C.1 General

This appendix deals with those sites where there is an unacceptable risk of instability and where, in the event of a slope failure, the proposed building development would be affected in some way. In such circumstances the only options available would be to either abandon the proposed building development or to implement remedial or preventive measures. The various types of remedial and preventive measures that are commonly employed to either remove the risk or reduce it to an acceptable level are briefly outlined below.

The choice and design of suitable remedial/preventive measures requires a good understanding of the existing or potential failure mechanism. It may be necessary to carry out additional subsurface investigation for design purposes. A suitable method of stability analysis should be carried out to quantify the improvement that may be achieved by implementing the proposed measures. It is also important that construction of the remedial/preventive works be supervised; where such works involve excavation, these should be inspected to ensure that the ground conditions encountered conform with those assumed in the design. For sites on which ground movement is occurring, movement monitoring should be carried out to check the effectiveness of the measures. Monitoring should be extended through at least one wet season following completion of the works and, only after the effectiveness of the measures has been confirmed, should building be allowed to proceed.

C.2 Drainage

C.2.1 General

In view of the significant influence of water on slope stability, drainage is probably the most common, and may also be the most effective, method of improving stability. It may also be substantially cheaper than other methods. The successful use of drainage for remedying two landslips in the Auckland area is reported by East (1974).

Drainage may serve two purposes. It may be aimed at preventing additional water entering the problem area, either in the form of surface runoff or as groundwater. Alternatively, drainage may be required to either lower the groundwater table or prevent it from rising within the problem area.

Whatever form of drainage system is installed, it is essential that groundwater monitoring is carried out to confirm that the desired effect is being achieved. Monitoring should be extended through at least one wet season following installation of the drainage system.

C.2.2 Interceptor Ditches

Surface runoff may be intercepted by open or lined ditches. If an interceptor ditch is unlined it is important that an adequate fall is provided and also that it is well maintained; otherwise, the ditch may promote rainfall infiltration.

C.2.3 Cut-Off Drains

A surface ditch may be deepened to form a cut-off drain to intercept groundwater. The depth to which a cut-off drain can be constructed will depend on the capabilities of the excavation plant available. The drain is normally backfilled with suitably graded granular material which has adequate permeability but which does not allow fines to enter and clog the drain. Where granular material of the correct grading is not readily available, filter cloth must be used to line the base and walls of the drain trench. The use of filter cloth is recommended anyway as an additional means of ensuring the long-term effectiveness of the drain. Water is usually carried away in a perforated pipe installed at the base of the trench. It is important that the base of the trench has a positive fall. The drain <u>must</u> be sealed at the surface to prevent ingress of surface runoff; the drain may otherwise act as a 'soakage pit'. Under certain circumstances, deep cut-off drains may be detrimental to stability by effectively reducing the length of a potential failure surface along which shear strength can be mobilised to resist movement. This should be taken into account when evaluating the expected improvement in stability.

C.2.4 Horizontal Drains

Horizontal drains consist of slotted or perforated pipes (generally of PVC) which are installed in pre-bored holes drilled to intersect the groundwater table or other water bearing zone which requires dewatering. The drains can be installed to lengths of 30 m and, under favourable conditions, to as long as 60 m. The pipes should be inclined slightly (minimum 10°) to allow gravity flow out of the drain. It is recommended that the pipes be sheathed with filter fabric to prevent the entry of silt and consequent clogging. Removable inner sleeves can be provided in the drains for periodic maintenance.

C.2.5 Counterfort Drains

These are of the same form as cut-off drains but are excavated downslope through the unstable or potentially unstable area. These drains improve stability primarily as a result of drainage although they may also provide a significant buttressing effect.

C.3 Other Methods of Stabilisation

C.3.1 Retaining/Support Structures

Retaining structures include those which rely on their self-weight to provide toe support, i.e. gravity structures such as crib walls and gabion walls, and those which resist movement by a cantilever effect, e.g. timber pole retaining walls and reinforced concrete cantilever retaining walls. Ground anchors and rock bolts rely primarily on their anchoring forces in the ground to support unstable or potentially unstable masses. A detailed discussion of the various types of structure is beyond the scope of this manual. There are however a large number of texts available on the subject of retaining wall design. Attention is drawn to the retaining wall design notes prepared by the Ministry of Works and Development, New Zealand.

C.3.2 Regrading

It may be feasible to reprofile the site by excavation to reduce the height or angle of a slope. Normally this will result in improved stability, although the validity of this assumption should be checked by analysis, as a reduction in the normal stress on a potential failure surface will result in a corresponding reduction in effective shear strength on that surface and may lead to failure.

There may be circumstances where removal of the unstable mass by excavation is possible, e.g. removal of a limited area of loose fill, scaling of potentially unstable blocks on rock faces, etc.

C.3.3 Biotechnical Stabilisation Methods

Reference has already been made to the generally beneficial effects of vegetation on slope stability. The use of selected vegetation, with or without associated structures, may be cheaper, more effective and more aesthetically attractive than the use of structures alone. A full manual for such work is provided by Gray and Leiser (1982).

It should be emphasised however that despite the beneficial effects of vegetation, it is difficult to quantify the improvement in stability achieved. Also, the long term effects cannot be relied on; vegetation may be removed by fire or as a result of gardening activities. Accordingly, for high consequence situations, biotechnical stabilisation should only be considered as a supplementary measure in conjunction with one of the other techniques mentioned previously.

C.4 References

East, G.R.W. (1974). Inclined Plane Slope Failures in the Auckland Waitemata Soils; Three Case Histories with Different Remedial Measures. pp 5.17 - 5.24 in Proc. Symposium on Stability of Slopes in Natural Ground, New Zealand Institution of Engineers, Wellington.

Gray, D.H. and Leiser, A.T. (1982). Biotechnical Slope Protection and Erosion Control. Van Nostrand Reinhold, New York, 271 p.

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