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Draft for Development

Recommendations for ground anchorages

Projet à développer
Recommendations relatives aux ancrages au sol

Entwurf zur weiteren Ausarbeitung
Empfehlungen für Verankerungen im Boden

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This publication is not to be regarded as a British Standard.

It is being issued in the Draft for Development series of publications and is of a provisional nature because of incomplete knowledge. It should be applied on this provisional basis, so that information and experience of its practical application may be obtained.

A review of this Draft for Development will be carried out not later than two years after its publication. Notification of the start of the review period, with a request for the submission of comments from users of this Draft for

Development will be made in an announcement in the appropriate issue of *BSI News*. Observations which it is felt should receive attention before the official call for comments will be welcomed.

According to the replies received, the responsible BSI Committee will judge whether the Draft for Development can be converted into a British Standard or what other action should be taken.

Foreword

This Draft for Development has been prepared under the direction of the Civil Engineering and Building Structures Standards Committee.

There has been a dramatic increase in the use of ground anchorages in Britain during the past 10 years, due primarily to the successful application of retaining wall tie-backs in continental Europe. Not only has the number of anchorage installations increased but the range of applications has widened considerably, ranging from dam stressing and reinforcement of underground excavations in rock to tying back earth retaining structures and holding down tower and bridge foundations. For these applications, anchorages can be employed to solve problems involving direct tension, sliding, overturning, dynamic loading and ground prestressing, which in turn demand a variety of design and testing requirements.

Although ground anchorage technology is still in an active stage of development, the use of anchorages is widespread for both temporary and permanent applications and there is clear evidence that a variety of design and testing concepts exists in current practice, some erring perhaps on the unsafe side in relation to topics such as safety factors, overall stability analyses, acceptance criteria and corrosion protection. Furthermore, in the absence of a British Standard covering ground anchorages, certain foreign standards (see appendix A) have been introduced on UK contracts and these standards have also increased the variety of acceptance criteria employed, some of which have evolved from fundamental concepts different to those developed in the UK.

This Draft for Development was prepared as a result of this, together with special evidence submitted in 1976 by Panel 4 of committee CSB/2 Earth retaining structures.

In recent years there have been changes in design philosophy in the construction industry and also in the standard system of units of measurement. As a consequence, this Draft for Development introduces limit state concepts in appendix B, but clauses in the main text retain recommendations on traditional safety factors, since more limit state design experience is required before optimum partial safety factors can be determined. SI units are used throughout.

Because of its provisional nature, BSI requests manufacturers not to mark products with claims of compliance with this publication. Since such claims would not have the effect of indicating conformity with a definitive national standard, they could be judged misleading trade descriptions.

BSI Committee CSB/22, whose constitution is listed at the end of this Draft for Development, takes collective responsibility for its preparation under the authority of the Board of BSI. However, the constitution of the Committee was unusual in that it included an exceptionally large number of coopted members; the personal contribution of the following is particularly acknowledged:

Dr G S Littlejohn	Mr J R Miller
Mr P A G Andrews	Mr J M Mitchell
Mr A D Barley	Mr P I Parker
Dr D A Greenwood	Prof D G Price
Mr J T C Harvey	Dr J C Sharp
Mr M Hubbard	Mr K W J Treadaway
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Mr J May	

Draft for Development

Recommendations for ground anchorages

0. Introduction

In recent years, new applications for ground anchorage systems have continued to appear regularly and today anchorages may be associated with retaining walls, dry docks, cofferdams, storm-water tanks, concrete gravity dams, tall buildings, suspension or arch bridges, tension roofs, pile and plate loading tests, transmission towers, radio masts, ski jumps, cliff stabilization, open mine pits, shafts, tunnels, underground caverns, pipelines and oil platforms.

Equally significant but perhaps more striking have been the developments in anchorage construction and descriptive terms such as multi-underream, gravel placement, lost point, straight shaft, compression tube, end plate, rotating plate, multi-helix, inflatable membrane, expandable wedge or plug, continuous auger, tube à manchette, resin cartridge and resin injection indicate the number of techniques now available.

