The Design of Rock Slopes and Foundations

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Summary

A critical review of the present "State of the Art" of the design of surface workings in rock is presented in this report which is divided into four sections:

1) Appraisal of rock masses
2) Design methods
3) Rock slopes
4) Rock foundations

Site investigation techniques, laboratory tests, mathematical and physical models are all examined in the light of their relevance to engineering design. The use of the factor of safety as a design index is discussed and an assessment is given of the most practical approach to designing rock slopes and foundations. The influence of groundwater on the stability of surface workings is considered and the use of drainage and grouting for groundwater control is discussed. Other methods for improving stability, including the use of controlled blasting techniques and the reinforcement of the rock mass, are considered.

Introduction

For centuries building on rock has been synonymous with building safely. During the past few decades this situation has changed and the increasing size of structures such as arch dams and opencast mines has presented engineers with an entirely new set of problems. The severity of these problems and the inadequacy of existing design methods has been emphasised by several catastrophic failures which have occurred in recent years.

The solution to these problems is not simple. Design methods in rock engineering evolve slowly, largely by trial and error since the physical and mechanical laws governing the behaviour of rock masses are poorly understood. Geologists, whose contribution to the development of rock engineering is vital, also find themselves in difficulty in attempting to quantify problems which have dimensions of both scale and time which are smaller than those with which the geologist is normally concerned.

As design methods are evolved, new problems which had not been anticipated arise. New failure modes or unusual combinations of forces are recognised and the rock engineer is faced with a new set of unsolved problems. It would be a mistake to regret this state of affairs. On the contrary, even if the engineer is frustrated by his inability to solve these new problems the very fact that these problems are recognised is a step towards increased safety.

In reflecting upon the current state of development of rock mechanics, the general reporters are greatly encouraged by one particular trend which has begun to emerge during the past decade and which suggests that the subject is slowly reaching maturity. This is the trend to work towards a balanced design; even if all the factors which contribute towards the overall behaviour of a structure are not known with any great precision, at least the influence of each factor is considered in arriving at an assessment of the probable behaviour of that structure.

In the past one tended to find "schools" or "techniques" emphasised. There was, for example, the "Austrian" school or the "South African" school and the "photoelastic" era and, more recently, the "finite element" era. While these individual approaches made and will continue to make valuable contributions to the development of rock mechanics, they did not provide a complete or a balanced picture of the whole. Just as the medical world has long realised that there is no one approach which will solve all the problems of illness, so the rock engineer is realising that no one method will solve all the problems which he is likely to encounter. Rock is an extremely complex engineering material and designing in rock requires the application of as much science as relevant, as much experience as available and as much common sense as possible. Above all, a design must be balanced in that every factor, even those which cannot be quantified, must be considered before reaching in final decision.

Turning now to the structure of this report on surface workings. Two sub-divisions are immediately obvious:

a) Rock slopes
b) Rock foundations

Flow charts showing the main steps required for the designs of these two types of construction are presented in Figures 1 and 2. It will be noted that there are many common elements in these two charts, particularly those areas concerned with geological data collection, preliminary stability analysis and shear strength testing. On the other hand, deformation behaviour is a crucial design consideration for foundations but not for slopes while controlled failure is acceptable for some slopes but totally unacceptable for foundations. This report is therefore divided into four major sections which deal with the problems which are particularly important in slope design and problems which are particularly important in foundation design.

Rather than present a catalogue of all the things which we can do well, the general reporters have chosen to place the main emphasis on those things which we do badly, where our knowledge is inadequate and where research is considered necessary. Many of the statements which are presented are controversial and certain parts may even be biased. This is because the general reporters are typical working engineers who have not attempted to read all the literature, who have not understood all that they have read and who have not necessarily formed unbiased opinions upon that which they have understood. This is a report on the state of the art in rock slope and foundation engineering as seen through the eyes of these two general reporters and it is hoped that it will stimulate others to look more closely at some of the questions raised.

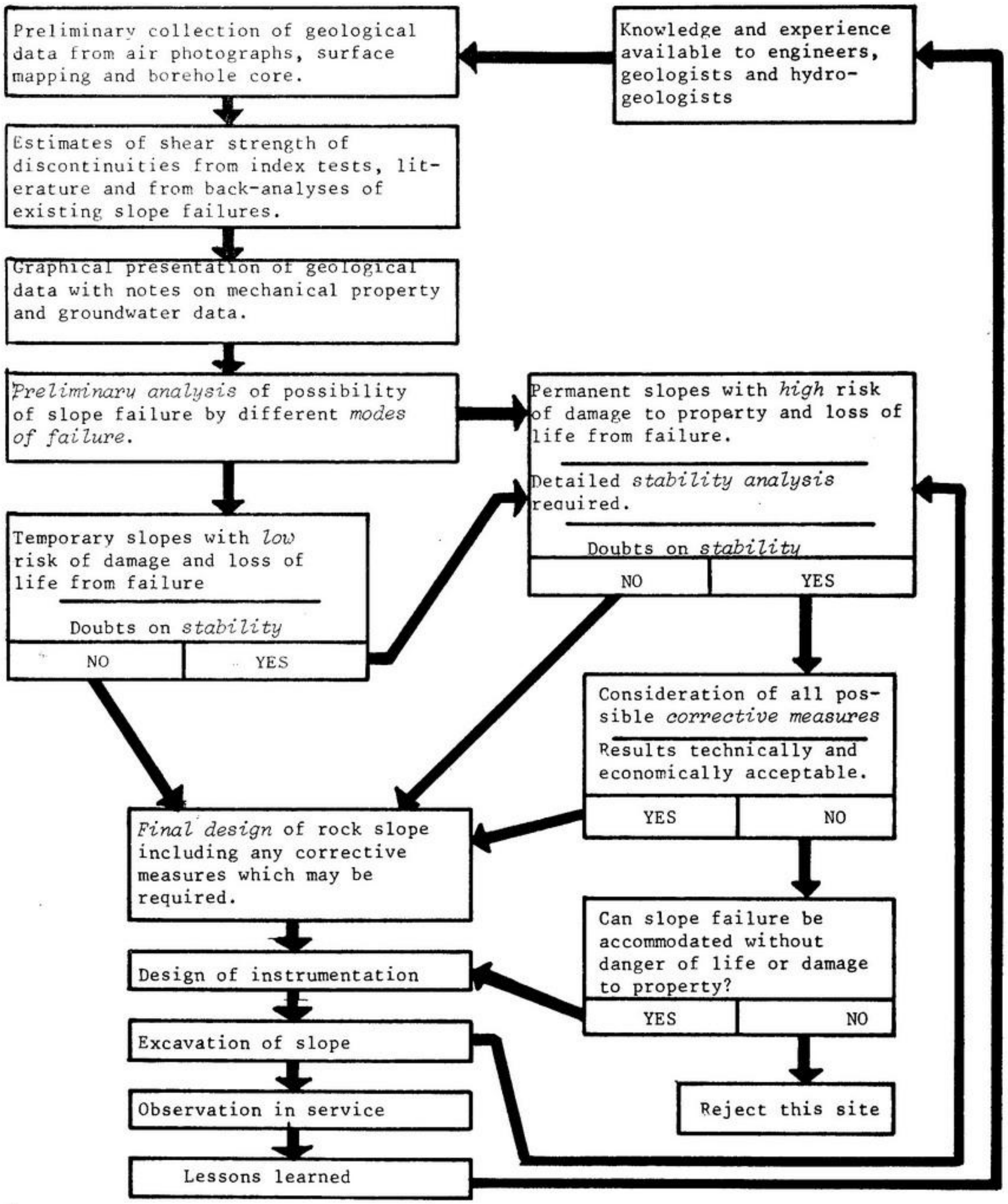


Figure 1. Rock slope design flow chart

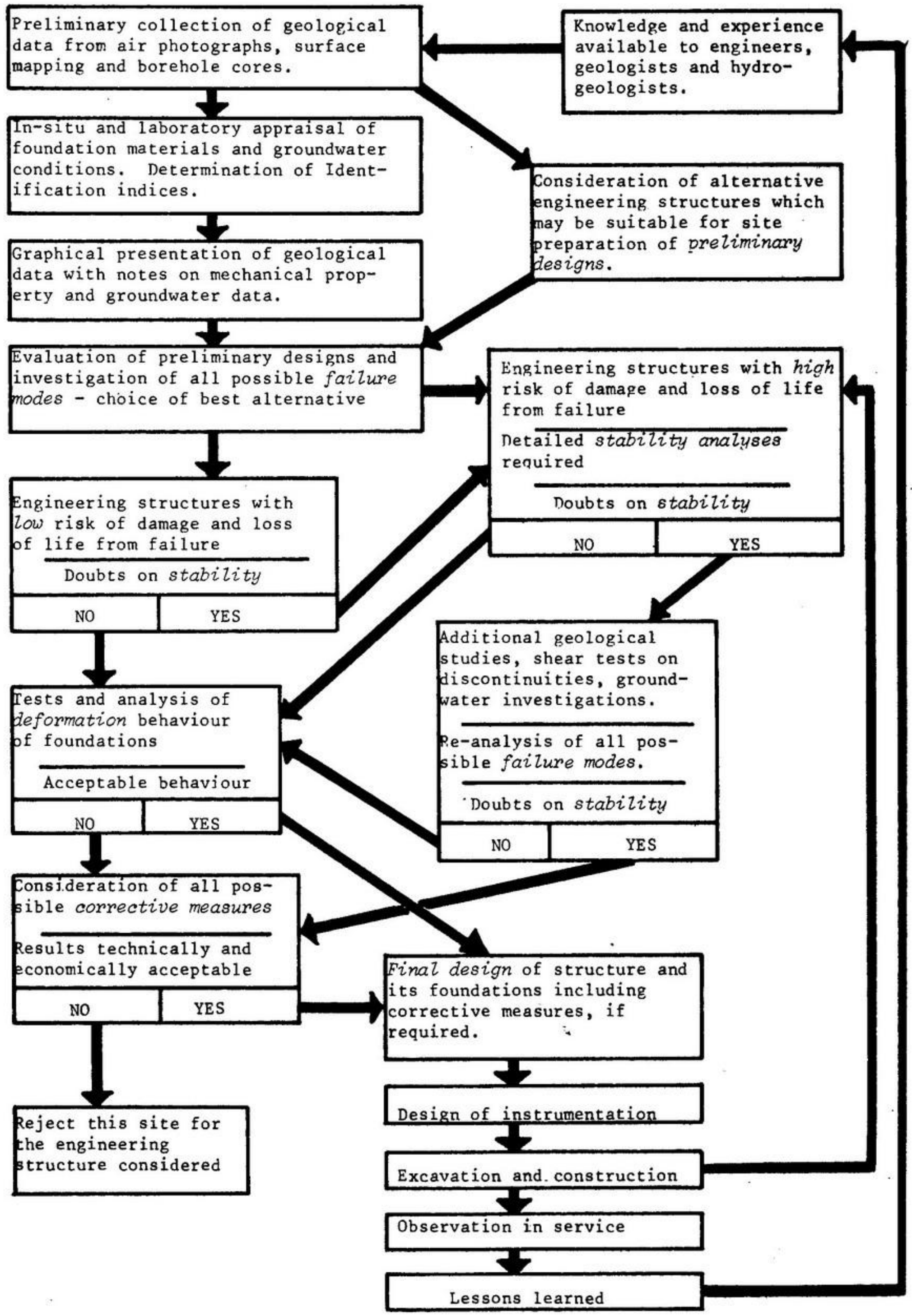


Figure 2. Foundation design flow chart

1. Appraisal of rock masses

1.0 Introduction

The engineering appraisal of a rock mass includes:

1. a qualitative estimate of the response of the rock mass to change in either geometry or loading. This includes an assessment of possible failure modes.
2. a quantitative measurement of parameters used in the numerical analysis of the behaviour of the slope or foundation.

Several means have to be used:

a. geology and hydrogeology
b. detailed description of the structure (geometry of discontinuities, infilling, etc.) and determination of engineering identification indices.
c. direct measurement of mechanical parameter meters for use in the analysis.
d. monitoring the behaviour of the rock mass with changes in load or with time.

Point (a) will not be dealt with in this report which is devoted to the mechanical aspects of slope and foundation behaviour. It is stressed, however, that *geology*, with its description of the rocks, their genesis and history and the sorts of features that characterise the region, together with *hydrogeology*, with its description of the groundwater regime, are vital for a complete understanding of the site.

1.1 List of methods of appraisal

Rock mechanics offers many methods for testing samples, investigation of rock masses and monitoring rock mass behaviour. Indeed, so many methods are available that many engineers are confused by the choice, sceptical about the reliability of the results and sometimes doubtful about the meaning of these results. The purpose of this report is to propose a selection of techniques which the general reporters consider most useful for the engineer who wants to know the significant engineering properties of rock masses.

Each method described is particularly applicable to a specific stage in the study of rock slopes or foundations. Some methods yield only rough qualitative indices which provide warnings or which facilitate comparison with other sites. Other methods supply quantitative measurements of variables which can be used for analysis.

The categories, which are common to both slopes and foundations, which are considered here, are in-situ investigations and laboratory tests. Instrumentation, together with methods of analysis, is required to fill different roles in rock slope and in foundation engineering and will be discussed under these headings later in the report.

The methods selected as the most reliable or the most promising are:

In-situ investigation
1. Mapping of structures on surface outcrops in exploration adits or on borehole cores.
2. Graphical presentation of structural geological data.
3. Geophysics
4. "Petite sismique"
5. Rock quality designation (RQD)
6. Lugeon tests
7. Jacking tests
8. Residual stress measurements

Laboratory tests
1. Compression and point load tests
2. Radial permeability tests
3. Shear tests on discontinuities

1.2 In-situ investigation

1.20 Introduction

This section is devoted to investigations carried out *on the site*. Some of these methods apply from the very first stage of the study while others can only be used when boreholes and adits are available. These methods are typical of recent developments in engineering geology.

There are tests other than those discussed here. The writers have selected only those which seem particularly relevant to the present purpose: the design of slopes and foundations as engineering structures.

1.21 Mapping of structural features

A description of the rock structure (geometry and nature of discontinuities such as faults and joints) is an essential ingredient in any analysis of rock slope stability or of foundation behaviour. The amount of detail required for different stages of the analysis depends upon whether one is designing a slope or a foundation and this difference is highlighted in Figures 1 and 2.

The rock slope engineer is frequently faced with the problem of designing miles of highway cut or open pit mine bench and it is clearly impossible to map all the structural features involved. Consequently the geological data collection is usually carried out in two stages, separated by a preliminary analysis which is intended to isolate critical slopes. Only these critical slopes are considered in detail.

On the other hand, the consequences of failure of a foundation are usually so serious that the preliminary design is carried out in much greater detail and the detailed geological data collection is required at a much earlier stage in the investigation. Since the foundation engineer is concerned with a particular site of limited extent, the amount of work is not usually excessive.

Mapping of surface outcrops of rock is one of the most reliable means of defining the structure of a rock mass. Mapping techniques such as those described by Broadbent and Rippere (1970) are well developed. Dangerous and inaccessible faces can be mapped by terrestial photogrammetric methods (Ross-Brown and Atkinson, 1972). In either case, appropriate corrections must be applied to compensate for mapping errors (Terzaghi, 1965).

These surface mapping techniques are most effective when applied to freshly exposed hard rock faces in slopes, trial excavations or in exploration adits although care must be taken to allow for blasting damage in these faces. Surface mapping is less effective when there is a considerable amount of over-burden soil or vegetation overlying the site or when the surface exposures are heavily weathered and the structural pattern ill-defined. In these cases, use must be made of sub-surface exploration methods.

Exploration adits, although by far the most expensive method of sub-surface exploration, are probably the most effective. Not only do they provide a large scale sample of the rock mass but, because the geologist can gain access to the interior of the rock mass, the nature and the orientation of structural features visible within the adit can be determined with considerable precision. Site investigation methods which do not provide information on the inclination and orientation of structural features are of little value to rock engineers since this information is vital in any stability analysis. With careful planning, these adits can be used for large scale drainage tests (Sharp, 1970) and can themselves become drainage and/or grouting galleries once the construction has commenced.

Trial trenches can only be used where the depth of overburden is small but, where this method is applicable, very valuable information can be obtained supplying a continuous perception of the rock and of its main geological features with no gaps over great lengths. Considering that excavation equipment for digging these trenches is readily available on most sites, it is surprising that so little use is made of this method for site investigations for rock slopes and foundations.

Diamond drilling is the most commonly used site investigation method for unexposed rock masses. Although diamond drilling equipment (Jeffers, 1966) and drilling methods (Rosengren, 1969) are highly developed, the results of a diamond drilling programme are frequently unsatisfactory. One of the major sources of difficulty is associated with core orientation. Unless the orientation of structural features visible in the core is known, the investment in a drilling programme will be largely wasted since the core will only be of qualitative value to the slope or foundation engineer. Methods of core orientation are available (Kempe, 1967) but, because they require careful treatment and because they introduce delays into the drilling timetable, these methods are disliked by most diamond drillers. The development of simple and reliable *core orientation systems* is a challenge to drilling equipment manufacturers and the successful development of such tools would represent a significant step forward in site investigation technology.

Of all the techniques used in site investigation, diamond drilling must surely be the one which is subjected to the most abuse. All too frequently, in order to satisfy a site investigation specification derived from some out-dated code of practice, an inexperienced driller is provided with antiquated drilling equipment and instructed to drill in a number of locations which have been chosen with little regard to local geological conditions. Payment on the basis of length of hole drilled rather than on the core recovered is also placing the emphasis incorrectly and the final result is usually of no use whatever. All core boxes should be systematically photographed, so as to keep a safe record of them. All too often the core boxes have disappeared when their examination is most required. Good colour pictures are adequate for checking important features.

Development of site investigation contract policies has simply not kept pace with development of equipment and with the *needs of the rock engineer*. This congress could benefit greatly from the presentation of a model diamond drilling contract for site investigations by an experienced geotechnical consultant who is familiar with the problems of negotiating such contracts in different parts of the world.

Sophisticated drilling techniques such as integral sampling (Rocha, 1967) although having great potential, are unlikely to gain wide acceptance while the quality of basic diamond drilling generally available is so poor.

Recognition of the difficulty of obtaining high quality diamond drilling has led some companies to advocate the use of *optical* or *television probes* for the examination of borehole walls. In theory, if such tools could be made effective and reliable, there would be no need for expensive diamond drilling and holes could be drilled with percussion equipment at low cost.

Unfortunately, this theory is far from realisation and currently available borehole probes are exceedingly costly and notoriously unreliable and are probably more expensive to use than high quality diamond drilling equipment. In the hands of specialised companies having the necessary technical expertise to maintain and to operate these units and to interpret the results, excellent results can be obtained particularly for the detection of thin soft layers which are likely to be missed by the coring. "Do-it-yourself" operations are to be avoided.

1.22 Graphical representation of structural data

The reader may consider it unusual that this topic is identified for special discussion and yet, when one considers that the graphical presentation of structural geology data is a vital link in the communications chain between the geologist and the engineer, it becomes obvious that this is not a trivial question. The graphical presentation of results which depend upon more than three parameters is a permanent source of worry for the engineer. Here we have more than ten variables, not all having the same significance, but all requiring presentation in a form which can be understood and utilised by the engineer. Further research into methods of data presentation would certainly be worth while. Improving the presentation would enable the engineer to understand the geological structure more clearly and to recognise mechanical behaviour patterns more easily. This improved presentation would also considerably ease the difficulties which occur in the dialogue between geologists and engineers.

Several methods have been proposed for presenting the three-dimensional geological structure of a site and some of these methods are briefly reviewed here.

Major features such as large faults can be drawn on a map, clearly showing their direction in space (e.g. Muller, 1963). Such maps are most important when considering the overall geological conditions of the site since smaller scale features which may have a more direct influence upon the stability of the site will usually be related to these major features.

One great difficulty in preparing geological maps is to recognise the *continuity* or *persistence* of structural features. Since a thin clay seam of large extent may be more critical than a large pocket of crushed material, the determination of continuity from outcrops and borehole intersections is an important part of this stage of the site investigation. The writers have found that scale models of the site (constructed from rigid plastic sheet or from rods) are extremely useful in this respect since it is possible to visualise the three-dimensional nature of the rock structure more easily (Figure 3). Duplicate models in the design and site offices will minimise misunderstanding.

Minor features such as thin joints, bedding planes etc. cannot be represented individually since there are too many of them and such features must be treated statistically in order to establish structural patterns. Polar diagrams, projections of a unit hemisphere, are widely used for this type of analysis (Phillips, 1971). The *equal area* projection (Figure 4a) is often preferred by structural geologists because it allows for easy plotting of the distribution frequency in space. The *stereographic* or equal angle projection (Figure 4b) is preferred by many engineers because all circles on the hemisphere remain circles on the projection and this property allows very convenient graphical treatment of stability problems. The errors in determination of the statistical distribution of structural features can be minimised by using a special grid for counting plotted points. The writers suggest that these counting errors have been over-emphasised since there are certainly systematic errors in the data collection process due to bias resulting from the direction of outcrops and adits in relation to the direction of the structures (Terzaghi, 1965).
Moreover, there are likely to be difference in the statistical results of two surveys carried out by two different teams. Hence, the writers suggest that the statistical treatment of structural patters and the subsequent graphical stability analyses can be carried out with comparable accuracy using either equal-area or stereographic projections. The choice of which method to be used can therefore be based upon convenience and personal preference.

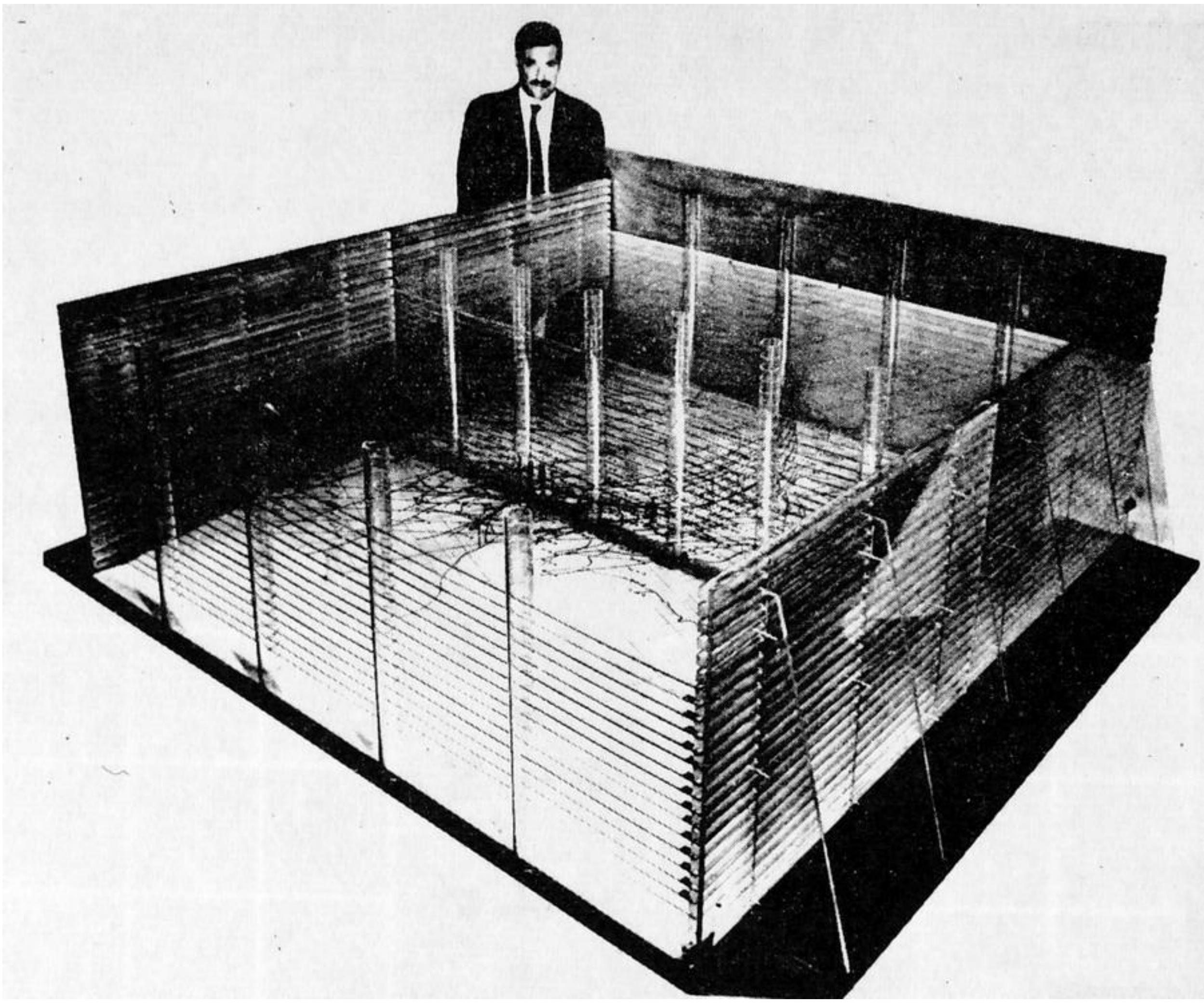
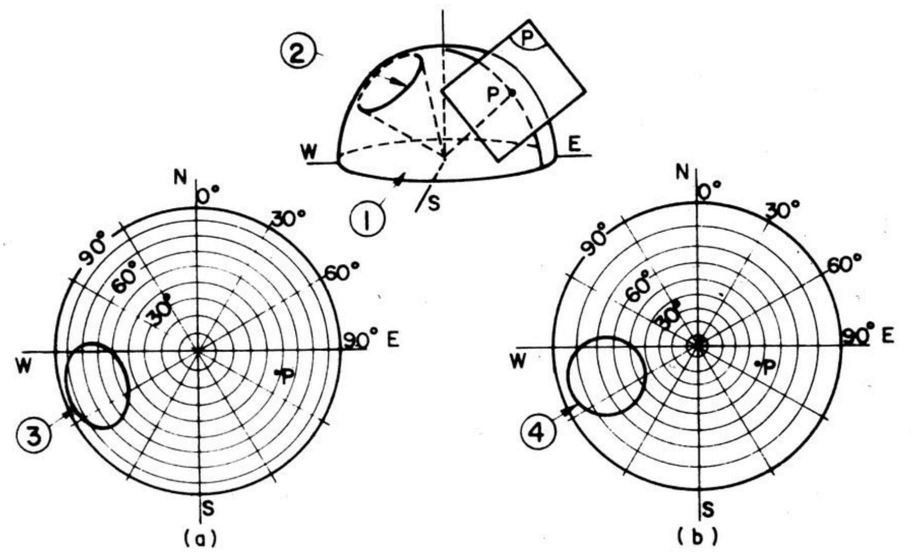


Figure 3. Plexiglass model of the geology of a dam site.

Use of these projections for presentation and analysis of structural data provides the engineer and the geologist with a very powerful tool. Once the user has become familiar with this tool, it is rapid, convenient and reliable to use. The general reporters wish to enter a strong plea that the use of these projections should form an essential part of any rock mechanics teaching programme.

In spite of the advantages of the methods already described, it must be pointed out that no one method of graphical presentation is entirely satisfactory because no one method can cover *all the parameters* of the problem: direction, spacing, continuity, opening, roughness and infilling of structural discontinuities. Hence, in addition to plots of structural patterns, the authors visualise the need for something similar to the grading curves used in soil mechanics. The development of such a system is a challenge to research workers in rock mechanics.

The surface roughness of structural discontinuities is a question of vital interest to rock slope and foundation engineers. The shear strength of the discontinuities and the permeability of the rock mass are significantly influenced by *dilatancy* of rough joints during shearing. This dilatancy is closely related to the shape of the surface irregularities and to the previous history of shear displacement. In other words, description of the surface roughness of joints at all scales is part of the geometric description of the rock structure, (Fecker & Rengers, 1971). How this description can be done is a vital question for discussion.

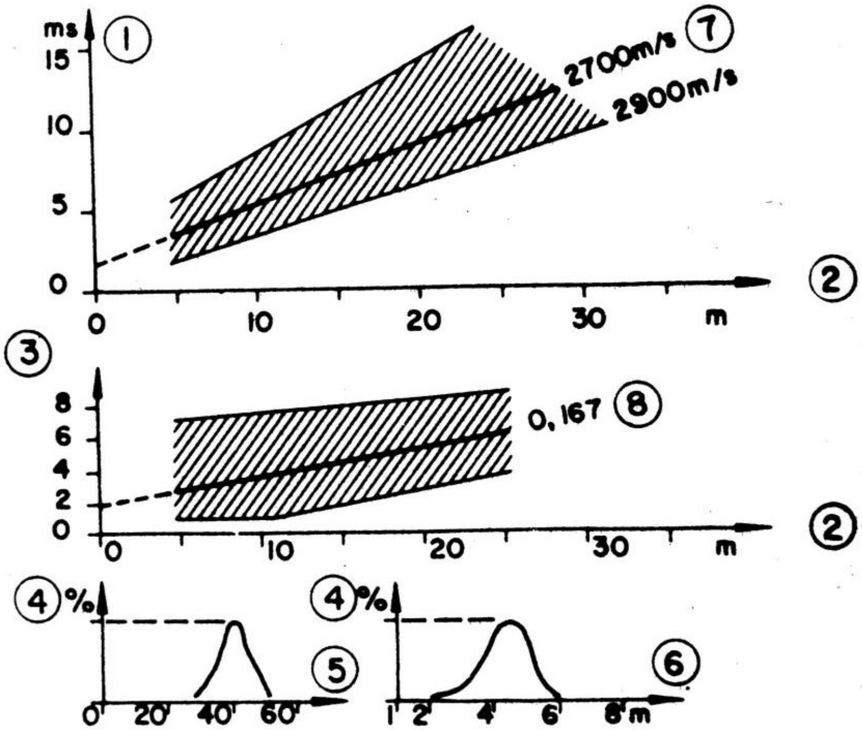


a) Equal area projection (Schmidt)
b) Equal angle projection (Wülf)
   1) Upper hemisphere
   2) Circle on sphere
   3) Projection of circle (not circular)
   4) Projection of circle (circular)

Figure 4. Point diagrams

1.23 Geophysics

Seismic refraction is a well established method used by geophysicists to measure the thickness of weathered rock or soil cover. It has proved extremely useful as a site investigation tool for rapid comparison between several sites. This method yields only a zoning of depth in terms of longitudinal velocities. It is well known that *longitudinal velocities* are not well correlated with other mechanical properties of rock. The question then raised is: can we rely upon this seismic survey for a first selection of sites?



1) Time for transverse wave
2) Distance between shock and geophones
3) Gain (dial units)
4) Frequency
5) Schmidt schlerometer readings
6) Half wave length (transverse wave)
7) Median velocity
8) Median attenuation

Figure 5. "Petite Sismique" card for a site, (Schneider 1967).

Another development which may in time play an important part in site investigation is that of *seismic logging* of boreholes as used by the oil industry. The advantage of these methods is that percussion drilling rather than diamond core drilling, can be used, resulting in a considerable cost reduction. Various types of logging tools are available and have shown promising results when applied to problems outside the petroleum engineering industry (Zemanek, 1968; Baltosser & Lawrence, 1970). Recent investigations (Lakshman & Allard, 1971) have shown that there is a good correlation between the fractured density within the rock mass and the transverse velocity of the seismic signals.

Finally, the recent improvements in *gravimetry* have made it possible to use this geophysical method for the detection of voids in rock formations. It has been successful since 1970, when high sensitivity gravimeters were built by Lacoste-Romberg, for localising buried quarries or karstic channels.

1.24 "Petite Sismique"

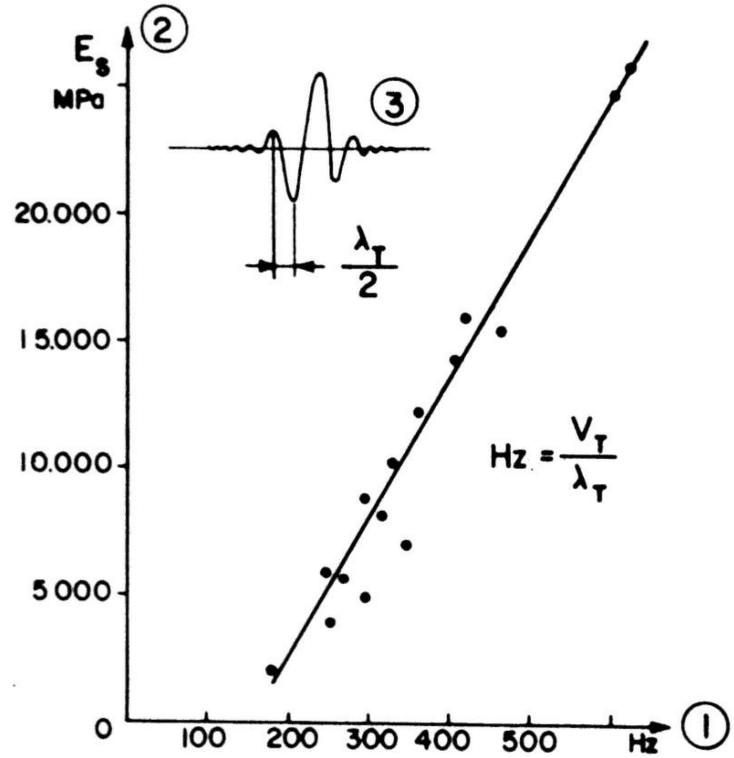
The method called Petite Sismique (Schneider, 1967) is entirely different in its principle of operation. Instead of one, several *seismic parameters* (particularly transverse velocity, wavelength and attenuation) are measured and shown on the card (Figure 5). Somewhat similar to a passport, which does not full describe its holder, but identifies him sufficiently for police officers, the Petite Sismique gives the *identification* of the site and enables the differences, or similarities, with other sites to be detected. This technique has been successfully used in a number of countries and probably deserves to be used more widely. Calibration of a qualitative index of this type can only be achieved by collection and comparison of the results of many successful applications.

Quantitative correlations have been established between Petite Sismique parameters and other engineer-parameters (e.g. Figure 6). Considering that Petite Sismique survey requires only one engineer for a relatively short space of time, it appears to be a cheap wav of getting useful information on a given rock foundation. The only condition is that there should be enough rock exposed, either in outcrops or preferably in adits.

1.25 Rock quality designation (RQD)

The rock quality designation (RQD) (Deere, 1968) is an index of core recovery obtained by summing the length of pieces of core *longer than 10 cm* and dividing this length by the total length of hole. It is an index of fracture frequency and has proved very useful on many sites for estimating the depth of excavation required before good quality rock suitable for foundations is reached. One of its main advantages is its extremely low cost; the computation of RQD for hundreds of metres can be done in a few hours, either on site or from photographs of the core boxes.

The main question is whether the quality of workmanship can influence the length of individual core pieces and hence the RQD value. It is believed that, provided the drilling operations are carried out by qualified personnel using modern equipment to produce core of at least 50mm in diameter, the RQD or similar fracture frequency indices are useful guides to the mechanical characteristics of a rock mass.



1) Frequency of transverse wave signal (Hertz)
2) Static modulus of deformation (MPa)
3) Transverse wave seismogram.

Figure 6. Correlation between static modulus of deformation and frequency of transverse wave signal obtained by "Petite Sismique" for various rocks.

The presentation of results of the successful experiences involving the use of RQD would be useful in clarifying some of the uncertainty associated with these techniques and in convincing sceptical engineers of their value.

1.26 Lugeon test

This well known test, originally proposed by Maurice Lugeon as a criterion for groutability, is widely used to estimate the permeability of rock masses. The test involves packing off a section of borehole and measuring the amount of water which can be injected into the rock mass through this section in a given period of time and at an excess pressure of 10 kg/cm^2 (1 M Pa).

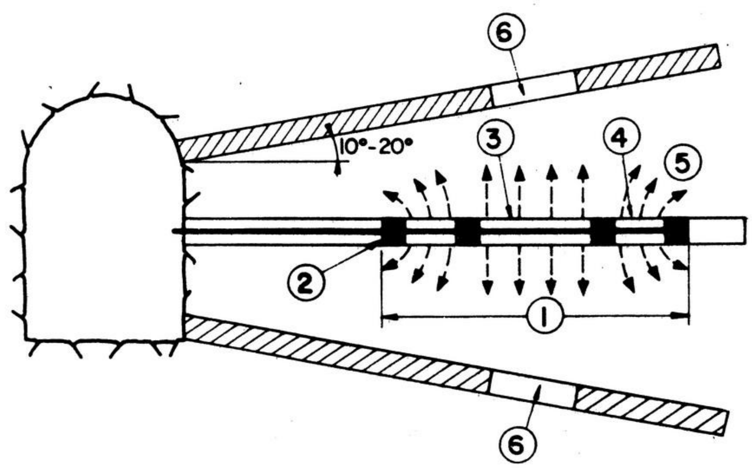
Several authors argue that this test is invalid in rock because an excess pressure of 10 kg/cm^2 is sufficient to open discontinuities and to change the permeability or the hydraulic conductivity of the rock mass. Indeed, if a great deal of trouble is taken to orient the hole at right angles to the set of fissures in which permeability is to be measured, to vary the packer spacing and the pressure of injection, a great deal of information can be deduced on the spacing and the opening of discontinuities. A further refinement to the Lugeon test, involving the use of four packers instead of two, has been proposed by Louis (1970). In this test, the central section between the second and third packers is the measuring section, while the two outer sections act as flow barriers which are designed to ensure that radial flow occurs in the measuring section (Figure 7).

A question that the general reporters raise is: are we really improving the Lugion test which is extremely simple and adequate for most sites? The modifications to this test described above may give the illusion of great accuracy but this accuracy may not in fact be obtainable in a medium as complex as rock.

While the reporters accept the need for the adoption of a scientific approach to the very difficult problem of water seepage in rock masses, they also feel that there is a need for a clear and unambiguous presentation of the results already achieved, so that the general reader can judge for himself whether progress is being made in this field. One question which will be discussed in more detail later in this report, but which has a bearing on the use of the Lugeon test, is: does the flow net concept derived from the consideration of flow through porous media apply to rock masses or is it necessary to use an approach based on flow through individual discontinuities?

1.27 Jacking tests

Jack tests designed to determine the modulus of elasticity of a rock mass are more relevant to foundations than to rock slope design. Nevertheless, these tests are discussed in this common section because there are some cases in which results of jacking tests may give information on rock mass behaviour, which is useful to the rock slope designer.



1). Probe
2). Packer (0.8 m long)
3). Central measuring section (2 to 5 m long)
4). Outer flow barrier section (2 m long)
5). Flow lines (radial in central section)
6). Piezometers in lined holes

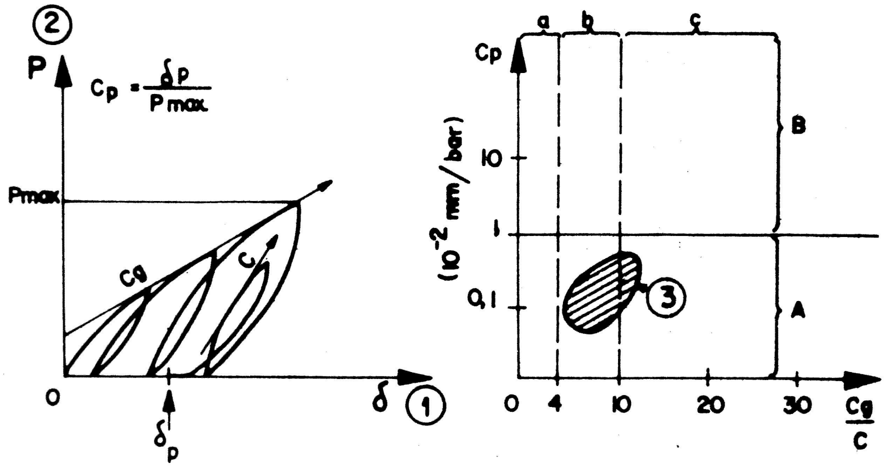
Figure 7. Hydraulic triple probe for water tests (Louis 1970).

Most jacking tests are interpreted in terms of the Boussinesq equations which provide a relationship between measured load and displacement and the modulus of elasticity. Since these equations are only valid for an elastic continuum, their use yields a modulus of elasticity for an "equivalent" continuous medium. Consequently, the first question which arises is: can the modulus of elasticity obtained by a jacking test be applied to the design of an engineering structure founded on a discontinuous rock mass?

Closer examination of the results of a lacking test shows that the relationship between load and deformation is generally non-linear. In other words, it is possible to infer from a given test several values of deformability depending on the magnitude and the sign (loading or unloading) of the applied load. In fact, these non-linear curves can be used as an additional identification index for the rock mass (Schneider, 1967). Correlations with other engineering properties have shown that various slopes of the curves (Figure 8) are indicative of the fracture frequency and the mechanical behaviour of the rock mass. These identifications indices may be useful during the preliminary site investigation. The non-linear load deformation curves obtained in jacking tests are also useful in establishing the stress limits beyond which the concept of modulus of elasticity becomes meaningless and where a foundation design based upon elastic theory could not be considered reliable.

Jacking tests are usually performed in adits where the reaction to the applied load is provided by the opposite side of the gallery. Surface tests can also be carried out if the load reaction is provided by deep anchors (Stagg 1967). The main point of controversy in the use of these devices relates to the size of the loaded area and the magnitude of the applied load; small load area and high stress, or large load area and low stress? The second alternative is more expensive but probably closer to the conditions which will apply to the full scale structure. In fact, the crucial point of this argument is the question of what effect the scale of the structure has upon the foundation deformations. It is unlikely that this question will be resolved by theoretical discussions and what is needed is a correlation between jacking test results and the deformation of foundations measured on full scale structures. Some attempts have been made to establish such correlations (Ward and Burland, 1969) but it cannot be claimed that this question has been adequately resolved.

Borehole Jacks have been developed in several countries and have the advantage of being capable of application at depth within a rock mass. The question of scale effect is even more important in this case and many engineers will remain sceptical about their use until it has been convincingly demonstrated that the results are relevant to full-scale foundation design. The walls of a borehole, however, are less disturbed than the walls of an adit excavated by blasting. This is in favour of the borehole jacks.