It is considered that if reliable performances are to be maintained, a technical appraisal of anchorage systems is required by the practising engineer, in addition to the routine comparisons on the basis of cost and duration of the contract. There is a need for a detailed knowledge of the ground, which may demand a supplementary investigation and a proper design related to static and dynamic loads, location of anchorages, load transfer lengths and overall stability. Bearing in mind the current trend towards the use of limit state design, account has to be taken of loads and accompanying deformations under service conditions, as well as the deformation mechanisms developed at failure. For temporary or permanent works, the corrosion protection applied to the anchorage has to be adequate for the circumstances.

With regard to anchorage construction, the importance of skilled operatives cannot be over-emphasized, since the quality of workmanship greatly influences subsequent performance. This workmanship factor limits the ability to predict anchorage performance accurately solely on the basis of empirical rules and ground investigation data.

Quality controls and record keeping are therefore strongly recommended during the construction stage and further, each anchorage, once installed, should be subjected to an initial proof load greater than the required working load followed by a check on the residual load after a short period of service. In this way, the safety and satisfactory performance of each anchorage should be ensured.

1. Scope

This Draft for Development gives recommendations for soil and rock anchorage systems of the grouted or mechanical type and whilst emphasis is placed on post-tensioned anchorages to reflect current trends and usage in the construction industry, many of the recommendations apply to passive anchorages.

The subject of rock bolts will be covered in greater detail when the draft is reviewed, by which time the Construction Industry Research and Information Association (CIRIA) report 'Rock reinforcement' should be available. A useful introduction to the practice of rock bolting in mining and tunnelling is provided by Schach et al (1979).

Tension piles and deadmen are excluded from this Draft for Development, since these subjects are covered by CP 2004* and Civil Engineering Code of Practice No 2 (1951)*†, respectively.

In relation to the ground anchorage systems covered, this Draft for Development gives recommendations for site and ground investigation requirements, design methods and philosophy, corrosion hazards and protective measures, construction techniques and quality controls, stressing procedures, testing of anchorage components and complete installations, acceptance criteria, maintenance during service and legal aspects. In addition, comments on potential health and safety hazards have been incorporated throughout.

This Draft for Development attempts to deal adequately with all important and relevant aspects of the subject but where there are shortcomings in our present knowledge and no firm guidance can be given on a particular section, this situation is highlighted in the hope that it will be drawn to the attention of research institutions (see appendix C).

A bibliography, containing details of both the references in the text and other relevant publications, is given in appendix P.

This Draft for Development has been written for qualified civil engineers.

NOTE 1. Two review publications by Littlejohn & Bruce (1977) and Hanna (1980) are recommended for study.

NOTE 2. The titles of the publications referred to in this Draft for Development are listed on the inside back cover.

*Under revision.

†Available from the Institution of Structural Engineers, 11 Upper Belgrave St., London, SW1.

2. Definitions

For the purposes of this Draft for Development the following definitions apply.

2.1 ground anchorage. An installation that is capable of transmitting an applied tensile load to a load bearing stratum. The installation consists basically of an anchor head, free anchor length, fixed anchor length and a tendon (see figure 1).

2.2 temporary anchorage. An installation that is often used during the construction phase of a project to withstand forces for a known short period of time, usually less than 2 years.

NOTE. If aggressive corrosion action is likely during service, special protective measures may be required.

2.3 permanent anchorage. An installation that is required to ensure the stability and satisfactory service performance of the permanent structure being supported. The service life is greater than 2 years and the tendon throughout its length is protected against corrosion.

2.4 rock bolt. A specific form of ground anchorage where a bar of steel or other appropriate material is fixed in rock.

2.5 resin anchorage. A rock bolt installation where resin, generally polyester, is used to bond the bar to the rock, particularly in rapid excavation situations.