1) Displacement of plate
2) Plate stress
3) Points for a given site

C, Cg Slopes.
Cp in 10^{-2} mm per bar
δ_p irreversible displacements

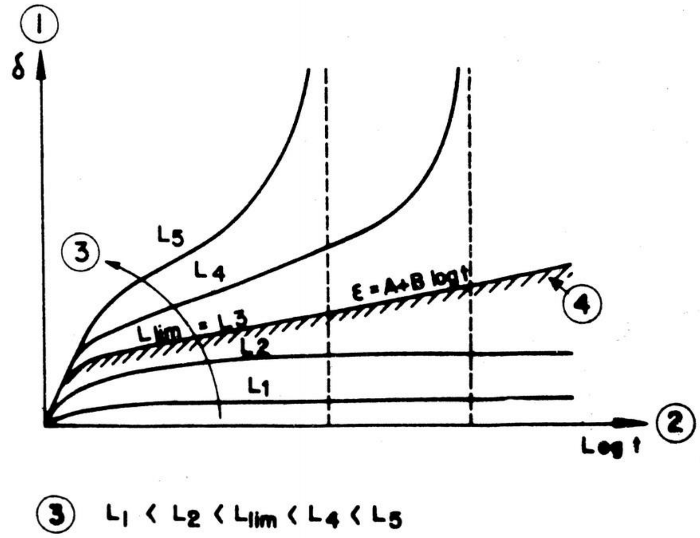
A	Zone of practically elastic deformations
B	Zone of important irreversible deformations
a	Zone of compact rock
b	Zone of average rock
c	Zone of open jointed rock

Figure 8 : The jacking test and its interpretation,, (Schneider, 1967).

Many jacking tests show that deformation is *time dependent.* It is therefore interesting to investigate the effect of sustained loading, although such tests are not often carried out in situ because of the high cost and time requirement, It has been suggested that the maximum strain rate under constant load that can be accepted is that corresponding to the upper linear curve in Figure 9, in which strain is slotted against the logarithm of time. The load giving this behaviour is the maximum permissible load above which failure of the foundation will occur after a finite lapse of time. The tests can be carried out by plate loading at the rock surface, or by the use of borehole jacks. The influence of the scale of the test, upon the time dependent characteristics measured is an important point requiring further investigation.

1.28 Residual stresses

Before applying a new load to a rock foundation, it may be important to know the magnitude of stresses of tectonic origin which already exist within the rock mass. A knowledge of these stresses is less important to rock slope engineers, although there may he cases where high stresses can develop near the surface, e.g. at the toe of a high cliff.



1) Displacement
2) Time (logarithmic scale)
3) Limiting load ($L_{lim} = L_3$)

Figure 9. Displacement versus time - Limiting load

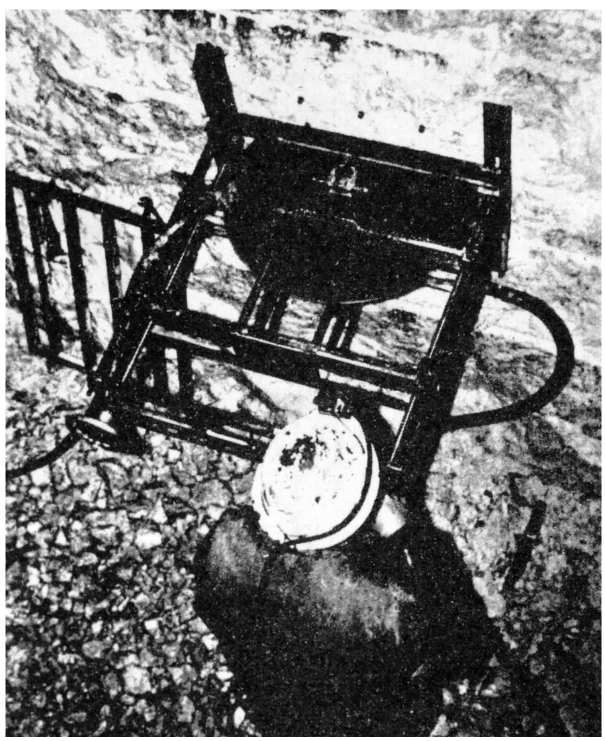


Figure 10. Flat jack test - cutting a slot with a circular saw.
(Photograph by courtesy of S.E.I.I., Paris).

One method of stress measurement is to use a *flat jack* which is inserted into a slot cut into the rock and pressurised to restore the readings on a deformation gauge set across the slot. This method has the advantage of giving a direct measurement of the stress acting across the slot, but it has the disadvantage of being limited to shallow depth from the rock surface (Figure10).

An alternative method is to use electrical resistance strain gauges, or photoelastic transducers which are glued into the borehole and stress relieved by over coring. These methods permit the measurement of stresses at depth within the rock mass, but the interpretation of the results, particularly in an anisotropic rock system, is difficult and there are sometimes significant variations between the measurements carried out at adjacent points in the same borehole.

In view of the relative unimportance of residual stress results in the design of surface workings, it is not considered appropriate that stress measuring techniques should be discussed in greater detail in this report. It is, however, hoped that the rock engineer's ability to measure stress will be improved as a result of the research activities of those who are concerned with underground excavation design and to whom residual stress is a crucial issue.

1.3 Laboratory tests

1.30 Introduction

Only a limited number of tests which can be carried out in the laboratory are considered relevant to rock slope or foundation design. The reason is that the behaviour o f the rock mass is governed by the orientation and nature of the discontinuities in the rock mass, whereas the samples sent to the laboratory generally consist of the stronger rock material. There are, however, two reasons for studying samples in the laboratory. The first is that the behaviour of the material gives a clue to some of the problems which are likely to arise on the scale of the rock mass. In fact, the rock material is often a small scale model of the rock mass because it has passed through the same tectonic and geological history and the small scale features in the material are frequently closely related to the large scale features in the rock mass. Consequently, a test on a small sample of intact rock can frequently give a useful *identification index* which can assist in the engineering appraisal of the rock mass. A second-reason for laboratory testing is that of convenience, provided that it is possible to obtain samples of rock and particularly of rock containing discontinuities such as bedding planes or joints. The best place to carry out these tests is in the laboratory. It must be emphasised that the laboratory need not be located in London or Paris and that a hut or caravan on some remote site can be an effective location for laboratory type work. The term laboratory testing is used here to differentiate between those tests which are carried out on samples which have been removed from the rock mass, and those tests carried out in situ.

The tests discussed here are only a few of those which can be carried out in the laboratory in order to understand the behaviour of rock. It has been assumed that the general reporter of Theme I will cover this subject more thoroughly and that only those topics of direct relevance to the design of surface workings will be dealt with in Theme III. It may be argued that many properties other than those discussed hereunder are useful for the study of rock properties required in the design of slopes and foundations. This question is open for discussion but the tests described are considered by these general reporters to be adequate for design purposes within the framework of currently available knowledge. These tests are:

1) Compression tests (including point load tests)
2) Radial permeabilitv
3) Shear strength of joints

1.31 Compression tests

The uniaxial unconfined compression test is a cheap and easy means for obtaining an identification index of the rack material. More elaborate tests, which are extremely numerous, have little practical value for slope and foundation design. The results of uniaxial compression tests, like all other tests on rock, invariably show a significant amount of scatter. This scatter is associated with the discontinuous nature of rock; the engineering properties being governed by discontinuities which may range from grain boundaries on a small scale to joints and faults on a large scale.

Some authors have argued that the amount of scatter associated with the uniaxial compressive test is reason enough for the test to be discarded. On the other hand, some argue that the amount of scatter gives a useful qualitative indication of some aspects of the nature of the rock mass. The general reporters suggest that the following qualitative indications may be obtained by uniaxial testing: (a) the mean value of strength allows an initial *classification* of the site, (b) the variation in strength from one zone to another gives an indication of the *heterogeneity* of the site, (c) variation in strength with the orientation of the sample gives an indication of the possible *anisotropy* of the rock mass, (d) scatter of the results of small sample tests gives an indication of the *microfracturing* of the rock as a result of previously applied tectonic stresses.

In order to minimise the time and expense involved in preparing the ends of specimens for uniaxial compression testing, it has been suggested that *point lead* or *Brazilian tests* yield results of comparable accuracy. In these tests an unprepared piece of rock core is loaded between two points (Figure 11) and the core is split as a result of tensile stresses developed across the core. This test, which is extremely cheap and quick to use during site investigations, provides results which are closely related to the strength of the rock material (D'Andreas et al., 1965).

In addition to strength testing, measurement of the *modulus of elasticity* on cores of rock subjected to uniaxial loading is a basic means for determining this property of the rock material. This value must obviously be reduced when considering the deformation of a rock mass and the extent of this reduction is a question requiring further investigation.

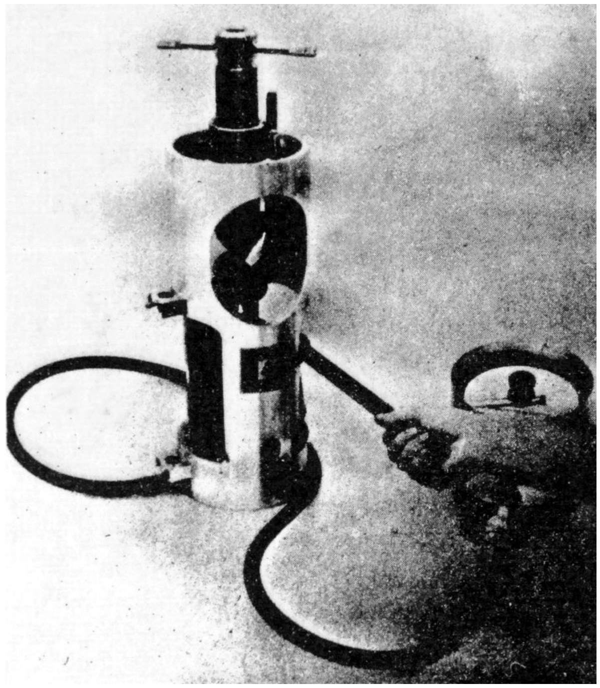


Figure 11a. Machine for point load strength determination.
(Manufactured by Robertson Research)

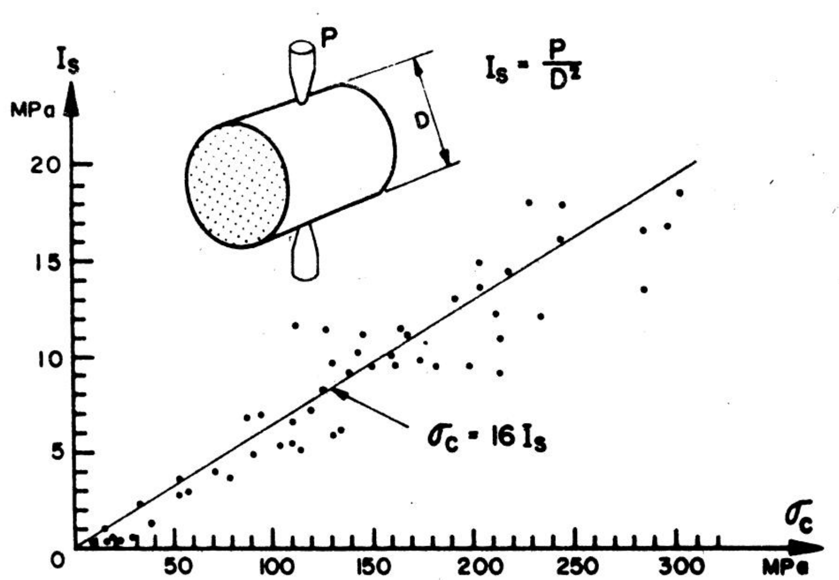
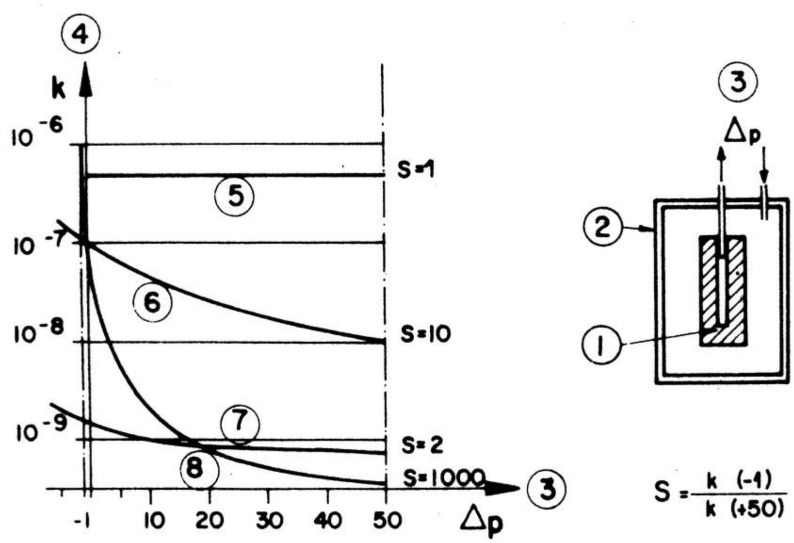


Figure 11b. Relationship between point load strength index Is and uniaxial compressive strength σ_c.

1.32 Radial permeability

Radial permeability is also an indirect measure of the degree of fracture of a sample of rock material (Bernaix, 1967). In this test, cores with an axial hole (Figure 12) are subjected to radial percolation of water under pressure. The index measured in this test is the ratio S = k(-l)/k(+50) in which k(-l) is the permeability measured for convergent flow under a differential pressure of 50 bar. When S is high the rock material permeability is very *sensitive to applied stresses*, a phenomenon which is typical of fractured rock. The main value of this test is not for the measurement of the permeability of the rock material, which generally has little influence on the hydraulic behaviour of a rock mass, but of the degree of fracturing of the rock material. A great number of tests have shown the existence of correlations between the ratio S and the scatter of strength values, or the scale effect on strength.

The use of this simple test is therefore similar to that of the uniaxial compression test. The value obtained is, however, more clearly related to the degree of fracture of the specimen and it has little relationship to the mineral composition of the rock.



1) Rock sample with axial hole
2) Pressure cell
3) Water pressure differential Δ_p (bar)
4) Permeability "k" (cm/s)
5) Oolithic limestone (no fissures)
6) Gneiss (average)
7) Gneiss (compact)
8) Gneiss (fissured)

Figure 12. Radial permeability test and curves for various values of index S (Bernaix 1967)

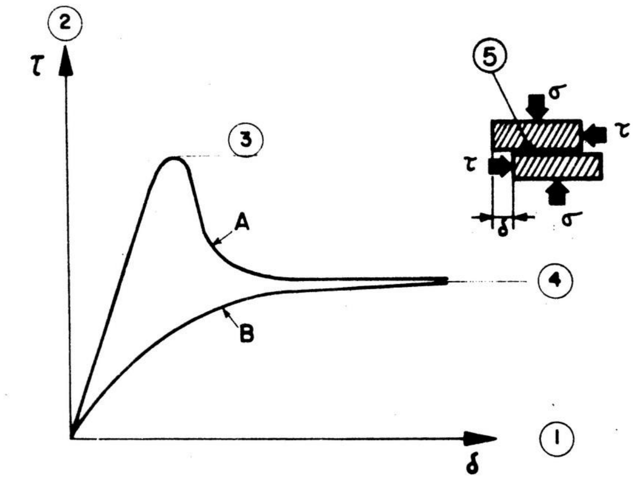
1.33 Shear strength of discontinuities

Because the stresses acting on rock slopes and foundations are low, fracture of intact rock is seldom involved in the failure of these structures; their mechanical behaviour being governed by shear movement on discontinuities such as faults and joints. Consequently, determination of the shear strength of these discontinuities is a question of fundamental importance in the design of surface workings.

The surfaces of separation (stratigraphic layers such as bedding planes and geologically induced fractures such as faults and joints) have a tensile strength which is for all practical purposes zero, and a shear strength which depends on wall roughness, the infilling material and the amount of imbrication (arrangement of individual blocks). The most dangerous for stability are obviously the surfaces that are planer, smooth, filled with soft materials, of large area and not interlocked. This is the case of shear fault. Less dangerous discontinuities are those which have not been subjected to large shear displacements in the geological past and where there is some interlocking of surface roughness or cementing of the surfaces by precipitated infilling.

The difference in mechanical behaviour between these two types of surface is illustrated in Figures 13 and 14 in which shear stress is plotted against displacement and against normal stress. In the case of rough surfaces (curve A in Figure 13) interlocking of surface irregularities causes the sample to behave in an approximately linear-elastic manner for small displacements. At a given displacement, the peak shear strength of the surface is overcome as a result of over-riding or shearing through of the interlocking irregularities and a rapid drop in shear strength occurs as displacement is continued. Eventually, when the surfaces have been ground smooth, a residual strength value is reached.

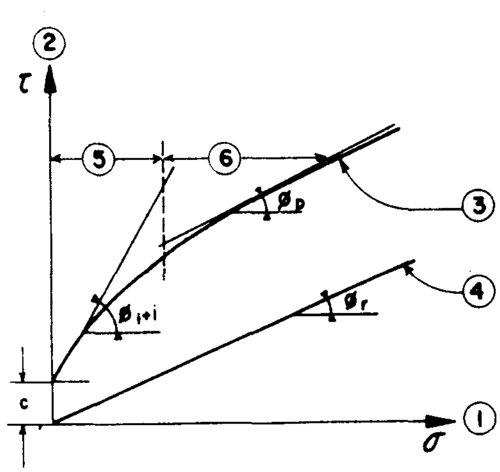
Considering the values of peak and residual strengths for various applied normal stress levels, the curves illustrated in Figure 14 are typical of the behaviour of rock surfaces. In the case of the peak strength behaviour, a small value of cohesion c may be present due to cementing of the surfaces. The curve relating shear strength and normal stress is generally non-linear as illustrated. This curve is steeply inclined at low normal stresses as a result of the interlocking of surface irregularities. Because of the high strength of the rock material from which these irregularities are formed, shear displacement at low normal stress takes place as a result of overriding or *dilation* in which the irregularities move over one another and the total volume of the specimen is increased. The slope of the curve at low normal stresses can be approximated by the angle ($\phi_i + i$) where ϕ_i is the friction angle of the material surface and i is the average angle of incidence of the surface irregularities to the direction of shearing (Patton, 1966). As the normal stress increases, the dilation of the specimen is inhibited and fracturing of or shearing through the interlocking surface irregularities commences. Eventually, the shear strength of the surface is controlled entirely by the shearing through of these irregularities and the inclination of the curve approaches the peak friction angle ϕ_p of the rock material.



1. Displacement
2. Shear stress
3. Peak shear strength
4. Residual shear strength
5. Pre-existing discontinuity

A Rough surface which has not been subjected to previous displacement
B Smooth surface which has been subjected to large displacement

Figure 13. Variation of shear resistance with displacement on a discontinuity.



1. Normal stress
2. Shear strength
3. Peak strength
4. Residual strength
5. Dilation
6. Shear

Figure l4. Variation of shear strength with normal stress for peak and residual strength.

In the case of the smooth surface (curve B in Figure 13), the residual strength behaviour is defined by the friction angle ϕ_r and the cohesion is, for all practical purposes zero. Note that the friction angles ϕ_p and ϕ_r are not necessarily equal since the infilling material in the case of the smooth surface may have been altered by weathering.

An extremely important point which Figure 14 is that the residual strength surfaces is *not influenced by the scale of the test*. This is because the friction angle ϕ_r is a dimensionless number and, provided that there is intercept, its value can be determined by tests on small samples (Londe, 1973). On the other hand, when large shear displacements have not already occurred in the geological past and when the sample peak strength behaviour (curve A in Figure

13), both the cohesion c and the roughness angle *i* will depend on the scale of the specimen tested. The basic question which must be considered here is: can the values of cohesion and roughness angle determined in small scale laboratory tests be relied upon for the large engineering structures?

This question can only be answered by considering the behaviour of full scale engineering structures, such as rock slopes and Figures 15 and 16 illustrate an example of this type of analysis. In Figure 15, the results of a number of shear tests on porphyry joints are plotted and the lines A, B, C and D define the limits of scatter of the peak and residual strength values. In Figure 16, critical slope height versus slope angle relationships have been derived from the results given in Figure 15 and are compared with the slope height-slope angle relationship for nine porphyry slope failures in the Rio Tinto area. It will be noted that all the slope failures fall within the region defined *by the residual strength parameters*, although it should be noted that a small cohesion intercept (0.1 mPa) has been assumed for this analysis (Hoek 1970). Note that, unless the small cohesion value is included, all slopes should have failed at the residual friction angle of approximately 35º.

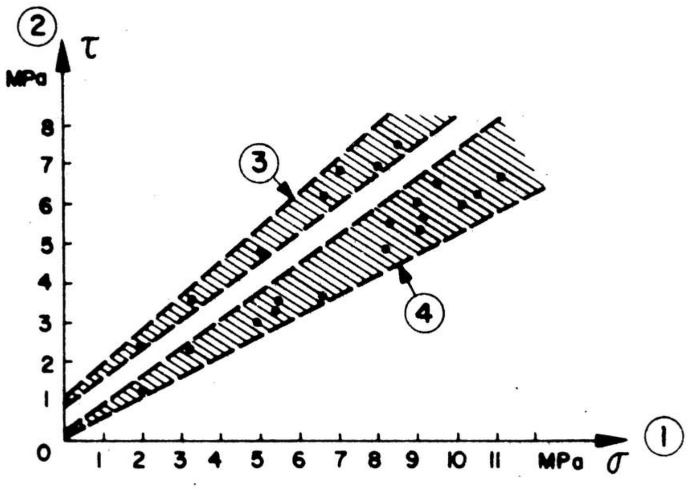
In Figure 17, values of cohesion and friction angle have been plotted from the results of a number of analyses, similar to that discussed above. Many of these results have been determined from relatively short term failures in small slopes and, included in the diagram, an arrow gives a qualitative indication of the influence of time and scale of the structure. This is a question which obviously requires a great deal of research and discussion but, as a result of their own experience, the reporters propose the following general rules:

a) When a very large structure such as an arch dam or major building foundation is being designed for conditions of long term stability (more than 100 years), it is recommended that the design based on zero cohesion and a residual friction angle ϕ_r, which can be determined in small scale laboratory tests.

b) Where temporary rock structures of limited size are being designed, it is permissible to allow some cohesion and non-linearity of the shear strength versus normal stress curve, provided that these values are checked against typical values obtained from back analysis of failures in similar materials.

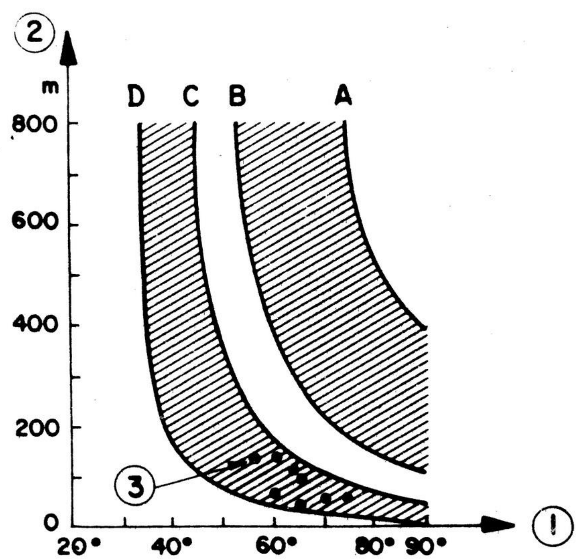
The general reporters regard it as irresponsible engineering practice to attempt to calculate the value of cohesion from the intact strength of small scale rock samples.

Several different types of direct shear machines have been designed and two typical designs are illustrated in Figure 18 and 19. The machine shown in Figure 18 is capable of testing relatively large specimens (approximately 400mm x 600mm) while the small machine shown in Figure 19 is designed for testing pieces of core or small hand samples. Friction angles measured in either of these types of machines tend to compare very well and, since the reporters do not advocate the determination of cohesion by laboratory testing, it makes little practical difference which one is used. The only reason for using the large one is when one has to test a thick joint.



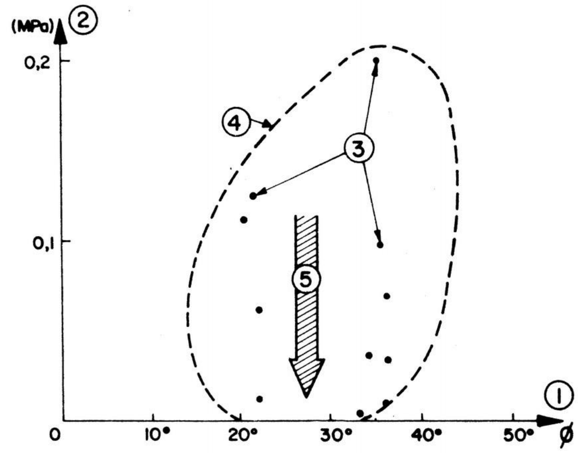
1) Normal stress (MPa)
2) Shear strength (MPa)
3) Peak strength
4) Residual strength
5)

Figure 15. Shear strength results for porphyry joints from Rio Tinto in Spain.



1) Slope angle (degrees)
2) Slope height (meters)
3) Slope failures

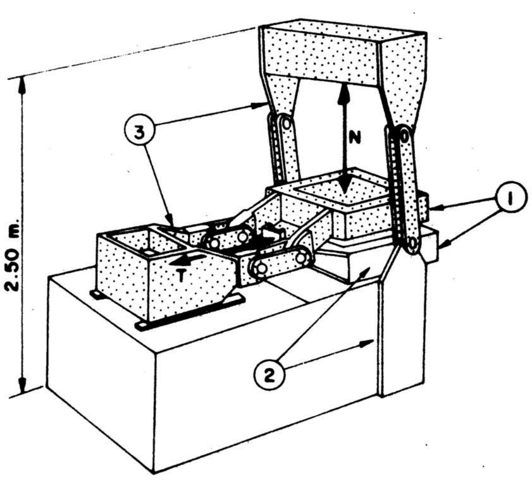
Figure 16. Critical slope height versus slope angle relationships derived from Figure 15, compared with mine slope failures in porphyry.



1) Friction angle ϕ (degrees)
2) Cohesion (Megapascals) (1 MPa = 10 Kg/cm^2 = 142 lb/in^2)
3) Each point corresponds to an observed slope failure.
4) Boundary of suggested range of values which can be used for slope design.
5) Influence of time and size.

Figure 17. Relationship between cohesion and friction angle determined by back-analysis of slope failures.

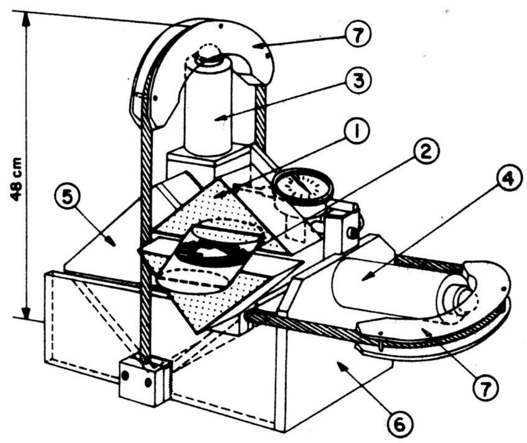
In order to estimate cohesion from large scale shear tests, many authors have reported the results of shear tests carried out in situ. In these tests, the specimen is cut free from the surrounding rock mass, with the exception of one side which is left attached.



N Normal force
T Shear force)

1. Upper and lower parts of shear box
2. Fixed parts
3. Moving parts

Figure 18. Direct shear apparatus for testing rock joints in the laboratory (Londe 1973).



1) Concrete or plaster cast specimen mount.
2) Shear surface.
3) Normal load jack.
4) Shear load jack.
5) Upper shear box.
6) Lower shear box
7) Rope load equaliser

Figure 19. Portable shear machine for use in site laboratories. Weight 400 N. (Hoek and Bray, 1973)

Shear and normal loads are generally applied by means of flat jacks or hydraulic ram jacks and, because of the large size of the equipment required, these tests are extremely expensive. These general reporters do not recommend in situ shear testing except under very special circumstances. Many readers may wish to disagree with this recommendation and the reporters would welcome a general discussion on this topic.

2. Design methods

2.0 Introduction

Before going on to discuss the design of rock slopes and foundations, it is necessary to consider the general question of how a design in rock should be approached. Having accumulated data on the geometry of the rock structure, the mechanical properties of the rock mass and the groundwater conditions, how is this information to be processed in order to arrive at as assessment of whether the overall design will be satisfactory?

Considering the large number of parameters which are involved in defining the behaviour of a rock mass, the fact that their measured values will be widely scattered and their inter-relationships ill-defined, it is clear that a precise assessment of the performance of the rock mass is not possible. In addition, it must be kept in mind that different criteria will have to be satisfied, depending upon the purpose of the rock structure. Hence, a safe slope may be regarded as one which remains standing for the duration of its working life while a foundation may be regarded as inadequate because of differential movements of relatively small magnitude which can induce failure in a structure such as a concrete dam.

In spite of these difficulties, it is, never-the-less, clearly necessary that some form of quantitative assessment of the performance of the rock slope or foundation should be attempted. The following chapter gives a brief review of the methods which can be used together with comments on the usefulness and limitation of each of the methods. Detailed discussions on the application of some of these methods to the design of rock slopes and foundations are given later in this report.

The following topics will be discussed in this chapter:

1) Model studies
2) Mathematical models
3) Limit equilibrium methods
4) Mechanical effects of water pressure
5) Factor of Safety

2.1 Model studies

Mechanical and civil engineers have made extensive use of models as design tools for many years. Hence, a complex component for an aeroplane, a car or a bridge can be made up at low cost as a reduced scale model and tested to destruction. Because the materials used are man-made and their behaviour is well known, precise model laws can be used for the interpretation of the results of such model tests. Consequently, such models are valid and valuable design tools.

Because of the difficulties involved in studying the behaviour of full scale rock structures, it is not surprising that many attempts have been made to use models in much the same way as they are used in other branches of engineering. Two distinct types of physical models must be considered:

a) *Phenomenological Models* which are designed to study general behaviour pattern
b) *Design Models* which are intended to provide quantitative information.

Models which are built up of simple bricks of plaster, cement, wood or any other material to represent a rock mass can provide extremely valuable information on behaviour patterns in such discontinuous systems. Such models have revealed previously unrecognised failure modes or have confirmed hypotheses built up by careful field observation. Note that these models are essentially <u>geometrical models</u> and that no serious attempt is usually made to simulate all the mechanical properties of the rock mass. Model studies of this type (Maury, 1970; Barton, 1970 Krsmanovic, 1967; Goodman, 1972) have proved invaluable as *research tools* and the writers strongly recommend the use of simple models to assist students and design engineers in understanding the basic behaviour patterns in discontinuous rock masses.

On the other hand, models which are intended to provide quantitative design information are not favoured by these general reporters. Even if it were possible to satisfy all the similitude requirements, the amount of time required and the cost of construction a detailed *design model* is such that it is most unlikely that more than one model will be built for any particular problem. Such a model, if well made, may create an illusion of great accuracy and may encourage the designer to accept a single set of result results without considering other failure modes and behaviour patterns. Hence, while value of models as research and educational tools is not questioned, their use as design tools is not recommended since their use defeats the basic object of a good design - to consider all possible combinations of parameters and to arrive at a *balanced judgement*. A "precise" answer based upon an inadequate set of assumptions is of no use to the design-engineer.

2.2 Mathematical models

Two types of mathematical model are relevant to this discussion:

a. Finite element models
b. Dynamic relaxation models

Recent developments in both finite element (Goodman & Dubois, 1972) and dynamic relaxation models (Cundall, 1971) have extended the methods to make it possible to deal with *discontinuous systems* and *simple three-dimensional problems*. Although the mechanical properties of all the elements in a discontinuous rock mass are difficult to represent and although the capacity of present computers limits the size of problem which can be dealt with, the writers are confident that further development of these techniques will provide engineers of the future with very powerful tools. Compared with physical models, these mathematical models will be both cheaper and quicker to operate. Their one outstanding advantage is the possibility, at small additional cost, to vary each of the parameters involved in order to check the sensitivity of the design to these variations.

In spite of general optimism about the development of these tools, there are still serious barriers to their effective use as design methods. These barriers involve the difficulty of supplying adequate input data for a meaningful analysis. Consider a relatively simple stability problem involving a rock mass with three intersecting sets of discontinuities and subjected to water seepage. The input data required for a mathematical model of this problem are:

a) 3 values for friction angles
b) 3 values for cohesion
c) 3 values for hydraulic conductivity

d) 3 values of compression modulus
e) 3 values for shear modulus
f) 3 values for dilatancy coefficient;

a total of 18 variables, each having a range within which its values can be scattered. In addition, the hydraulic boundary conditions (generally very poorly known) have to be defined.

The simple question which must, therefore, be considered is - can input data be obtained for real problems which will permit a meaningful mathematical model to be used for design purposes? The answer, in the case of typical problems encountered by the design engineer, is no. Consequently, the conclusion must be that these mathematical models are extremely useful research tools but must be used with caution if applied to real problems.

2.3 Limit equilibrium methods

The most important failure modes in rock masses which are subjected to low loads (i.e. surface workings) are associated with movement on preexisting discontinuity surfaces (faults, bedding planes, joints etc.). If failure of the intact rock material and deformations within the rock mass are ignored, a simplified mathematical model of the failure process in a rock mass can be constructed. In this model it is assumed that sliding of blocks of material occurs when a condition of *limited equilibrium* is reached, i.e. when the driving forces due to gravity and water pressure are exactly balanced by the resisting forces due to friction and cohesion. Because deformation of the rock mass is not considered, large blocks, which are assumed to remain intact, can be considered and the force system can be simplified to a few total forces acting at specific points on the surface of the blocks. The problem of a wedge of rock resting on three intersecting discontinuities can now be solved on the basis of:

a) 3 values for friction
b) 3 values for cohesion
c) 3 values for forces due to water pressure;

a total of nine variables. As discussed in Section 1.33, a critical structure is normally designed on the basis of zero cohesion and hence this number of variables can be reduced to 6 for such cases.

Graphical and analytical limit equilibrium solutions to a variety of rock stability problems have been published (Wittke, 1965; Londe, 1965; John 1968; Londe et al., 1969, 1970; Hendron et al 1971; Hoek et al.,1973). These methods are the most widely accepted and commonly used *design tools* in surface rock engineering because they are simple and quick to apply and because they permit a rapid assessment of the influence of variations in all the parameters involved in the solution. The graphical methods are particularly useful for *field applications* and can play an important part in the progressive design of site investigations - each step in the investigation being designed to check specific features which the analysis has shown to be important.

This approach has, of course, some limitations. The conditions of limiting equilibrium are assessed without taking the deformations of the rock mass into account. If the rock mass is to act as a foundation, these unknown deformations may be unacceptably large and it is therefore necessary to carry out additional work (Figure 2) to check this deformation behaviour. The assumption that the sliding mass remains intact may also be unrealistic and practical observations suggest that the breaking up of a block of rock during the early stages of sliding will have a significant influence upon the behaviour of a slope. In some cases, improved drainage due to opening up of fractures may be sufficient to stabilise the slope.

Are these limitations serious enough to overcome the advantages of the method? The answer seems to depend upon which point of view is taken. The responsible engineer should be concerned with the detection of factors important in controlling the stability of his particular site rather than with "accurate" computations. Once these factors have been identified, realistic practical decisions can then be taken on the steps which are necessary to ensure that the rock mass will behave in a reasonably predictable manner. On the other hand, the research scientist is concerned with understanding the full picture, hopefully in order than he may be able to evolve better design methods. Consequently, he may feel that the assumptions upon which the limit equilibrium methods are based are unacceptable and that the more comprehensive treatment provided by mathematical models is preferable.

The general reports feel that both points of view are valid and the development of these and other methods is necessary provided that the final aim of designing safe rock structures is kept clearly in mind.

2.4 Mechanical effects of water pressure

2.40 Introduction

A rock mass is seldom dry. Water seeps through fissures as soon as a hydraulic gradient develops, either from rainfall or from water present in a dam or from the creation of an excavation below the water table.

Only the *mechanical effects* of water seepage will be considered here, that is the influence of fissure water pressure upon stability - an influence which is unusually important and is sometimes the governing factor is a slope or foundation design.

In order to determine the pattern of water forces developed by the flow of water in a rock mass, the designer has to know or to make assumptions on the flow conditions. This is an extremely difficult problem.

Firstly, the answer depends upon the geometry of the structural discontinuities in the rock mass and, as pointed out in Section 1.21, this geometry is difficult to ascertain. Secondly, it depends upon the boundary conditions of the hydraulic field (including time - transient or steady state seepage). Thirdly, changes in fissure opening as a result of deformation (some of which are due to the water pressure itself) can significantly influence the hydraulic conductivity of the rock mass. Fourthly, there is a marked scale effect in hydraulic conductivity measurements.

No general solutions which will allow all these conditions to be considered are yet available. There are, however, some simplified models which are very useful to the designer in that they enable him to appreciate the possible influence of water pressures on the stability of rock masses and, also, provide guidance on appropriate corrective actions.

2.41 Forces developed by water seepage

The water flowing in fissures in a rock mass has a hydraulic head at each point and this allows us to extend, to these systems, the concept of *potential gradient* used in the hydraulics of porous media. The forces developed by seepage flow are body forces applied to the intact rock and are proportional to the potential gradient. These forces have to be added to the forces generated by buoyancy.

The *hydraulic conductivity* of a rock mass is governed by the discontinuities which have a much higher "permeability" than the rock material. Because of the inherently anisotropic nature of the rock mass, the hydraulic conductivity is *anisotropic* and the forces due to water pressure have preferred directions. In some cases, these forces are detrimental to stability since they have magnitudes approaching that of other forces (such as weight of the rock mass or the thrust from a structure) and act in unfavourable directions (such as towards the free faces of the rock mass).

The concept of a *conductivity tensor* to represent both magnitude and direction of hydraulic conductivity in a rock mass is an interesting research topic (Maini, 1971) but it cannot be claimed that it is a practical design tool. Consequently, the only approach available to the design engineer is to consider a number of simplified models of possible flow behaviour in order to obtain a qualitative assessment of the influence of the forces developed by water flow in a rock mass. Hence, schematic flow nets which allow for the anistropic nature of the rock can be used to estimate the magnitude of water pressures which can be used in stability analyses (Sharp, Hoek & Brawner, 1972). It is important that the method of stability analysis should allow a wide range of possible forces due to water pressure to be considered in order that the sensitivity of the design to these variations can be assessed (Londe et al., 1969 and 1970).

A disadvantage of using flow nets for assessing water forces is that they assume a static flow situation. In fact, forces due to water pressure may change in magnitude and direction due to deformation of the rock mass and, under some circumstances, the forces due to water pressure may disappear due to increased permeability resulting from deformation while, in other cases where a large supply of water is available from a reservoir, the forces may persist due to the greater flow volumes. Consequently, the concept of *water energy* is probably necessary for a full understanding of the response of a rock mass to water flow. An interesting question for discussion is whether it is possible to introduce this concept into a practical stability analysis.

A considerable amount of attention has been devoted to defining the type of water flow in rock masses - whether it is laminar or turbulent. Research studies have shown that the type of flow has relatively little influence upon the forces which are developed but that the quantity of flow can be significantly different from that predicted by simple models, (Louis, 1970; Sharp, 1971; Jouanna, 1972).

2.42 The planar fissure model

Several authors have shown, by theory or by experiment, that in a rock mass where all the discontinuities are planar and of constant opening from node to node, the modulus of deformation of the rock mass is very low as compared with the modulus of deformation of the rock material. Obviously, in such a system, the opening of the discontinuities will change significantly with applied load.

Applying the laws of hydraulics, linear or otherwise, to this behaviour may produce extremely spectacular changes in hydraulic conductivity for moderate variations in stress (Serafim & Del Campo, 1965; Londe & Sabarly, 1966). These changes could result in the completed engineering structure having a behaviour pattern entirely different from that predicted from site investigations carried out on an unloaded rock mass. The application of this model to engineering design has two important consequences. Firstly, any stability analysis must include extreme water pressure conditions resulting from stress changes and, secondly, the design of remedial measures should take this extreme behaviour into account.

A discussion on the validity of this model would be useful since it has a great practical significance, particularly for foundation design.

2.43 The preferential channel model

Practical observations of the flow of water from discontinuities exposed in adits shows that, in some rock types, water flows through preferential channels which are usually located within the planes of the discontinuities (Sabarly et al., 1970).

Examination of a model where all water seepage occurs through such preferential channels leads to an important conclusion: in this case, drainage will not have a significant influence upon the flow conditions except where a drain happens to intersect a channel. Consequently, drainage will not be effective as a corrective measure for improving stability. This conclusion has very serious implications since drainage is an essential feature in the design of many foundations and slopes.

Whatever one's personal opinion on this model it seems important to answer the following questions:

a) How can the seepage of water which takes place through channels in a rock mass be detected?
b) Can the "permeability" of a rock mass with preferential channels be controlled by grouting?
c) Is it possible to drain such a rock mass, possibly by different drainage systems?

While it is unlikely that an actual rock mass will correspond to either the planar fissure model or to the preferential channel model, these models do represent extreme situations which the designer has to consider as "the most unfavourable mechanical possibilities which could be expected" (Terzaghi, 1929). This is a basic principle of rock design when the safety of a large structure is involved.

2.5 Comments on the use of a Factor of Safety

One of the most controversial questions in rock engineering is concerned with the use of the factor of safety concept. Is the factor of safety of a slope or a foundation meaningful or is it, as some writers have suggested, a totally misleading and useless concept?

The factor of safety for a rock slope may be defined as the ratio of the total force available to resist failure to the total driving force tending to induce failure. In the case of a foundation, the factor of safety may be considered as the ratio of the amount of deformation anticipated as a result of movements within the rock mass to the allowable deformation of the structure.

In the case of a rock structure in which a large number of ill-defined parameters interact in a complex manner, the calculation of safety is a much less satisfactory process.

Should the entire concept be rejected? Are there alternative methods which are more acceptable? One possible approach which has been discussed by several authors is the *probabilistic analysis* of variables leading to a concept of safety in terns of a given probability of failure. This definition of safety is, in itself, a problem since many clients find it extremely difficult to accept an admission by the consulting or design engineer that there is a possibility however small, of failure. A factor of safety of 1.5 or 2.0 may be regarded as acceptable because it represents a familiar situation which experience suggests will be safe while a probability of failure of 1 in 100,000, which may mean precisely the same thing, will be treated with suspicion.

If the probabilistic approach was inherently superior to the factor of safety approach, this problem of definition could be overcome in time since it is basically a question of education. A more serious difficulty, however, is the difficulty of dealing with the large number of variables involved in the problem. Some mathematicians may be confident that these problems can be solved by probabilistic methods, but most engineers are certainly not convinced that these methods are reliable - even if they can understand the mathematical jargon which tends to be used to excess.

These general reporters feel that probabilistic methods have a great deal of merit and that further developments and a greater familiarity with the techniques will eventually result in these methods gaining wider acceptance as practical design tools. The present conclusion, however, is that probabilistic methods are not yet sufficiently developed for general application in rock engineering.