2.6 resin end type anchorage. A resin anchorage that has only the fixed anchor zone bonded to permit subsequent stressing.

2.7 full column single speed resin type anchorage. A resin anchorage that has the bar fully bonded and is not suitable for subsequent stressing.

2.8 two-speed resin type anchorage. A resin anchorage that utilizes fast and slow setting resin capsules that allow the bar to be stressed against the fast setting fixed anchor zone and subsequently fully bonded by the slow setting resin filling the remainder of the hole.

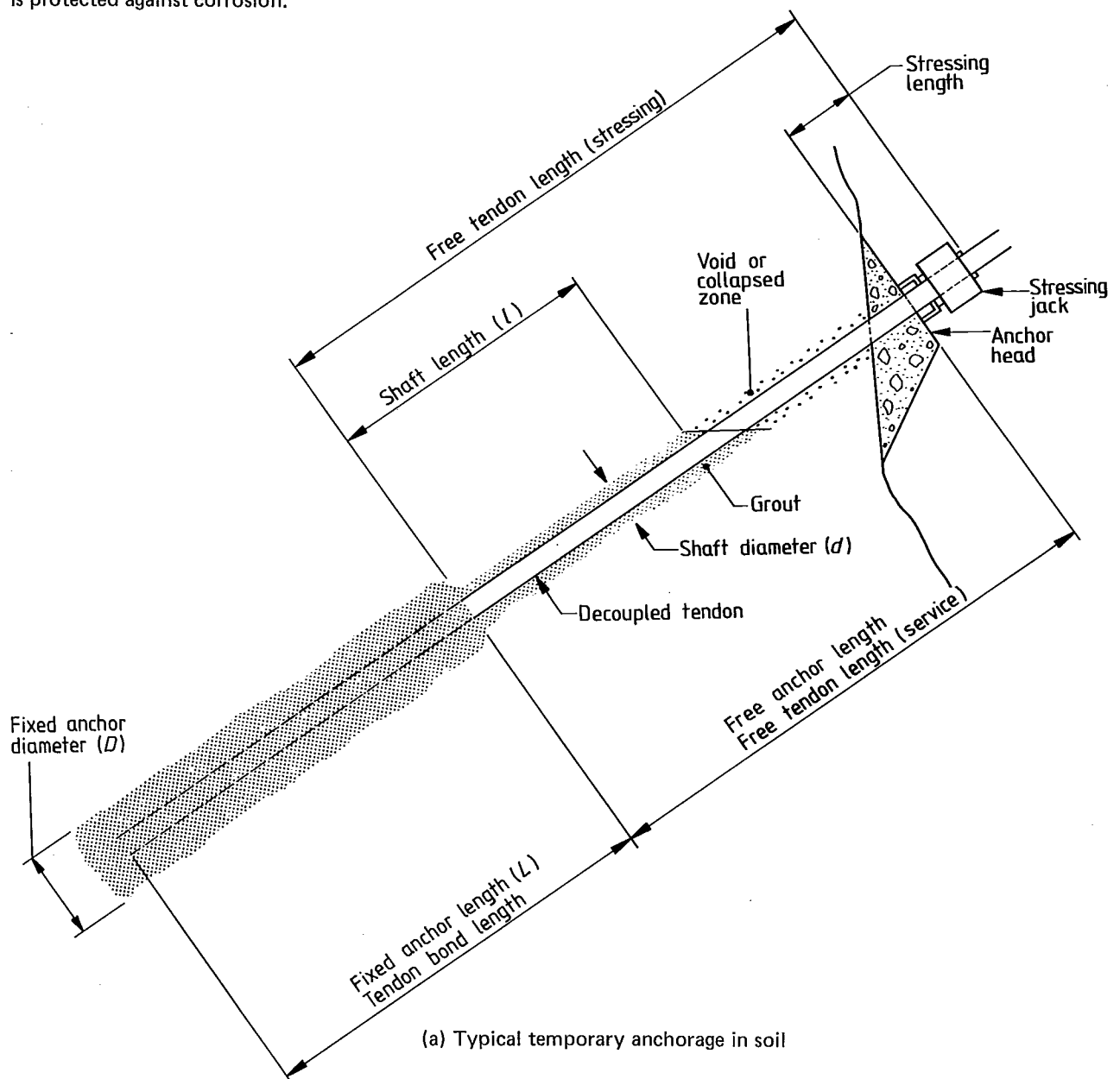
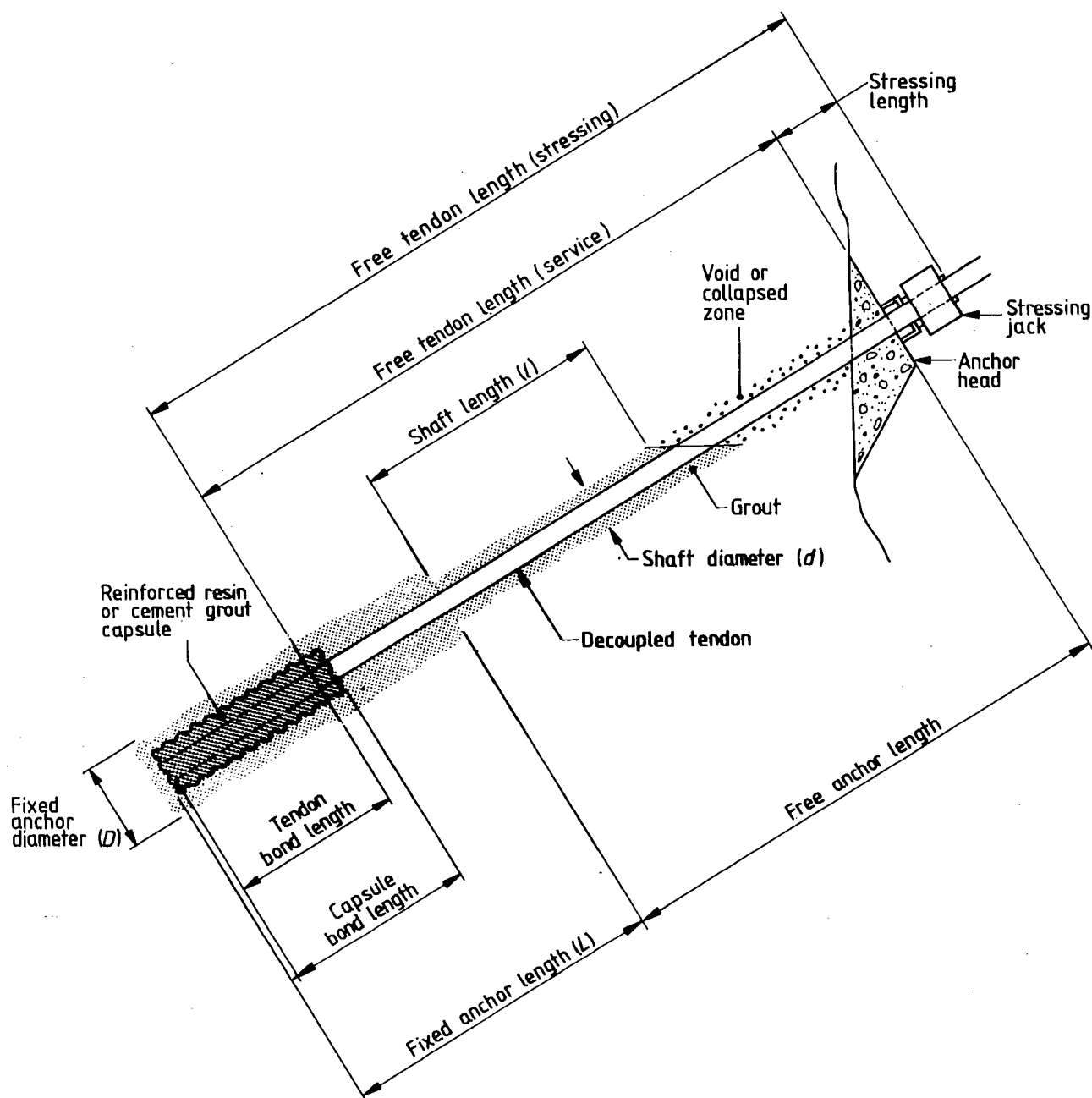


Figure 1. Ground anchorage nomenclature



(b) Typical permanent anchorage in soil

Figure 1. (continued)

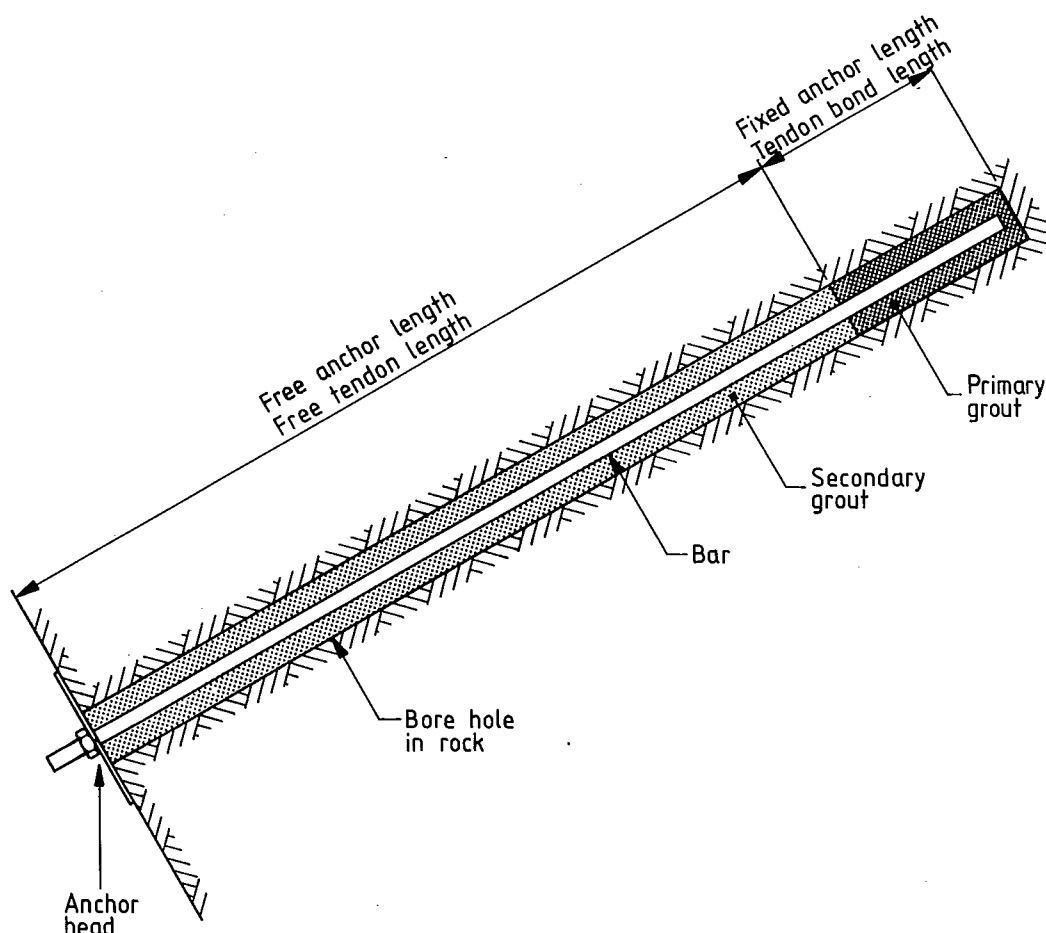
2.9 primary reinforcement. A term used for rock anchorages installed in underground excavations to maintain overall stability of the excavation.