In the absence of acceptable probabilistic methods and as an alternative to the use of a single value for the factor of safety in an engineering design, an approach which is frequently used is to analyse the sensitivity of the design to changes in significant parameters. There are several methods available for doing such a sensitivity analysis and two examples are given below:

a) For the condition of limiting equilibrium, calculate the value of one of the important parameters required to satisfy the conditions being studied for a range of values of the other parameters involved. Hence, the value of cohesion required to satisfy the condition of limiting equilibrium in a slope problem can be calculated for a range of friction angles and groundwater conditions. An example of this type of analysis is given in Section 3.3.

b) By varying each significant parameter in turn while keeping the values of other parameters constant, the sensitivity of the factor of safety to variations in each parameter can be evaluated. The rate of change of factor of safety probably has more significance in engineering design than the value of the factor safety itself because this rate of change is indicative of the importance of each parameter and of whether the behaviour of the structure can be controlled by artificially inducing changes in these parameters.

Graphical presentation of the results of these sensitivity analyses is of the utmost importance since it is only when the variations which have been computed are clearly displayed that they can form the basis of sound engineering decision making. The computer, with its ability to check a large number of variations rapidly and to display the results of these computations in various graphical forms has a very important part in this type of analysis.

The conclusion of this section is that the concept of factor of safety is not easily used in rock engineering but that the rate of change of factor of safety is probably the most reliable indicator of engineering behaviour which is currently available. This subject certainly deserves a wide and open discussion and it may well form the theme for a future Congress.

3. Rock Slopes

3.0 Introduction

This chapter is devoted to the application of rock mechanics to rock slope engineering.

The rock slope engineer is primarily concerned with ensuring that a slope will not fail or that, if failure is allowable, it should occur in a predictable manner. Except when a slope is also to act as a foundation, the deformation of the rock mass into which the slope is cut is of secondary importance.

In contrast to the foundation engineer, who is generally concerned with a specific site of limited extent, the slope engineer may be involved in designing many kilometers of highway cuttings or the overall slopes of an open pit mine. Since neither the time scale nor the economics of such a project allows a detailed investigation of each slope, it is essential that the slope engineer should work to a system which allows him to eliminate stable slopes at a very early stage of his investigations and to concentrate his attention onto those slopes which are critical.

Figure 1 shows that very crude stability analyses should be carried out at an extremely early stage of a project when only the most rudimentary geological data is available. These analyses should permit the engineer to differentiate between those slopes which are obviously stable and those in which some risk of failure exists. They should also be used as an aid to the planning of site investigations to ensure that a maximum amount of relevant information is obtained at minimum cost. More detailed types of analyses, applied only to critical slopes are only justified when detailed information on the structural geology, the groundwater conditions and the mechanical properties of the rock mass is available. Such analyses should permit a consideration of the widest possible range of conditions rather than being confined to the production of a "precise" answer for a particular set of assumptions.

Having established that a given slope is potentially unstable, the designer has then to consider whether its stability can be improved by changes in geometry, by drainage or by reinforcing the rock mass. In some special circumstances, particularly in mining, an economical solution may be to accent the risk of failure and to make provision to predict and to accommodate this failure with the minimum of risk of loss of life or damage to property.

The following topic will be discussed in this chapter:

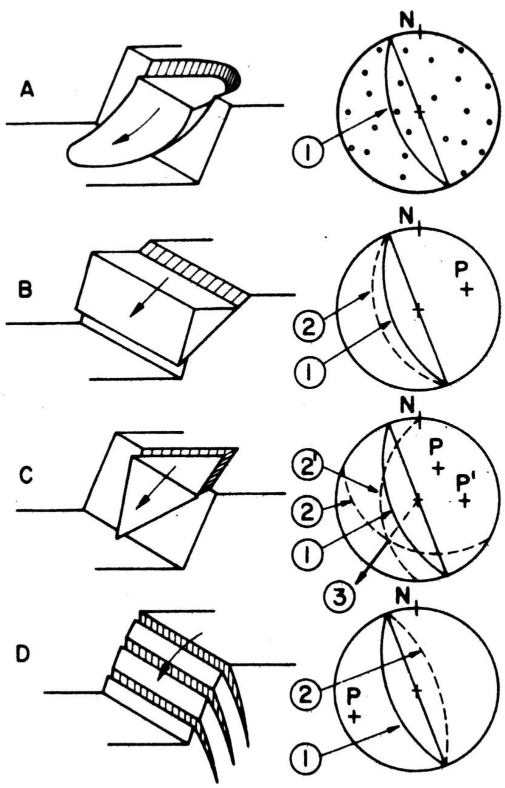
Recognition of slope failure modes
Simple slope design charts
Influence of water pressure on stability
Design of critical slopes
Increasing the stability of slopes
Prediction of slope failure

3.1 Recognition of slope failure modes

The importance of structural geology in controlling the stability of a rock slope has already been emphasised and the first stage in any stability analysis involves a recognition of the most likely failure modes for a particular combination of geological features.

Without doubt, the most effective means of recognising these different failure modes is to examine a graphical presentation of all the relevant structural geology data together with the proposed slope geometry with the aim of detecting patterns which are representative of the different types of failure. A convenient presentation is to use a large topographic map of the site and to plot the geological data on small diameter stereonets (equal-area or equal angle spherical projections) which are pasted onto the map at the observation points (positions of boreholes or outcrops). The proposed slope geometry can then be overlaid on these plots to check the likelihood of different types of failure. Recognition diagrams for *four important types of slope failure* are presented in Figure 20 and, once the designer has become familiar with these diagrams, the recognition of potential failure is relatively simple.

An essential feature of this early consideration of stability is that the designer should attempt to keep an entirely open mind, being prepared to consider all possible types of slope failure, including those which he knows that he will be unable to analyse. The early recognition of a potential failure will allow remedial measures to be carried out at the design and construction stage. Such measures are invariably cheaper and more effective than corrective measures which have to be taken in the case of a slope which is found to be unstable during an advanced stage of the construction.



A Circular failure in soil, waste rock or heavily fractured rock with no identifiable structural pattern
B Plane failure in highly ordered structure, e.g. slate
C Wedge failure involving sliding along the line of intersection of two planes
D Toppling failure in hard rock with steeply dipping discontinuities

(l) Slope face
(2) (2') Discontinuity planes P, P' Poles of planes
(3) Direction of sliding.

Figure 20. Recognition diagram for different types of slope failure.
(Lower hemisphere equal area projection).

It must, however, be made clear that not all potential slope failures can be recognised before construction commences since critical geological features may not be exposed or may have been missed during preliminary site investigations. The designer must, therefore make provision for both time and finances to deal with *unexpected problems* which may arise during construction. He should also ensure that facilities are available at short notice for the implementation of any remedial measures which may be required.

3.2 Simple slope design charts

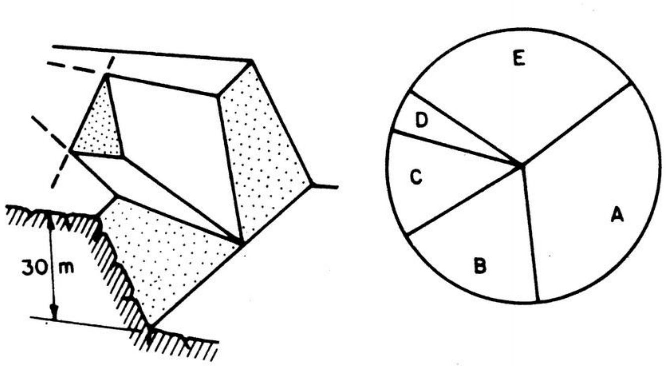
Ever since Taylor published his simple slope design charts in 1937 (Taylor, 1948), soil engineers have made use of these and of more elaborate charts for the preliminary analysis of circular failure in soil slopes. These charts have proved to be invaluable aids to the designer in that they permit a rapid assessment of stability under conditions where a detailed analysis would not be justified. Can such charts be used for rock slopes in which failure is controlled by pre-existing discontinuities?

Since it is only possible to graph a limited number of variables, the first step in producing a meaningful rock slope design chart is to consider whether there are a few variables which are so important that, by setting all other variables to zero and considering only these few, a reasonable approximation to the answer can be obtained. In the case of a rock slope design, this process can best be illustrated by means of a practical example.

Figure 21 illustrates the geometry of a rock slope containing a wedge separated from the rock mass by three intersecting discontinuities - one tension crack and two planes on which sliding can occur. This type of problem has been analysed by Hoek, Bray & Boyd (1973) and the proportions of the forces acting on a tyipcal wedge are given in the pie-chart in Figure 21.

Note that the two items, A (frictional resistance on the sliding surfaces) and E (component of the weight of the wedge acting down the line of intersection) contribute 63% of the total of all the forces acting on the wedge. Both of these items depend upon the geometry of the wedge and it can be shown that only six variables (the dips and dip directions and the angles of friction of the two planes on which sliding takes place) are necessary completely to define A and E (Hoek, 1973). A set of simple charts have been prepared by combining these variables into groups and these charts may be used to improve upon the assessment of stability provided by the recognition diagrams illustrated in Figure 20. An example of the use of these charts is presented in Figure 22.

Note that the factor of safety derived from these charts is independent of the height or the angle of the slope face. This is because the only strength parameters involved in the calculation are the friction angles which, as pointed out in Section 1.33, are independent of the dimensions of the sample. Although these calculations are based upon a very much simplified set of assumptions, and do not therefore provide absolute values, they have made possible the production of a very useful design index for rock slope engineers. An interesting question is - are there other simple relationships of this type which could be utilised in deriving simple design charts for other modes?



Resisting forces:	A	Friction on sliding planes	33%
	B	Resistance due to cohesion	19%
Disturbing forces:	C	Uplift forces due to water pressure on sliding planes	13%
	D	Force due to water pressure in tension crack	5%
	E	Component of wedge weight acting down line of intersection	30%

Figure 21. Contribution of different forces to the stability of a wedge separated from the rock mass by three intersecting discontinuities.

3.3 Influence of water pressure

Figure 21 shows that, for the example considered, water pressure in the tension crack and on the planes along which sliding occurs contribute 18% of the total of the forces acting on the wedge. For steeper slopes with very deep filled tension cracks, this proportion can rise as high as 50%. A consideration of the influence of water pressure upon the stability of a slope is obviously of major importance but how should this influence be evaluated?

The difficulties of adequately defining the water flow pattern in a rock mass have already been discussed (Section 2.4) and the reader will appreciate that a precise calculation of the influence of water pressure upon slope stability is not possible. However, in view of its importance, the only reasonable approach is to base the calculation upon the worst set of conditions which can be anticipated and to use the results of these calculations as an aid to judging the consequences of probable groundwater conditions in the rock mass under consideration.

An example of such a calculation is presented in Figure 23 in which the shear strength (friction and cohesion) required for limited equilibrium in a 25 meter slope, in which two-dimensional plane failure occurs, is plotted for a number of different assumptions. The dotted line included in this figure surrounds the shear strength values obtained from the back-analysis of a number of slope failures (Figure 17) and this type of composite plot assists the slope designer in judging how important various changes are in relationship to the shear strength available.

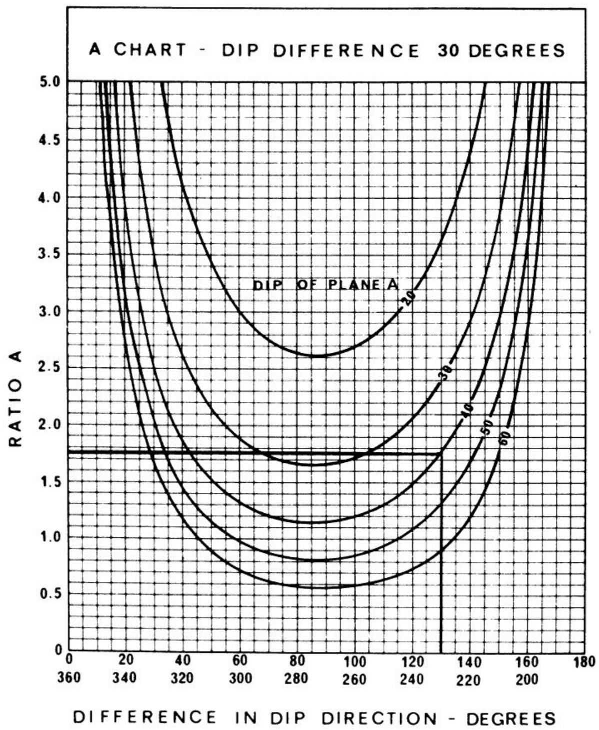
In this example, relatively low shear strength values are required to ensure the stability of a dry slope. Note that the presence of a tension crack (line 2) does not significantly reduce the stability of the slope provided that there is no water present. When the tension crack becomes water-filled under conditions of heavy rain or due to poor control of surface drainage, a significant increase in shear strength is required to maintain stability (line 3). The most severe conditions which could occur in very heavy and prolonged rain which could result in the slope becoming completely saturated (line 4) would almost certainly produce failure in this slope. While the conditions giving rise to line 4 may be very rare, their inclusion in the calculations give a clear indication of the sensitivity of the slave to water pressure. An example of a slope which failed with considerable violence due to the filling of a deep tension crack during heavy raid has been analysed by Roberts and Hoek (Roberts & Hoek, 1973). In this case, the factor of safety of the slope was found to reduce from approximately 1.9 for a dry slope to about 0.8 for a saturated slope. Although these values themselves may not be accurate, their difference and the understanding of the mechanism which leads to this difference is important and this analysis enabled the designers to implement simple drainage measures to prevent the recurrence of these extreme conditions.

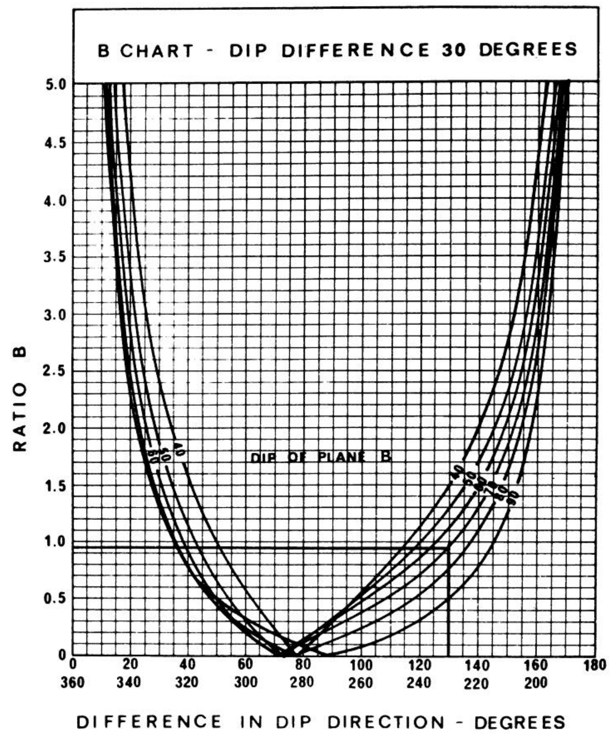
Some indications on the drainage measures which can be applied to a rock slope will be given in a later section of this chapter.

3.4 Design of critical slopes

A large proportion of the total number of slopes which the average engineer will be called upon to design can be dealt with by means of the simple techniques already described. Occasionally, however, a situation may arise in which obvious and inexpensive steps such as minor changes in slope geometry or simple drainage measures cannot be applied. Under these circumstances, the slope designer may be faced with a critical problem in which it is essential that a more detailed evaluation of the stability of the slope and of the effectiveness of more elaborate corrective measures should be undertaken.

The first and most important step in this analysis is the acquisition of reliable data on the structural geology, the mechanical properties of the rock mass and the possible variation in groundwater conditions. Unless such data is obtained, any subsequent calculation will not only be a waste of time but may even be misleading since it may generate a false sense of security in the designer who has been through the calculations but who may have failed to account for some critical factor in the slope. The collection of this data may involve the drilling of additional boreholes, the testing of samples to establish the shear strength of the discontinuities and the carrying out of pumping tests and the installation of piezometers to detect changes in groundwater conditions. Whenever possible, existing slope failures in the same rock types in the area should be carefully studied and an attempt made to deduce the shear strength which was mobilised in these failures, (Natural slope failures may give misleading values because of the very long time scale involved in such failures and back-analysis should therefore be confined to excavated slopes).



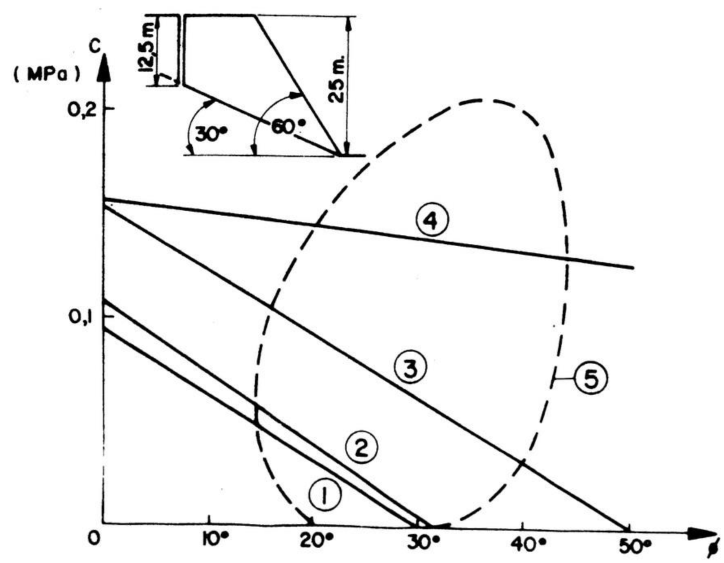


	Dip	Dip direction	Friction angle	*Tan*ϕ
Plane A	40°	105°	20°	0.364
Plane B	70°	235°	30°	0.577
Difference	30°	130°		

From charts: A = 1.75 and B = 0.95,
Factor of Safety $F = A \cdot Tan\phi_A + B \cdot Tan\phi_B = 1.75 \times 0.364 + 0.95 \times 0.577 = 1.18$

Figure 22. Example of determination of factor of safety for dry cohesionless slope with potential wedge failure

Once this data has been obtained, a detailed analysis of the stability of the slope and of the effectiveness of remedial measures can then be carried out by means of techniques which permit the inclusion of all the relevant variables in the analysis. Such techniques have been described by Londe, Vigier & Vormeringer (1970), Hendron, Cording & Aiyer (1971) and Hoek, Bray & Boyd (1973). Because of the complex inter-relationships between the large number of variables involved in these problems, the calculations are generally carried out with the assistance of computers.



1) Dry slope with no tension crack.
2) Dry slope with tension crack.
3) Slope with water-filled tension crack.
4) Slope with water-filled tension crack and water pressure on failure surface.
5) Boundary of zone of observed slope failures (see Figure 17).

Figure 23. Shear strength mobilised for various conditions of two-dimensional plane failure.

It must be emphasised that, in spite of the versatility of these types of calculations, they are still based upon simplified models of the actual failure processes which take place in the slope. The designer should, therefore, beware of falling into the trap of relying too heavily upon the results of such analyses which should be used to assist but not to replace the judgement of the engineer. These analyses show, in the same way as do less elaborate methods the sensitivity of the slope to the various assumptions which have been made.

3.5 Increasing the stability of slopes

There are four basic methods for increasing the stability of rock slopes:

a) Changing the slope geometry
b) Drainage of groundwater in the slope
c) Reinforcement of the rock mass
d) Control of blasting

3.51 Changing the slope geometry

Changing the geometry of the slope generally mean means reducing the slope height or reducing the angle of the slope and, when it is possible to implement this remedial measure, it is generally the cheapest means of improving the stability of the slope. It is however, not always the most effective measure since reducing the height or the angle of the slope not only reduces the driving force tending to induce failure but it also reduces the normal stress and hence the frictional force resisting sliding. Consequently, before implementing this measure, it is essential to check whether it will be effective. As a general rule, very steep slopes can most effectively be stabilised by reducing their height while relatively flat high slopes can be stabilised by reducing the slope angle, provided that the stability is not controlled by major geological structures such as faults.

In addition to the slope height and the slope angle, the geometry of the slope as seen in plan has a significant influence upon stability. Correct alignment of the slope face with respect to the dip directions of the major structural features in the rock mass will reduce the number of these features which will "daylight" in the slope and hence improve the stability of the slope.

Relatively small changes in the position or alignment of the slope face can result in considerable improvements in stability and this should be regarded as both a design and a remedial measure. Whenever possible, the creation of "noses" in slopes should be avoided since slopes which are convex and in which a number of features daylight are inherently less stable than concave slopes where good lateral restraint is provided by the curvature of the face.

One major advantage which changing the slope geometry has over other methods of improving the stability of slopes is that its effects are permanent. This is because the improvement in stability is achieved by a more effective utilisation of the inherent properties of the rock mass and by making permanent changes to the force system in the slope. This force system can also be changed by drainage and by reinforcement but these changes may be reversed if the drains become blocked or if the load carrying capacity of the reinforcement is reduced. Consequently when methods other than changing the slope geometry are used to improve stability, it is essential that these remedial measures be maintained and that a check should he made at least once a year (preferably just before the wettest season) to ensure that these measures are still effective.

3.52 Slope drainage

From the discussion on the influence of water pressure on the stability of slopes (Section 3.3), it will be clear that the presence of groundwater in the rock mass into which a slope has been cut is always detrimental to stability. It follows that drainage of this groundwater will always improve stability but the questions which concern the slope engineer are - how much improvement can be achieved by drainage and how much will it cost?