2.10 secondary reinforcement. A term used for rock anchorages (generally low capacity rock bolts) installed in underground excavations to overcome localized problems of instability, e.g. spalling.

2.11 anchor head. The component of a ground anchorage that is capable of transmitting the tensile load from the tendon to the surface of the ground or structure requiring support.

2.12 normal type anchor head. An anchor head that is designed to permit the load in the tendon to be raised or lowered within the limits $0\% f_{pu}$ to $80\% f_{pu}$ (see 2.41) and measured, when necessary to comply with the requirements of acceptance testing. This facility is available during the initial stressing phase and subsequently, if the tendon is shortened, further measurement or adjustment is not possible.

2.13 restressable type anchor head. An anchor head that has all the properties of the normal head (see 2.12) and in addition permits the tendon, throughout the life of the structure, to be measured by check lifting and small losses, up to 10 % of working load, to be recovered by shimming or thread-turning.



(c) Typical rock bolt

Figure 1. (concluded)

2.14 detensionable type anchor head. An anchor head that has all the properties of the restressable head (see 2.13) and in addition permits the tendon to be detensioned in a controlled way at any time during the life of the structure.

2.15 stressing head. The component of the anchor head assembly that attaches the tendon to the anchor head, e.g. nut for a rock bolt or steel plate with tapered holes to accommodate wedges in the case of a wire or strand tendon.

2.16 fixed anchor length. The designed length of the anchorage over which the tensile load is transmitted to the surrounding ground.

2.17 free anchor length. The distance between the proximal end of the fixed anchor and the anchor head.

2.18 proximal. Situated nearest to the anchor head.

2.19 distal. Situated furthest from the anchor head.

2.20 shaft length. The length of grout filled hole in front of the proximal end of the fixed anchor. The grout is often placed prior to stressing and therefore is capable of mobilizing resistance to withdrawal.

2.21 tendon. The part of an anchorage that is capable of transmitting the tensile load from the fixed anchor to the anchor head.

2.22 tendon bond length. The length of tendon that is bonded directly to the grout and capable of transmitting the applied tensile load.

2.23 capsule bond length. The overall length of the outer perimeter of the capsule, which is bonded to the surrounding grout in the case of a doubly protected permanent anchorage.

2.24 free tendon length. The length of the tendon that is decoupled from the surrounding ground or grout during stressing.

NOTE. During the initial stressing operation, the free tendon length may be extended by the stressing length, depending on the type of stressing system employed (see figure 1).

2.25 primary grout. The grout placed or injected before or after tendon homing and prior to stressing, to secure the fixed anchor to the surrounding ground.

2.26 secondary grout. The grout normally injected after stressing to bond and/or protect the free length of the tendon.

NOTE. With the development of restressable anchorages where the free length is decoupled, e.g. greased and sheathed, the complete injection may be carried out in a single stage using primary grout.

2.27 double protection. Protection consisting of at least two physical barriers to corrosion.

2.28 proof load. The maximum test load to which the anchorage is subjected during the initial stressing phase.

2.29 lock-off load. The load transferred to the anchor head immediately on completion of a stressing operation.

2.30 lift-off load. The minimum load monitored during a restressing operation that permits a locking nut to turn on a bar tendon or provides a clearance or lift (see 8.6.3.2) in the case of a wire or strand tendon.

2.31 residual load. The load remaining in the anchorage at any time during service.

2.32 relaxation. The decrease of stress with time while the tendon is held under constant strain.

2.33 creep. The change in strain of the tendon with time under constant stress.

2.34 adhesion bond. The initial bond before slip that arises mainly from the physical interlocking, i.e. gluing, of the microscopically rough steel and the surrounding grout. Molecular attraction is also thought to act. Adhesion bond is considered to disappear when slip comparable with the size of the micro-indentations on the steel occurs.