The simplest and cheapest form of groundwater control is to minimise the amount of water which can collect in pools on the top of the slope. Simple calculations show that water which can enter open tension cracks from the top of the slope is very dangerous since it has the potential for generating high water pressures within the slope. There is no excuse for allowing water to collect on the top of a slope and good engineering practice requires that these areas should be graded to encourage the free run-off of surface water and that surface drains, when they are installed, should be properly maintained to ensure that they remain effective. Where tension cracks are visible in the tops of critical slopes in areas of high rainfall intensity, it is advisable to fill these cracks with porous material such as gravel and then to seal the top of the crack with impervious material such as clay. This will prevent direct entry of surface water, particularly during heavy rain, but will allow water already in the rock mass to drain freely towards the slope face.

Percussion drilled horizontal boreholes can be very effective in draining a rock mass but very few quantitative design guides can be given for the spacing of these holes since their effectiveness depends almost entirely upon whether or not they have intersected water-bearing fissures. In heavily fractured rock, the holes may be regularly spaced since the permeability of the rock mass will be reasonably uniform. In rock masses with widely spaced fissures, the holes should be drilled to intersect those fissures which are believed to be heavily water-bearing. Generally, the holes should be drilled a horizontal distance into the slope approximately equal to the height of the slope. The main advantage of this method of slope drainage is that it is cheap to install and to operate since the water drains under gravity and pumping is not generally required.
Vertical boreholes, drilled from the surface and fitted with down-hole pumps, have the advantage that they can be operative before the slope is excavated and can be used to improve the stability of slopes which are only required to remain stable during a limited period. Permanent drainage by pumped vertical boreholes is expensive and is liable to become ineffective at the most critical times due to pump or power failure.

Drainage galleries, while certainly the most expensive form of drainage, are probably the most effective means of controlling the groundwater in a critical slope. These galleries have the advantage of exposing a large number of fissures within the rock mass through which water can drain freely by gravity. When additional drainage paths are required, these can be created by drilling from within the gallery. While it is difficult to justify the construction of a gallery for drainage only, it is frequently possible to reduce the cost of this measure by careful planning. Hence, an exploration adit can become a drainage gallery at a later stage in a project or existing underground excavations, particularly in mines, can be utilised provided that care is taken to remove the water which accumulates in these excavations.

3.53 Reinforcement of slopes

Improving the stability of rock slopes by artificially reinforcing the rock mass is generally only economically feasible for relatively small slopes or for stabilising blocks of limited size on slopes. This is because the forces which have to be applied by the rockbolts or cables may be as high as 20% of the total weight of the rock which is potentially unstable. The installation of reinforcement is a slope in which instability is already evident is the least effective form of reinforcement since much of the strength of the rock mass will already have been lost due to the opening up of fractures and displacements along discontinuities. On the other hand, if reinforcement is used as part of the design system and is installed during construction of the slope so that dilation of the rock mass is inhibited, the effectiveness of the reinforcement is greatly enhanced. A more detailed discussion on the reinforcement of rock masses is given in Section 4.45 of this report.

3.54 Control of blasting

A final question which must be mentioned in this section is that of the control of blasting during excavation of a slope. While this may not generally be regarded as a means for improving the stability of slopes, there is no doubt that the damage due to blasting has a very significant influence upon stability. Experience suggests that a slope which has been created by carefully controlled blasting may be stable at an angle which is 5 to 10 degrees steeper than a slope which has been subjected to the heavy blasting which is now common in open pit mining. The aim, therefore, should be to minimise the damage to the rock mass which is to form the final slope and this can generally be achieved by the use of presplitting or smooth blasting techniques (Langefors & Kilhstrom1963). The use of these methods is now fairly common in civil engineering but they have not gained wide acceptance in mining because of the relatively high cost of drilling which is involved. Although the actual drilling cost is high, it is believed that the total cost of creating and maintaining a slope by the use of controlled blasting, accounting for the smaller volumes which have to be excavated and the reduction of slope maintenance, will be lower than the cost of an equivalent slope excavated by normal heavy blasting. A comparison of such costs recorded on actual projects would be of great interest to slope engineers and the general reporters suggest that this comparison would form an excellent topic for a short research project.

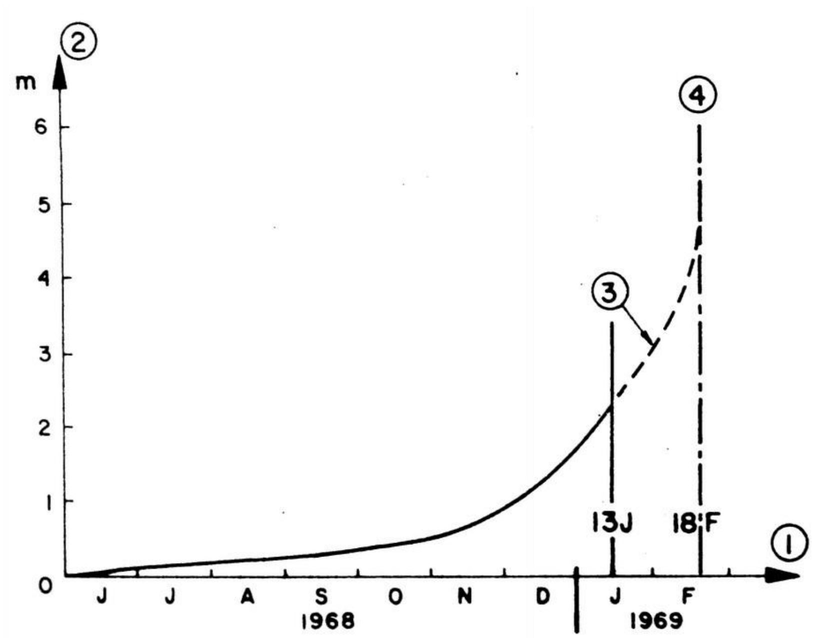
3.6 Prediction of slope failure

When all efforts to stabilise a slope have failed and it is clear that failure is inevitable, it is sometimes necessary to attempt to predict the behaviour of the slope in order that men and equipment may be moved past it before it fails. One of the best known case histories of slope failure prediction is that of the spectacularly accurate prediction of the date on which a very large failure occurred at the Chuquicamata mine in Chile (Kennedy & Niermayer, 1970). Figure 24 shows the plot of surface displacement versus- time on which this prediction was based and this curve is typical of several examples of slope behaviour prior to failure which have been observed.

Generally, the first obvious sign of instability is the formation of one or more tension cracks on the top of the slope. These tension cracks nay occur several years before the failure takes place but model studies (Barton 1971) have shown that these tension cracks are the first manifestation of deep-seated shear movement in the rock mass and that they must be regarded as warnings of instability. Simple measurements of the opening of tension cracks with time can give valuable information on the behaviour of the slope and it will generally be found that the rate of opening increases with time. When the measurements are carried out with sufficient frequency and accuracy, a close correlation between opening of the tension crack and recorded rainfall on the site will frequently be found.

Opening of tension cracks will generally be followed by slumping of the crest of the slope and by bulging of the toe of the slope. Because the movement of the rock mass is controlled by pre-existing discontinuities, these changes may be less obvious than those which occur in soil slopes. Sometimes the movement of an unstable block of rock may be oblique to the face of the slope and it may be difficult to detect these subsequent movements without measurement of a number of points on the surface of the slope

Because of the complexity of the movement pattern which takes place in a rock slope, the installation of sub-surface measuring devices such as boreholes extensometers may not be effective since it may be extremely difficult to interpret the results. It is also usually both dangerous and difficult to install these devices and to keep them effective for the life of the slope.



1) Date.
2) Displacement in meters.
3) Extrapolation of data collected up to 13th January 1969.
4) Predicted failure date.

Figure 24 : Plot of displacement of fastest moving target on the face of the Chuquicamata mine (Chile). The failure, involving approximately 12 million tons of material, occurred on l8th February 1969.

The most successful slope monitoring systems which have been used to date are those based upon simple survey type measurements of the movement of targets placed on the surface of slopes. These measurements may be by normal triangulations or they may utilise one of the electro-optical distance measuring devices which are now commercially available (St. John & Thomas, 1972) The latest developments in stereophotogrammetry are also promising (Ross-Brown and Atkinson, 1972).

No means for quantitative evaluation of the results of such measurements are currently available and, in view of the large number of parameters involved, may never be available, However, experience suggests that the accelerating displacement curve reproduced in Figure 24 is typical of slope failure and that it gives as good an indication as any which is likely to be available in the forseeable future of the likely failure date.

4. Rock Foundations

4.0 Introduction

This chapter is devoted to specific applications of rock mechanics to rock foundation engineering.

The flow chart shown in Figure 2 illustrates the main steps of the appraisal, design, construction and monitoring of the rock foundation of a large engineering structure. This chart, of course, is very crude as compared with the actual approach, which entails much knowledge and judgement and subtle relationships between several fields of engineering, geology, science and craftsmanship.

For the sake of clarity, this chapter has been divided into five categories of problems:

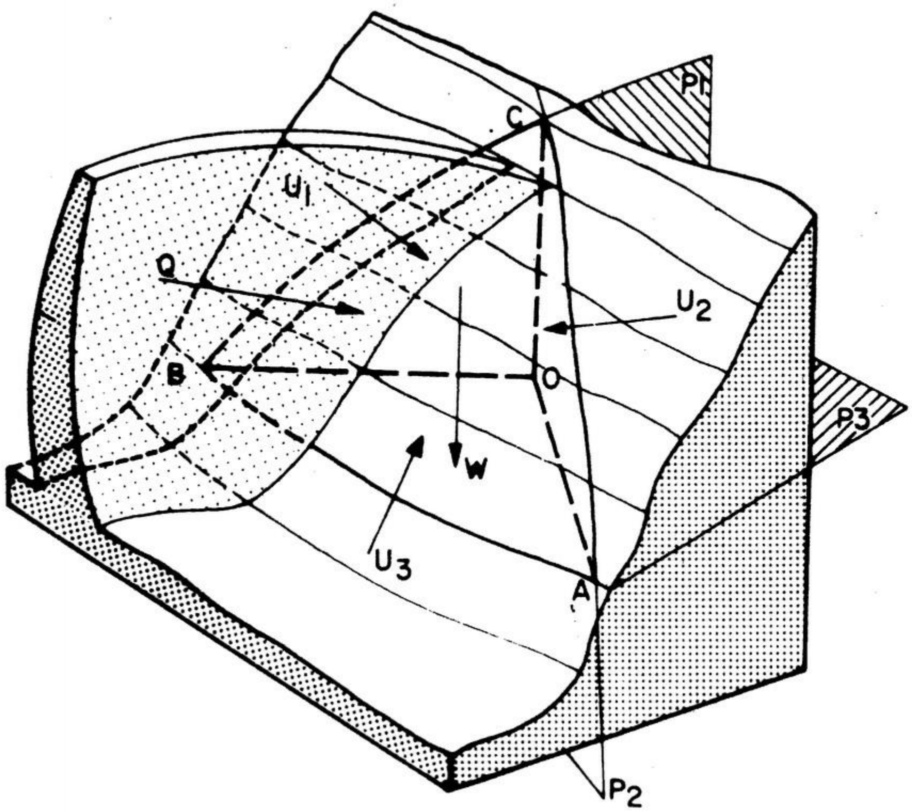
a) Resistance to failure i.e.safety as regards *rupture*
b) *Deformations* and their effects on the stresses in the foundation rock and the structure
c) Mechanical effects of *water seepage* through the fissures of the rock mass
d) *Corrective action* that the designer can take
e) *Monitoring* of the foundation rock

Rock mechanics offers a number of tools to help solve these problems. As already stated in the introduction to this general report, although the design analysis is very crude, owing to our limited knowledge, the rock mechanics approach leads to a correct understanding of the basic or possible behaviour patterns. This is vital for the foundation designer, whose main concern is not to compute accurately but to judge soundly.

4.1 Rupture of rock foundations

The designer's main concern is to prevent failure of the foundations as it represents the worst possible case.

The methods of assessing stability, involving geological appraisal, determination of possible failure modes and analysis of the conditions of limiting equilibrium (Figure 25), have already been dealt with in chapters 1 and 2 of this report. Although it is not necessary to repeat these methods in this chapter, it is necessary to emphasise that the consideration of the resistance to failure of a foundation must always precede the study of foundation deformations, since all subsequent calculations are only relevant when it has been demonstrated, beyond reasonable doubt, that the foundation will not fail.



OABC Tetrahedron.
P_1, P_2, P_3 Geological surfaces of separation.
U_1, U_2, U_3 Water pressure forces.
W Weight.
Q Thrust of dam

Figure 25. Stability of a tetrahedral rock volume (Londe, 1973)

4.2 Deformation of rock foundations

4 .20 Introduction

In considering the deformation of rock foundations, it is necessary to differentiate between deformations within the rock mass and surface displacements. The first category is useful for understanding the intrinsic behaviour of the foundation whereas the second is adequate for analysing the engineering structure built on the rock. The rock mass in comparable, in that respect, to an equivalent continuous medium which has the same surface deflections.

On the other hand, the deformation within the rock mass cannot be determined without considering the actual discontinuous medium, or at least a representative model of it. Because of the contribution of the surfaces of separation, the deformations within a discontinuous rock mass under low confining stresses, will be significantly different from those in a continuous medium. This will be the case for surface workings. In the case of underground workings the confining stresses are higher and, once the discontinuities within the rock mass have been forced into intimate contact, the deformation behaviour will approximate to that of the continuous medium. The theory of elasticity is satisfactory for the analysis of deflections of the equivalent continuous medium, at least as a first approximation, provided that the irreversible part of the first loading cycle (due to closing of the fissures) is considered separately.

Determination of the deformations within a discontinuous rock mass requires the use of mathematical or small scale models.

4.21 Determination of elastic parameters

The equivalent continuous medium can be defined by Young's modulus and Poison's ratio, giving the same displacements at the surface as those of the actual site. Since the deformations within the rock mass are different from those in a continuous medium, this approach is necessarily a rough approximation. In practice, however, it is reasonable to assume an elastic behaviour for most rock foundations, the only restrictions being to use an appropriate elastic modulus for the stress range under consideration. Comparisons between the results of analyses and measurement of foundation deformations on many dams have shown this approximation to be valid. Such comparisons are not generally available for other types of surface structure.

How can the equivalent elastic parameters be determined at the design stage? Jacking tests are almost the only practical means available for such determinations and yet the interpretation of the results of such tests is open to question. In Section 1.27, the influences of scale and of duration of loading upon the results of a jacking test were queried. Even if these questions are ignored, many different moduli can be derived from the non-linear curves obtained from a jacking test. Although it has been proposed that these results are useful identification indices for the rock mass, the question which must now be considered is: can they provide numerical parameters for use in a deformation analysis?

Experience suggests that, provided a large number of tests are carried out in situ, the mean value and the scatter found from such tests allows a reasonable estimate of full scale deformability. It seems likely that, in a rock mass, the small samples are models of larger samples, themselves models of still larger blocks, this series being closely related to what has been called, in Section 1.22, the "grading curve" of the rock mass. If this concept is valid, it would explain why the scale effect does not result in extremely low moduli for very large dimensions and also the fact that scale effect does not appear to have too significant an influence on jacking tests.

For the analysis of the foundation behaviour of the Auburn dam, the U.S. Bureau of Reclamation has worked out a method aimed at reducing the number of in-situ jacking tests required. The principle is to combine the two components of deformability of the mass (a) the modulus measured on cores and (b) the surfaces of separation (spacing, thickness, infilling). This "analytical" method has to be calibrated on each site (Von Thun & Tarbox, 1971).

There are cases where the jacking test has given lower values than the moduli worked out from the overall behaviour of the completed structures. This was the case at the Vouglans dam where plate tests (28 cm diameter) gave an average modulus of 16,000 MPa, whereas the dam loading gave 30,000 MPa (Groupe de Travail, CFGB, 1967). Several explanations are, of course, possible for this "reverse" scale effect: fissures under the jack plate, higher test stresses and, most important perhaps, the fact that the Boussinesq and Vogt solutions used to derive the moduli do not apply to the discontinuous system. The concept of the equivalent continuous medium is therefore possibly responsible for the discrepancy.

A final remark concerns Poisson's ratio, which is assumed, not measured. This concept is probably far from applicable to a rock mass where, not only do the lateral deformations of intact laboratory specimens show wide variations, but the presence of discontinuities will have a significant influence upon the lateral deformation behaviour of the rock mass.

The discussion is therefore open. How shall we measure the deformability of a rock foundation for design purposes? Jacking tests, either on the surface or in boreholes, appear to be the only practical means currently available, even if their results have to be treated with caution. Is it possible to improve the test procedure and our comprehension of the tests?

4.22 Influence of rock deformation on engineering structures

Whereas permissible displacements of rock slopes are usually large, those of a foundation are extremely small, owing to the damage which they can induce in the engineering structure.

This structure is sensitive to two separate effects:

a) The absolute magnitude of the deformations

b) The relative displacements, from one zone to another of the foundation area.

Effect b) is generally more detrimental than effect a). Hundreds of dams have been designed using the Trial Load Method of analysis which required an assumption on the ratio EC/ER between the modulus of the concrete and that of the rock mass. It has been checked that, provided this ratio is nearly constant over the whole foundation area, its influence on the maximum stresses in a high arch dam is slight. For example, a variation from 1 to 5 may result in an alteration of the critical stresses by 20% (some are increased, others are decreased). This means that great accuracy in measuring ER is not required, at least for this part of the design, and that the scale effect is not such a serious drawback to the determination of deformability. In low or rigid structures, the influence of EC/ER is much more marked but, fortunately the stresses are seldom critical in such cases.

On the other hand, however, local variations of EC/ER have a strong influence on stresses in the vicinity. For instance, an arch dam can span a fault zone of several meters in thickness with practically no change in the stress pattern, except locally where special arrangements must be made. Should a zone of softer (or harder) rock be much wider than the thickness of the dam, the problem is more serious. Finally, when a major part of the bank has a modulus different from the remainder, it results in mechanical asymmetry which is much more important on the stress pattern than geometrical asymmetry.

The difficulties met in determining the deformability of a rock mass are therefore, at least partly, offset even for an indeterminate structure by two favourable conditions:

a) It is not necessary to measure the rock modulus with any great accuracy
b) Relative variations, resulting from heterogeneity are probably obtained with adequate accuracy, from small scale tests.

Can these conditions be relied upon? This question certainly warrants further discussion.

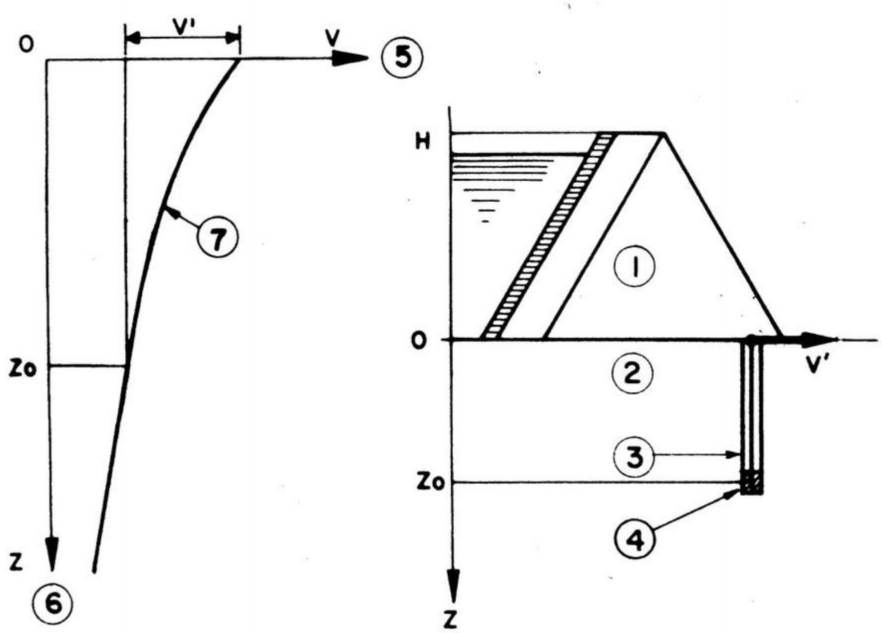
4.23 Irreversible deformations

Jacking tests as well as monitoring of rock foundations have shown that a part of the first deformation is irreversible, especially near the ground surface. This is due to the closing of fissures and to some local minute shear failures. The equivalent modulus for the first loading is therefore lower than the modulus which applies to the further loading cycles. This behaviour is what could be called strain hardening, as in metals, with the difference that in rock it develops from the beginning of loading instead of beyond a threshold. This point is vital for the structure, because the low modulus at the first loading may create the most critical conditions. The problem is all the more serious as this irreversible displacement is extremely difficult to estimate from small jacking tests. Furthermore, it is not the same over the whole foundation area and hence the elastic heterogeneity of the foundation is exaggerated.

A major point is therefore to establish whether the designer can obtain the irreversible part of the foundation displacement from jacking, or other tests. Is a measurement or estimate of the scale effect possible as it would most probably be very large? The stress used in the test has undoubtedly a governing influence on the irreversible deformations, as would be expected in a strain hardening phenomenon.

4.24 Influence of rock deformations on instrumentation

When a fixed reference point is required for geodetic measurements or for anchoring an inverted pendulum, it has to be at a distance increasing with the magnitude of load applied to the rock foundation. Dams with thrusts amounting to millions of kilo newtons and influenced by billions of kilo newtons of water weight, are particularly interesting. What is the distance required to obtain fixed points? This computation is seldom made.



1) Multiple-arch dam. Cross section.
2) Rock foundation (continuous elastic medium).
3) Inverted pendulum.
4) Anchor point at depth Z_o.
5) Horizontal displacement V.
6) Depth.
7) Actual curve V (%).
   V^l Measured displacement.

Figure 26. Example of influence of depth of anchorage of inverted pendulum on measured relative displacement.

It was, however, carried out recently using elastic theory and published in the form of charts for different types of loading (Mladvenovitch, 1970). It is realised that the engineering structure displaces the supporting medium far and deep; much farther and deeper than commonly reckoned. For instance, an inverted pendulum should be anchored at a depth of about 100 meters so as to give a reasonable measurement for the displacement of the base of a dam 100 meters high (Figure 26).

4.25 Deformations within the rock mass

All the preceding comments concerning the surface of the rock mass could, more or less, be dealt with by assuming an equivalent continuous medium. This assumption is not valid inside the mass, where the stress and strain patterns are governed by the discontinuous nature of the medium. For instance, the transmission in depth of a compressive stress field applied at the surface will differ drastically from the continuous solution and close the fissures in much narrower bands and at a greater depth than shown by Boussinesq's equations (Figure 27).

Although several eminent authors have tackled this difficult problem their results do not lead to a convenient tool for the designers, owing to the extreme complexity of the data. The models, both physical and mathematical, are the same as those discussed in paragraphs 2.1 and 2.2. The discussion and comments would also be the same.

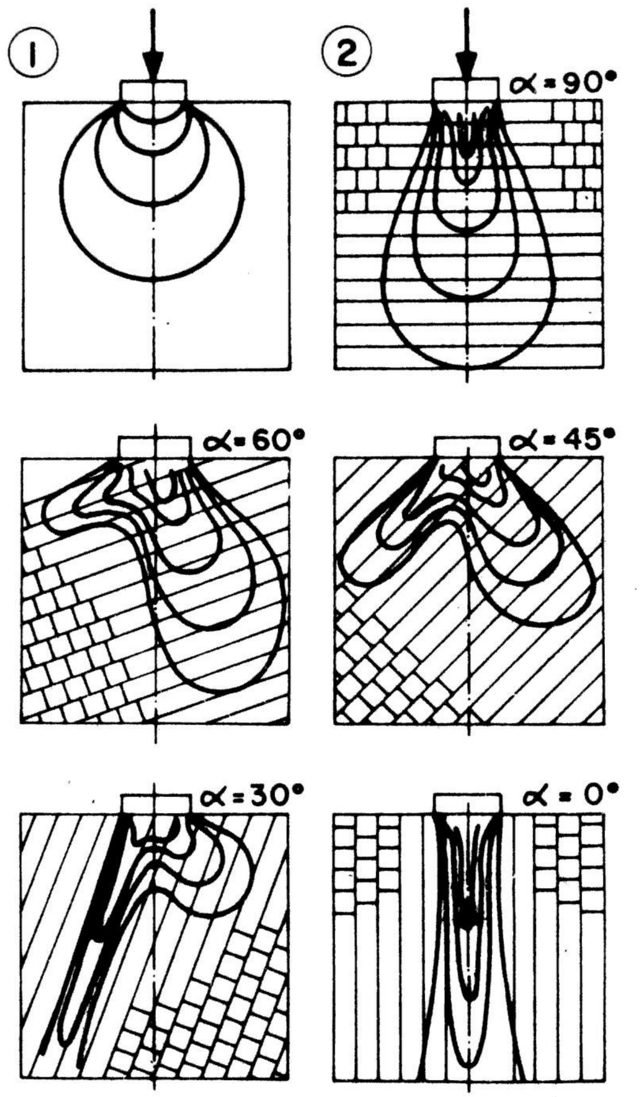
Fortunately the design can generally be carried out on the basis of qualitative reasoning. It is not always necessary to compute to arrive at a sound engineering answer. The main concept to remember is that compressions applied at the surface of fissured rock, act along deep and narrow bands within the rock mass, closing fissures, and that conversely tensions applied at the surface open fissures only in the close vicinity of the applied load.

This effect is of fundamental importance in the hydraulic regime of seepage, and therefore on the resulting water pressures.

More instrumentation is required to investigate these mechanisms which, for the time being, are mostly theoretical. Meanwhile the designers have to make allowance for them in order to avoid the dangerous conditions they could create, should they really fully develop.

4.26 Special case of deep excavations

Some heavy engineering structures, particularly large dams and tall buildings with many basement levels required the excavation of deep cuts into overburden and sometimes the rock below. The applied forces during the excavation process are therefore a system unloading the rock foundation, before re-loading with the structure. The analysis of the foundation rock deformations in this case is extremely difficult, and has very little to do with the results of small scale jacking tests, where the unloading stage cannot be simulated. It would be extremely interesting to discuss the point of how to forecast the behaviour of a rock formation during a cycle of unloading followed by re-loading. The problem is net simple, and includes the considering of pre-existing residual stresses.



1) Continuous medium
2) Series of discontinuous rock masses
(α = angle between bedding planes and load)

Figure 27 : Distribution of stresses in a jointed rock mass of varying bedding dips under an applied external normal load (Gaziev and Erlikhan , 1971)

There has been a case of a sound limestone bed, 10m thick, lying horizontally over a softer formation, in which the application of a load of 2,000,000 kN by a tall building, and, elsewhere, the unloading up to 10,000,000 kN by a deep excavation, resulted in punch-shear failure through the whole thickness of the bed.

4.3 Mechanical effects of water seepage

This topic has been discussed in Section 2.4 and the details will not be repeated here. It must, however, be emphasised that the control of water pressure is of fundamental importance in the design of rock foundations which are required to support large engineering structures. It is the dominant factor in the case of dams.

4.4 Foundation treatment methods

4.40 Introduction

The engineer can improve the properties of a rock foundation by three different categories of corrective action:

Reducing deformations
Increasing strength
Controlling the hydraulic forces

All these means are not equally efficient at a given foundation site. Moreover, their effect is not always clearly understood, owing to the inadequacy of knowledge stij.1 prevailing in several fields of rock mechanics. There is therefore a part of guesswork in many decisions taken about foundation treatment. There are a number of rock foundations where no corrective action whatsoever has been taken, and there are others, like at El Atazar dam, where practically every possible type of corrective action has been taken (Guerrero and Serafim 1970).

The purpose of this section is to make comments and speculate on some usual or less usual methods so as to promote discussion and, with a little luck, improvements of our present techniques. The means of corrective action dealt with are:

Consolidation grouting
Presplitting
Excavation and concreting of joints and faults
Surface strengthening Reinforcement with steel
Curtain grouting and drainage

4.41 Consolidation grouting

It is possible to increase the stiffness of a rock mass by injecting cement grout in the open cracks. This treatment, conventionally applied in the near foundation zone of practically all large structures, has two main effects: the first is to reduce the irreversible part of the deformation, and the other is to increase the modulus of elasticity. This result can be achieved however only if the cracks are open, and if they are groutable.

The first condition is often met near the ground surface, where the lack of confinement leads to a loosening of the blocks (Snow 1968). The opening of the fissures near the surface is clearly indicated by the high hydraulic conductivities generally measured in the upper sections of water tests.

This necessary condition is not, however, sufficient. It is also required that the grout should penetrate the fissures at the moderate pressures permissible near the surface. For cement grout the minimum groutable opening is about 0.2mm. It should be remembered that such an opening corresponds to a high hydraulic conductivity. For instance, 0.2mm cracks at 1 metre spacing give a permeability in their direction of about 50 Lugeon units. The tentative conclusion is that consolidation grouting with cement is probably useless in rock zones where the water tests have given less than say 50 Lugeon units.

In rocks with fine cracks chemical grouts can be used: silica gels or synthetic resins. The resins are restricted to extremely rare cases, owing to their cost (Price & Plaisted, 1970).

Attempts were sometimes made to jet out the soft filling materials before grouting. The process is difficult and requires great skill. Usually, series of holes are used for injecting air and water, with or without chemicals such as bicarbonate, while some other holes act as outlets for the eroded materials. It seems that the high cost and always doubtful results of the operation hinder its development.

The efficiency of consolidation grouting has not often been checked. There are a few cases in literature, mentioning either an increase of modulus measured in jacking tests performed before and after the treatment, or an increase of seismic velocity. But in most cases the question remains: what is the real gain of stiffness obtained? Another point is: how to check the result? This latter aspect is important contractually for the acceptance of the works by the owner.

It seems, however, that the main result is to reduce the deformation heterogeneity over the foundation area, the zones with wide open cracks being equivalent, after treatment, to the other zones. That is probably why the treatment is very generally applied, even if not properly understood.

4.42 Presplitting

Another possible action to reduce the deformability of the foundation is to open the excavation by presplitting. It greatly reduces the tendency for the blocks near the bottom of the excavation to become loose under the action of shock waves. The result is again, a lower irreversible deformation and a higher modulus. The theory of presplitting has been attempted in a continuous medium. The mechanism in a fissured, therefore discontinuous, medium is not well understood and spacing of holes together with their explosive loads are still empirical. Practice has shown the great advantages of the process, widely used at present. Consolidation grouting is still required, as ore splitting does not correct the natural heterogeneity of deformability of the rock mass.

4.43 Excavation and concreting of joints and faults

The presence of major joints or faults in the foundation of a large engineering structure is not a counter-indication although it gives rise to occasional severe difficulties. There are very few dam sites for instance with no major geological feature crossing the foundation area.

When the filling materials are soft, or when there are open voids, the common practice is to fill them with concrete, either by hand or by injection, after the necessary excavations have been done.

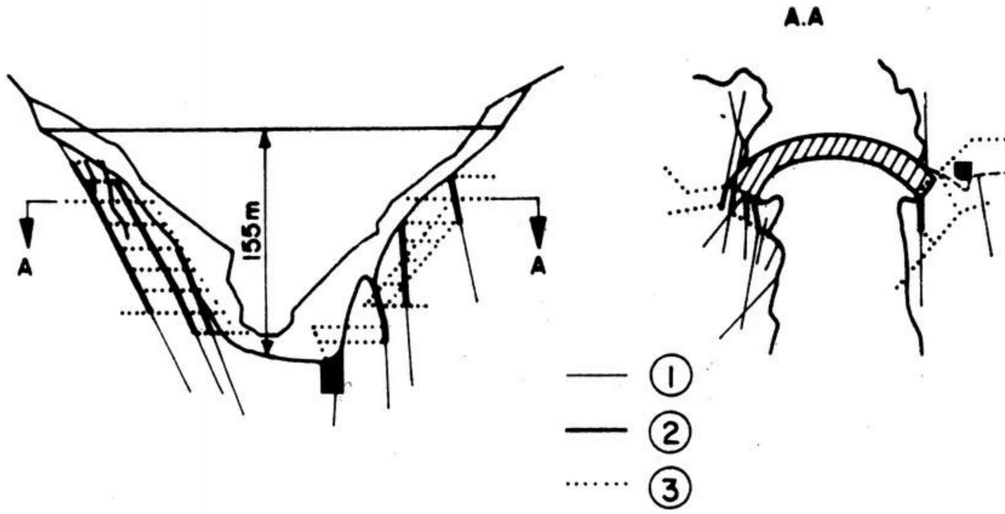
It is not always reckoned necessary to treat the whole surface of the joint. The concreting of a rectangular network of galleries and shafts within the plane of the huge vertical joint called "Julie la Rousse" in the Monteynard arch dam right abutment is an example of successful partial treatment (Faivre D'Arcier & Conte, 1964). The more thorough filling of several faults in Magawado dam abutments (Figure 28) used 20,000 cubic metres of concrete (Fujii, 1970). The excavation of the solid and broken rock material was done with a high pressure water jet (10 MPa) for fear that explosives would shake and disturb the foundation granite. There may be some other applications of this excavation method for "dental work" of that nature.

The problems raised by replacing soft materials in the foundation with concrete are different depending upon the nature of the stresses to be transmitted, but in all cases the most difficult point to answer is: what area of the fault or joint has to be concreted? The answer depends upon the assumptions made on the distribution of stresses within the rock mass, a very doubtful field of rock mechanics.

On the other hand, when considering shear strength the fact that concrete has usually a higher modulus than fissured rock may induce local concentration of higher shear stresses and progressive failure. That is why small concrete key ways, as sometimes contemplated across shear zones, are of doubtful efficiency. This question deserves further study. In any event such corrective action should always be contemplated with a serious monitoring of the treated zone. The foundation treatment of the Carsi highway bridge, France (Figure 29) is composed of concrete shafts, excavated down to a bed that cannot slide into the valley. A difficult question is: what would be the load on the piles in case of movements of a higher bed of the rock formation?

4.44 Surface strengthening

Action at the rock surface is possible by means of concrete buttresses, struts, shotcrete and gunite. Although the forces supported by these elements are slight as compared with the total forces involved in the equilibrium of the rock mass, practical experience has proved that this type of corrective action is often adequate.



(1) Fault.
(2) Concreted excavation in fault.
(3) Adit.

Figure 28. Nagawado Arch-dam concreted faults. (Fujii 1970)

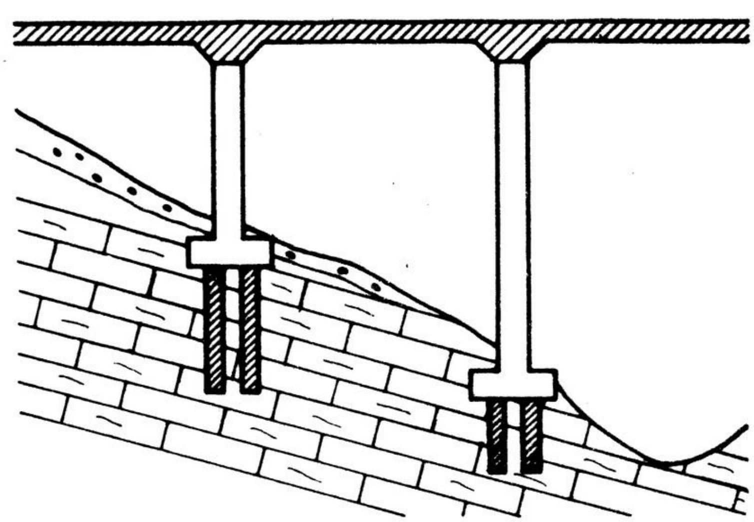
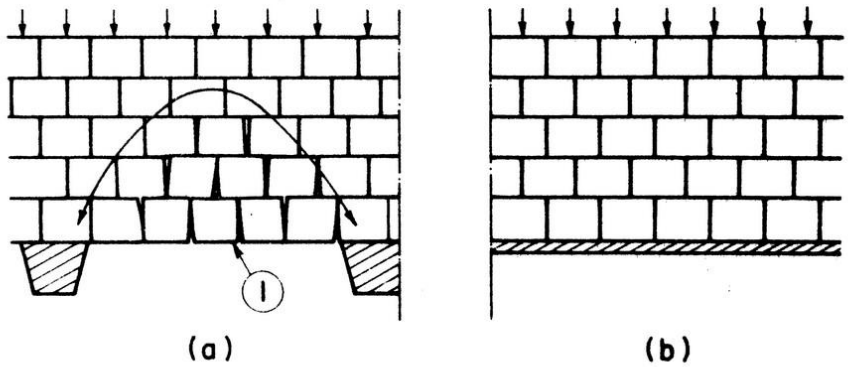


Figure 29. Concreted shafts under Carei bridge piers (France).



a) with butresses.
b) with shotcrete.
(I) Zone of loosening of rock blocks.

Figure 30. Comparison of surface strengthening of a discontinuous rock mass.

The progressive loosening of fissured rock starts at the free surface. Its cause may be either the stress relief due to excavation or the slow alteration of the matrix, or more often, the softening of the materials filling the joints. Slight opening of the joints goes with imperceptible rotations and sliding of rock blocks, large enough, however, to reduce very appreciably the strength and stiffness of the rock mass as a whole, even at depth.

A shotcrete or gunite layer applied immediately after the opening of the excavation obviously provides superficial protection against weathering. As the projected material moulds itself around all the irregularities and penetrates into cracks, even of minute size, it develops a spectacular increase of the stiffness of the "skin" of the rock mass. It is, sometimes, however, argued that the efficiency of shotcrete or gunite is more doubtful on a surface which is not concave, and even, in some places, sharply convex. The role of a reinforcing mesh, even light, is then probably essential. Again, the point is to apply a resisting force across all joints where they daylight at the surface so as to prevent their first displacement from starting. Experience shows that this force can be extremely small. It is interesting to mention that the protective lining is so flexible that it can follow the general displacement of the rock mass without breaking, therefore keeping its full efficiency.

Rigid buttresses or struts look stronger than thin linings and the forces are able to withstand can be computed. Is this the reason why some designers trust them more? It should be remembered, however, that in surface workings as in underground workings, the forces are all the higher as the support is more rigid. The main point in modern techniques is to avoid the progressive deterioration of the compactness of the rock mass, originating always at the free surface. There is therefore a weakness in strengthening by localised rigid units; the surface left unsupported between units is not protected at all (Figure 3D), unless it is covered by a layer of gunite.

The discussion is open on the relative merits of flexible continuous protection and rigid discrete supports. One of the factors to be considered is obviously the deformability of the rock mass proper, the geological structure and also the sequence of the works.

The theory of the mechanism of surface strengthening has yet to be developed. Engineers are unable to put forward a quantitative analysis of the interaction between rock loads and surface protection. They are therefore unable to prove the design arrangements. There are, however, a number of successful applications.

4.45 Reinforcement with steel

Rock is a material with practically no tensile strength and often low shear strength, owing to its numerous surfaces of separation. The idea of reinforcing it with steel bars, as is done for concrete, is therefore very logical. The two principles used in concrete are also used in rock: "passive" steel as in reinforced concrete, "active" steel as in prestressed concrete (Figure 31).

Two main reasons however preclude any complete analogy with reinforced or prestressed concrete:

a) The rock mass is a discontinuous medium with a mechanical behaviour drastically different from that of concrete.
b) The steel cannot generally be installed in rock masses either at the optimum location or at the optimum time.
c)

In fact, the choice between passive and active steel is still open to discussion because the theory has not yet been developed.

The prestressing solution results in a clearer conception of the forces. Each bar or cable is equivalent to a given and well known applied load. It can be introduced into any mathematical or physical model. Of course this applied load has several effects. In the first place it can reduce, by vectorial addition, the effect of other applied loads which are detrimental to stability. In the second place, it can increase the friction resistance of joints by adding a normal compressive stress to the existing stress. It is also possible to introduce a high enough compression to avoid the development of tension, i.e. opening of cracks. One may even claim that prestressing reduces the irreversible part of foundation displacement by closing some open cracks.

The method, however, has some drawbacks. Although the total loads that can be practically applied are still very small as compared with the forces present in the rock mass, the stresses near the anchor zones are high, approaching the compressive or shear strength of the rock. In addition, there is always the threat of failure of high tensile steel wires by stress corrosion, particularly in rocks where the chemical composition of water might be much more unfavourable than in concrete. The process has, however, proved very successful in some cases such as the surface strengthening of the banks at Vajont Dam, or the tightening of treated fault zones in the Nagawado dam abutments.

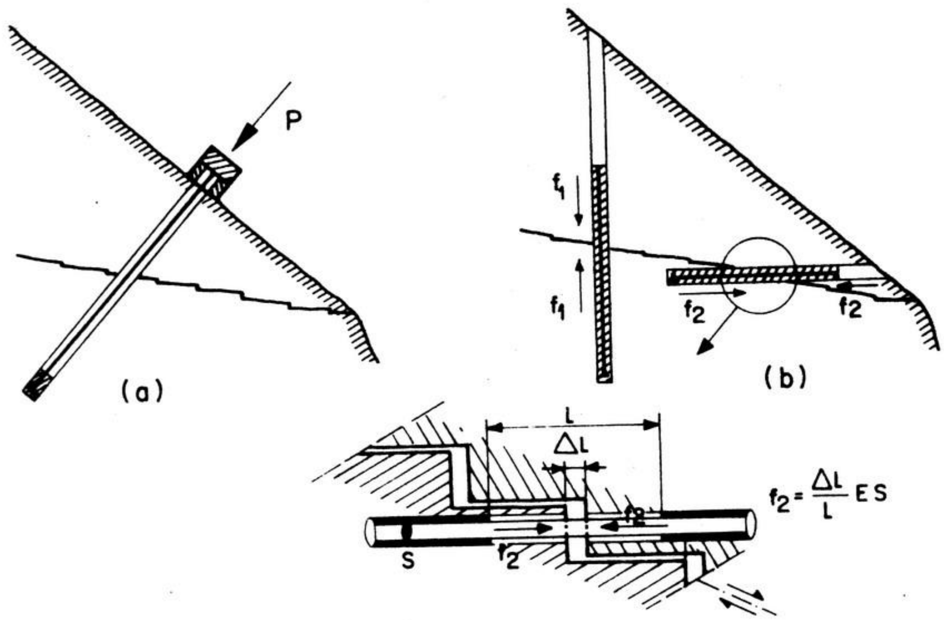
Ordinary reinforcement bars embedded over their whole length have been used more often. Although the; mechanism of their action is more difficult to understand, they have proved successful in many cases. The principal is to introduce into the rock mass additional tensile and shear resistance at the surfaces of separation crossed by the steel. The maximum available force is determined by the steel cross section but the stress actually developed is not known.