2.35 friction bond. The bond that depends on the confining pressure, the surface characteristics of the steel and the amount of slip but is largely independent of the magnitude of the tendon stress. The phenomena of dilatancy and wedge action also contribute to this frictional resistance as radial strains are mobilized where the longitudinal strain changes.

2.36 mechanical interlock. The bond similar to micro-mechanical locking but on a much larger scale, as the shear strength of the grout is mobilized against major tendon irregularities, e.g. ribs or twists.

2.37 skin friction. A term commonly used in practice to define the ultimate bond value at the ground/grout interface of the fixed anchor.

2.38 debonding. The breakdown of adhesion bond and friction bond at the grout/tendon or ground/grout interfaces.

2.39 decoupling. The separation of components when tendons are separated from the surrounding ground by grease and sheathing. The term also relates to the breakdown into subdivisions of the grouted fixed anchor or capsule due to the presence of spacers or centralizers that inhibit the flow of grout during the injection process or create artificial boundaries, thereby producing a discontinuous medium.

2.40 slenderness ratio. The depth of the top of the fixed anchor divided by the diameter of the fixed anchor for downward inclined anchorages.

2.41 characteristic strength. The value of cube strength of grout or concrete (f_{cu}) or the ultimate load of a prestressing tendon (f_{pu}), below which not more than 5 % of the test results fall.

2.42 absolute accuracy. The deviation from the true value, i.e. where the measuring instruments have been calibrated against dead weight apparatus or loading machines and the accuracy is known.

2.43 relative accuracy. The deviation from the measured value, i.e. the error in measurement where small changes in load or displacement are monitored against time.

2.44 pitting. The condition where the rate of corrosion into a metal from the surface is significantly greater than the rate of attack parallel to the metal surface. A width/depth ratio of one is commonly recognized for a pit (see J.2.3).

3. Site investigation

3.1 General. A ground anchorage system may be required:

- (a) as an integral part of the design concept of a project;
- (b) to resolve a problem revealed by a preliminary or general ground investigation for the overall project;
- (c) as a remedial or improvement measure to an existing structure.

The information on ground conditions available at the stage of considering the feasibility of a ground anchorage system will therefore depend on the initial requirement for the installation. Whilst there may be adequate data to indicate both the feasibility and advantages of an anchorage system, there may well be insufficient detailed information to permit its safe and economic design or construction. The geometry of a ground anchorage and its mode of operation requires, in particular, a detailed knowledge of ground conditions local to the fixed anchor. Hence, minor variations in ground conditions can assume a greater degree of importance than, for example, in foundation design.

The majority of techniques available in ground investigation are fully described in BS 5930 for site investigations.

It is now generally accepted that investigations are most satisfactorily undertaken in a number of stages (see 4.1 of BS 5930 : 1981). These can be summarized broadly as:

- (a) initial desk and field study;
- (b) main field and laboratory investigation;
- (c) investigation during construction.

The point of entry into such a programme of investigation will clearly depend upon the data available at the time of considering the feasibility of a ground anchorage system.

Similarly, the work required in any one stage will be dependent on the nature of the project. For example, the consideration of a rock bolt system for a limited depth rock cut might be based predominantly on a visual field study, whilst that for a major retaining wall could require that the main field and laboratory investigation stage be split into preliminary and detailed phases.

The adoption of a stage programme of investigation also provides significant advantages where the constraints on the project are predominantly those of economy rather than time. Each stage can be undertaken after due consideration to the available data and following a clear commitment to proceed further with the study.

It should be noted that, with only a few exceptions, the data required for the safe design of temporary works is similar to that necessary for an installation that forms part of the permanent structure.

The layout of this clause follows the general sequence of a staged programme of investigation relevant to anchorage design and construction. Additional detail on some facets of the work is given in appendix D.