It has been argued that passive steel can contribute a stabilising force only after rock has deformed, i.e. after failure. This is true but the rock strain necessary to mobilise the steel reaction is extremely small owing, first, to the fact that all deformation is concentrated within the thickness of the joints and second to the fact that the steel is perfectly embedded in its hole. This point is vital for the proper functioning of the reinforcement. A joint opening of 0.2 mm, for instance, would have to open only 10^{-4} mm more to develop a stress of 100 MPa (permissible stress of mild steel). Should the bond fail over a certain length on both sides, the opening of the joint will remain an extremely small fraction of a millimeter.

The theory of the reinforcement is however not yet available.' It seems that there are two distinct cases to consider.

In the first case, steel is used for stabilising a possible failure by sliding on one or two single, smooth geological features such as bedding planes or faults. The computation can be done assuming that the strain will be limited to a low value due to the reinforcement, and allowing a certain shear strength to develop. With joints exhibiting peak strength, it may be possible to keep a part of this peak strength. The question is: how to compute this available cohesion? There is no answer yet, although one might feel that it could be given by a close examination of the process of progressive failure.

In the second case, steel can be used in an imbricated rock mass, or for stabilising shear sure-faces with some degree of roughness. In this case, shear strain is accompanied by dilatancy. The joint crossed by reinforcement opens up and puts the steel under tension as soon as a shear failure starts. Another way of looking at the mechanism is to consider the intrinsic curve of a rough joint. If irregularities are arranged in a random pattern, their angles vary and are higher for smaller irregularities and one obtains a curve, with a very steep slope near the origin (Figure 14). This means that for low normal stresses the angle of friction is much higher than usually assumed. The consequence is that, with the low normal stress developed in the steel bar by a slight dilatancy of the joint, a relatively high shear strength is available. The effect of the reinforcement is therefore to translate the intrinsic curve as a whole towards the left resulting in an appreciable cohesion. At the same time, the interlocking action of the steel, which, with a moderate force, prevents the smallest irregularities of the joint from slipping, probably results in increased stiffness. The reinforcement might therefore be visualised as a means of improving the modulus of deformation of the rock foundation. The results obtained with mine rock bolts (Leone et al 1971) tend to show that embedded bars are better than free, anchored bars.



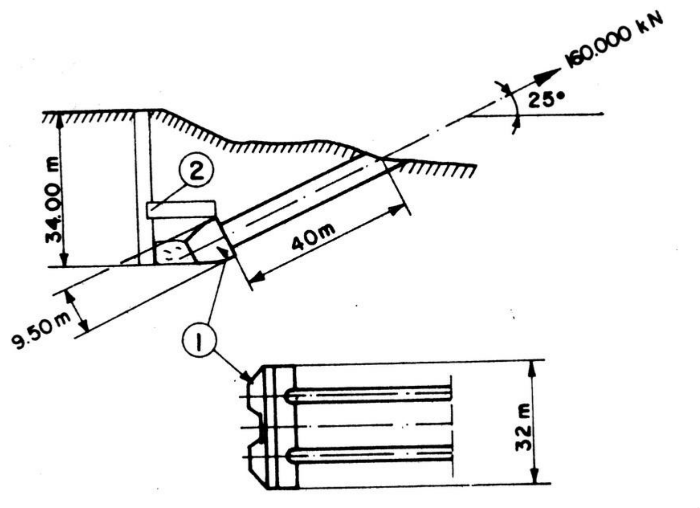
a) Prestressed cable.
b) "Passive" anchor bars.

Figure 31. Reinforcement with steel.

A difficult problem is raised by the anchoring of main cables of large suspension bridges, which apply shear and tensile loads of high magnitude to the rock mass. At Tancarville bridge, France (Figure 32), the load was 160,000 kilo newtons. It was anchored in chalk. All possible failure surfaces were investigated, in terms of the structural data, and grouting, together with placement of "passive" reinforcing bars was done.

In the cases where millions of newtons are required, the use of prestressing would lead to serious problems of stress concentration, whereas ordinary reinforcement, in adits backfilled with concrete, is a cheap and straightforward operation.

The extension of this concept plus a better understanding of the mechanical behaviour would lead to a new material for the design of foundations: reinforced rock. It is fully realised that much has to be studied yet and that several statements in the foregoing are controversial, but it is considered that the prospects are promising enough to stimulate at least a lively discussion.



a) Concreted key excavated in rock.
b) Shafts and adits.

Figure 32. Anchorage of Tancarville suspension bridge, Right bank (Esquillan 196l).

4.46 Curtain grouting and drainage

When the decision is made to act on seepage forces the two main tools are grouting and drainage. They are complementary, although sometimes only one is used. In the past, say before 1960, most of the rock foundations were only grouted when water seepage was an obvious nuisance (particularly to reduce loss of water from reservoirs). It is interesting to note that, until recently, rock foundations of dams had deep grout curtains, and only gravity dams had drainage curtains, usually very shallow and practically limited to the rock-concrete contact. After the effect of water seepage on the stability of foundations was better understood, that is practically after the Malpasset Dam abutment failure, drainage was considered the best action in most cases. It is at any rate the only efficient treatment in rock of low hydraulic conductivity, such as all rock with fine fissures.

In paragraph 2.4, comments were made on the basic understanding we-have acquired at present, of water seepage in fissured rock and of forces that it develops. Even if this knowledge is still qualitative, it is adequate for directing the engineer's work. As it often happens in foundation engineering the main point is not to have a perfect technique of analysis but rather a sound understanding of the possible mechanisms. There is, however, a limitation: what is the effectiveness of any corrective measure? The danger is believing that the action is efficient while in fact it may not be. Grout curtains, with one or several lines of holes, aim at plugging the water paths by grout. An ideal grout curtain would support the whole water pressure on one face, no water remaining on the other side. Unfortunately, there are several reasons which prevent grout curtains from acting in this perfect way. First the limitations given in paragraph 4.41 are still valid: cement grout does not penetrate thin fissures, and does not remove sandy fillings. The use of chemicals and the jetting out of fissures is generally too expensive for the purpose. Even more as the efficiency of a thin curtain is extremely sensitive to a minor and local defect. This point, strongly made by Professor. A. Casagrande at the First Rankine Lecture - 1961, was then questioned by several authors but is now commonly accepted.

Fortunately in finely fissured rocks, where a grout curtain is not valid, drainage is a suitable alternative. It fully controls the hydraulic potential on the downstream side. In other words it achieves exactly what was required from the grout curtain, the only difference being that the drainage increases the amount of leakage, whilst the grouting reduces it. This is without any consequence in most rocks where the hydraulic conductivity is low. Conversely, if the conductivity is high grouting has to be carried out, should it be only as a consolidation treatment. To summarise, it can be stated that, for fissured rocks:

a) of low permeability (say less than 5 Lugeon units), drainage is generally essential, whereas grouting is useless,
b) of high permeability (say more than 50 Lugeon units), grouting is required for controlling water leakage whereas drainage is not necessary.
c) for medium permeability, drainage is always useful,its cost is low, and the decision on whether to carry out a grout curtain can be made on the basis of economics (permissible water loss or cost of pumping leakage).

The theory of change of conductivity of rock under stress, as discussed in paragraph 2.42, leads to other considerations (Ter-Minassian et al, 1967) which have a particular significance in dam foundations, but may also have to be considered in other cases. The fact that the stresses applied by the engineering- structure act at depth and night render the rock extremely tight if it is finely fissured makes the limitations of the grout curtain mentioned above still more pronounced. It also helps to locate the drains in a zone of the foundation where they are not "masked" by the watertight barrier due to stresses. In the case of arch dams they should be directed in an upstream direction.

This theory, although checked in the few cases where the foundation rock was adequately instrumented for the purpose, is still controversial. It would be extremely useful to the profession to know of cases where the behaviour of water seepage has confirmed or invalidated this model. On the other hand, it is likely that in the future, more drainage tests than grouting tests will be carried out at the design stage. This would be a normal trend as "drainability" might be a vital part of the design of a large structure foundation (Pena et al, 1970).

Another important point, made in paragraph 2.43 is whether the drainage can be effective in a rock formation where water flows through preferential channels. As a single line of drains gives no protection in this case, it may be necessary to contemplate a uniform distribution of drains through the whole rock mass. This mechanism has to be studied in more detail. It is a vital subject of investigation, because a number of foundations protected with a conventional drainage curtain are perhaps not drained at all. Of course many rocks probably have neither the ideal plane fissure flow type nor the equally ideal preferential channel flow type. A number of recent observations however have shown that the preferential channel flow is frequent and the governing factor for the efficiency of drainage is the proportion of flow drained as soon as channels are present the ratio between water discharge via channels and discharge via fissures is very high. The result is that a drain which does not intersect a channel does not significantly alter the flow net and the corresponding pressures.

For all the previous reasons, the effect of drainage, often vital for the stability of the foundation, should be monitored. Piezometers are therefore considered as an integral part of the drainage design. They would also detect the ageing of the system, as it is well known that drains have to be maintained against clogging by fine grains of soil or chemical deposits. Only piezometers can give warning in time that drains have to be reamed out or new drains have to be drilled.

4.5 Monitoring of rock foundations

4.50 Introduction

It was realised, rather recently, by civil engineers that instrumentation and monitoring of the foundations of major works was a vital part of design. However, before 1960 hardly any rock foundations were monitored. It is generally considered at present that monitoring of the foundations is at least as important as monitoring of structure. The French word for instrumentation is "auscultation" from the medical term meaning an investigation through specific sounds or noises. As in the medical field, it is not necessary to assume that the patient is ill before practising "auscultation". As a matter of fact, the role of instrumentation, as a medicine, is twofold: research into the normal behaviour and early detection of any significant divergence from it. The information obtained is all the more valuable, the earlier readings are started. When possible, instrumentation should be installed before the structure is built.

Finally, instruments left within the rock mass should be robust and the reading operations should be simple since the conditions on a site are far different from those in a laboratory. The methods discussed here do not cover all available instrumentation, but are reckoned to be the most reliable and suitable for rock foundations.

4.51 Geodetic measurements

Two types of measurements, based on geodesy, are commonly performed: displacements in directions x,y, z by triangulation, and displacements in the vertical direction z only, by levelling. The sensitivity of levelling is ten times more (0.1mm at 50 m distance) than that of triangulation.

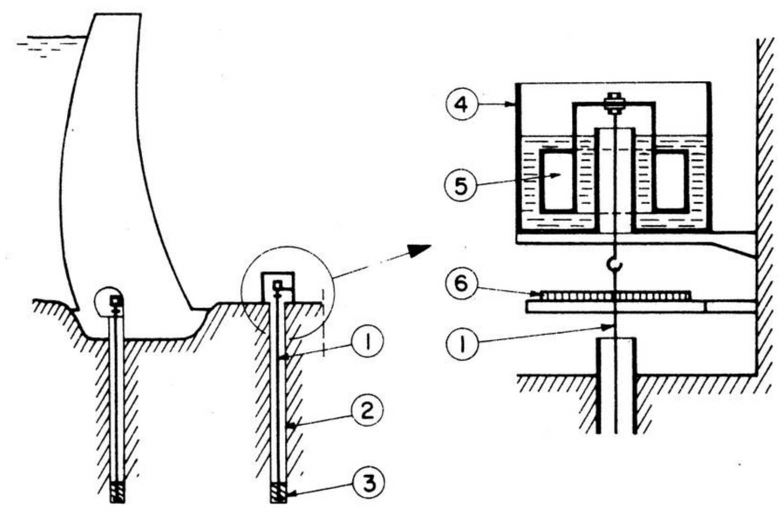
Recent developments in electro-optical distance measured devices (Penman, 1971; Thomas & St. John, 1972) have added an important factor of rapid and accurate measurement of distance to geodetic measurements. A combination of these new methods and traditional optical survey methods provides the engineer with a powerful set of monitoring techniques.

The main drawback in all these methods, however, is the possibility, which has often been observed, of unstable reference points. The small displacements to be measured in a rock foundation may be exceeded by errors from reference base movements. The latter movements may come from elastic deformations of the ground under applied loads and also from erratic displacements of the surface layers where the monuments are founded.

It is therefore suggested that geodetic measurements should not be relied upon for the detection of the small displacements which are associated with the normal behaviour of an engineering structure. They can, however, provide a means of detecting large displacements which are indicative of abnormal behaviour.

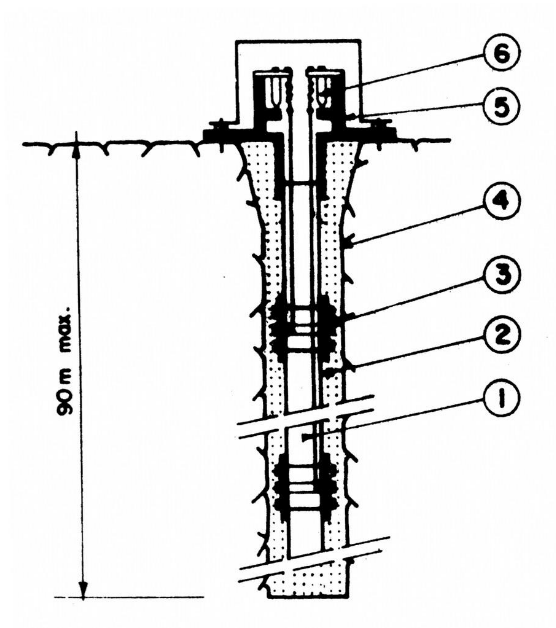
4.52 Inverted pendulums

In foundation rocks, pendulums are usually of the inverted type: the wire is anchored at the bottom of a shaft and kept in a vertical position by a ring float at the upper part (Figure 33). Normal pendulums can be used however when placed between adits like in Monteynard dam abutments. The sensitivity is about 0.05 mm in x and y directions.



1) Stainless steel wire.
2) Large diamterer borehole.
3) Anchor zone.
4) Ring-shaped tank.
5) Ring-shaped float.
6) Reading scale.

Figure 33. Inverted pendulum in foundation rock (Electricite de France).



1) Stainless steel wire (8 wires).
2) Watertight PVC casing (50mm diameter).
3) Anchor ring for wire no. 1.
4) Borehole (75 mm diameter).
5) Measuring head.
6) Vibrating wire device.

Figure 34. Borehole "Elongameter". (Manufactured by TELEMAC)

Inverted pendulums are probably the most accurate instruments that one can place in a rock foundation. They give a very reliable value of the horizontal displacement vector provided the fixed point is really fixed. What should therefore be the depth of the shaft? That is a question still to be answered. It depends of course on the loads applied to the foundation, but also on the geological structure of this foundation. Recent computations (Mladyenovitch, 1970) have shown that most of the pendulums now in operation are not anchored deep enough to give a good approximation of the absolute displacement (see section 4.24). In spite of this drawback they would however detect very early deviation from normal behaviour. A mention should be made also of the difficulty of drilling deep, straight vertical holes to be sure that the wire does not come into contact with the walls at any level.

4.53 Wires in boreholes

The relative displacement along the pendulum wire itself could be measured, but in practice, special wires, not necessarily vertical, are used for this purpose. The main difficulty is to eliminate length variations due to stress and temperature. Invar wires have to be used for years to calibrate the geodetic base lengths and the technology is the same, except that the wires are installed in adits or boreholes. The systems have been used for many years in mines but the development of the method is recent for rock foundations. It is now common to install eight lengths in the same hole (Figure 34).

The main difficulties for the installation of this valuable device are:

a) Drilling straight holes, particularly when they are long and near the horizontal, and executed from a narrow adit.
b) Avoiding possible friction along the walls by an adequate tension.
c) Avoiding creep of the wire due to too high a tension.
d) Anchoring correctly the different wires of different lengths

According to the local conditions and length of wire the sensitivity varies: it is approximately 0.l mm for a range of 5cm.

Although a definite improvement, this multiple wire device is still discontinuous; the fissures cannot be localised exactly within each section between anchor points. The ideal would be a long extensometer, able to measure the strain over its whole length (Bernaix, 1969).

4.54 Clinometers

Two types of clinometers are used in rock foundations: fixed instruments and sliding cells lowered into boreholes.

The first type is extremely accurate. The vibrating wire clinometers for instance give a sensitivity of 1 to 5 x 10^{-6} radians, and they give the direction in x and y of the variation of slope. A number of them are installed in adits, shafts or underground chambers, to detect any possible anomaly of deformation.

The second type, using boreholes lined with a plastic casing equipped with guiding groves is very commonly in soil. Its lower accuracy is due to the imperfection of the guides, deformation of the hole, and inaccuracy in positioning the cell at the same place for each series of readings. It is therefore not a good device for measuring the real deformation of a rock mass by integration of elementary slope variations. It is however useful to detect any possible shear zone or surface along the borehole. The only drawback is that beyond a certain shear strain the measuring cell will jam in the hole and not give any further information on the section below.

The chain deflectometer (Muller & Muller, 1970) is an instrument of intermediate type; although removable from the borehole for repair or calibration, it is left homed-in for several series of measurements.

4.55 Geophysics

Monitoring by geophysics has been attempted several times. Although not often applied, these methods are perhaps worth developing, and it would be interesting to gather the experiences, positive or negative, obtained on rock foundations. It is tempting to use geophysical methods as they act somewhat like radiology in medicine: they "look" inside a large body of rock.

Electric conductivity could bring valuable information on changes of permeability, but above all on alteraction or dissolution of rock owing to the resulting change in ion content.

Seismic refraction or transmission between fixed points could detect a possible change in fissure openings, in other words, in stresses. The investigations made in the foundation of Gage 2 Dam in France (Faurox et al, 1968) are encouraging. It would be interesting to know; whether other experiments have been attempted, and what are the most significant seismic parameters: velocity, length, attenuation of waves? which waves? For instance, at Gage 2 dam, the variations between empty reservoir and full reservoir conditions, were 20% for the wave velocity and 90% for attenuation of energy.

Finally microseismic recording by highly sensitive seismographs of minute shocks originated in the foundation may detect either a normal adaption to the new stress field, or the onset of failure. This method of micro-seismic measurement, mainly used in monitoring rock slopes, could probably be used in foundations as well, provided it is interpreted with great skill; otherwise there might be needless concern at quite normal developments.

4.56 Piezometers and drains

The drains, which are usually installed in foundations, at least when the hydraulic gradients could develop forces detrimental to stability (e.g. in dam foundations), are not only efficient corrective measures (see 4.46) but also useful monitoring instruments. The increase in discharge, or the drying out, of a drain obviously has a meaning. However, no interpretation is possible without the second term of the flow net, i.e. the hydraulic potential. That is why all designers now agree on the absolute need for piezometric measurements together with drain discharge readings. The whole is what has been called in French "auscultation hydraulique". It seems that it is a powerful means of detection of any rearrangement of strains in the rock foundation. As a slight deformation of the rock mass entails a much larger deformation of the fissures, which in turn result in spectacular changes in hydraulic conductivity, it is claimed that the slightest modification of rock strains should react on the flow net, i.e. on the piezometer readings and drain flow rates.

Although this behaviour has been observed in a few cases, it is of utmost importance to gather further confirmation, because it would give a powerful means of warning, probably before any anomaly is detectable by other instruments.

It should be remembered, however, that the theory of sensitivity of flow net to fissure width variation does not hold when the flow is concentrated along channel-like paths. This point, discussed in 2.4 has to be considered seriously for the interpretation of "hydraulic instrumentation."

A mention should be made here about the reliability of piezometric measurements

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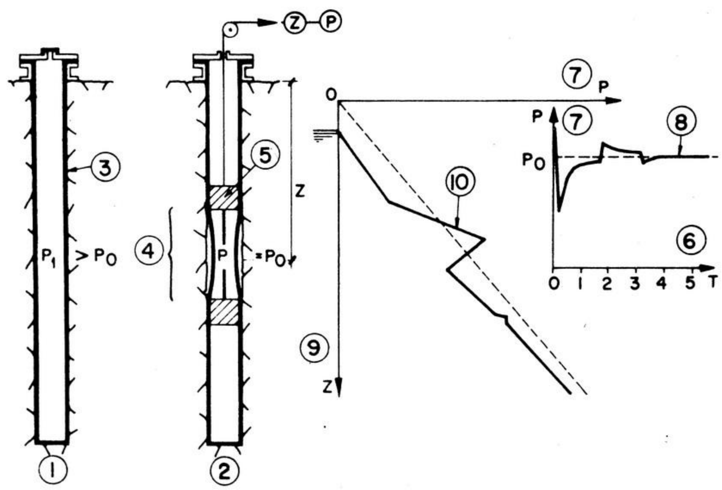
The piezometer tips are either too short, giving only a local value difficult to use, or too long, giving a wrong "mean" value by permitting circulation of water between levels at different potentials. The continuous borehole piezometer (Figure 35) worked out in France (Groupe de Travail CFGB, 1970) is an important step towards proper piezometric readings: a complete log of pressure is possible for the whole length of the borehole, which, in addition, does not allow circulation of water, thanks to a rubber membrane.

Finally, drain holes could also he used to perform Lugeon tests, at constant locations during the operation of the structure (Louis, 1971). Tests of this nature are not often done, although they deserve to be, to enable a better understanding of foundation rock behaviour.



1) Borehole with rubber membrane under pressure (P** Pq)'
2) Borehole during measurement at depth Z (P = Pq^"
3) Rubber membrane.
4) Measuring probe.
5) Packer.
6) Time in minutes.
7) Water pressure inside probe.
8) Curve of pressure vs. time at depth Z.
9) Depth.
10) "log" of water pressure.

Figure 35. Continuous borehole piezometer (Groupe de Travail du CFGB , 1970)

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