The methods of ground investigation, together with the associated field and laboratory testing (see 3.3, 3.4 and 3.5) cover some of the available techniques that are particularly suited to anchorage design. It is necessary for the designer to select those techniques that are applicable to both the geology of the site and the type of project under study.

3.2 Initial desk and field study (see 4.2 and 4.3 of BS 5930 : 1981). It is sometimes found that a preceding investigation that, for example, determined the need for ground anchorages as a solution to an engineering problem, included an adequate desk and field study. However, it should be noted that the varying qualities of investigations imply an inherent doubt that adequate initial studies have been undertaken in all cases or that they have looked into such details of ground conditions as might be useful for anchorage design. It is prudent to repeat these initial exercises. A number of aspects that should be considered are outlined in appendix D.

Prior to the start of anchorage work, an all party approved report on the condition of the surrounding land, including buildings, streets and underground services, where appropriate, should be drawn up.

3.3 Ground investigation

3.3.1 Extent and intensity of investigation (see clause 10 of BS 5930 : 1981). The extent and intensity of the ground investigation should relate to the character of the ground conditions in relation to the nature of the proposed engineering work. By its very geometry, the behaviour of an anchorage is extremely sensitive to local ground conditions. The effect of an unexpected change of ground in the fixed anchor zone can be serious in terms of time and cost.

The aim of the investigation is therefore to determine, by the most economic means, the nature of the block of ground that is influenced by, or influences, the installation and behaviour of ground anchorages. Because inclined ground anchorages are installed as commonly as vertical anchorages, lateral variations in ground properties should be investigated as thoroughly as the more easily investigated vertical variations. Since investigation methods are often vertical, it may follow that an inclined anchorage system requires rather more boreholes than is usual for other structures, particularly in soils.

Where the fixed anchor length lies beneath an existing structure, the drilling of inclined boreholes may be possible. Whilst such a technique is within standard practice for rocks, practical difficulties generally preclude the boring of inclined investigation boreholes in soil. In this case, it may be necessary to employ supplementary investigation ahead of anchorage construction during the contract period (see 3.8.1).

The number of locations investigated by boreholes, probes or in situ tests and the depths to which the investigation has to extend, will be dependent on the type of project, the plan dimensions of the site, where this is defined, so as

to include the probable fixed anchor zone and the data available from previous investigations or from the initial desk and field study (see 3.2). Whilst these variables preclude general rules, it is to be recommended that a sequential investigation is made under the continuous supervision of a geotechnical engineer or engineering geologist.

Thus, for example, on a site for which little previous data are available, a number of key boreholes should be sunk to identify the geological sequence and those strata that are of particular interest, either from the point of view of overall stability or for the design of the fixed anchor length. These key boreholes should be located at site extremities, so that the strata profile over the relevant zone can be initially interpolated between the boreholes rather than extrapolated outside the area investigated. The depth of these boreholes should be adequate to ensure that either:

- (a) a known geological formation is proved; or
- (b) no underlying stratum will affect design.

This preliminary work can then be followed by further boreholes and/or in situ tests, as applicable, to confirm the predicted strata sequence and obtain more detailed data on their strength and deformation characteristics. The number of locations investigated in this stage will depend upon the confidence that can be placed on the uniformity of the relevant strata. Thus, for example, a greater number of locations would be required in glacial drift than would be necessary to confirm the level and properties of a well known solid geological formation.

Notwithstanding these considerations, it has to be emphasized that minor structural or fabric changes can occur in a known geological formation that can significantly affect the behaviour of a ground anchorage. It is therefore recommended that the maximum centres of investigation locations should not exceed 20 m.

For retaining structures, the locations should be sited both along the line of the structure and, where practicable, along the line of the probable fixed anchor zone. Where long anchorages may be required, a number of locations should be sited at intermediate points to investigate the strata that will be penetrated by the drilling operations. Whilst it is not practicable to ensure the discovery of minor geological anomalies, the locations should so cover the site as to limit the possibility that a major geological feature, e.g. buried stream bed, can lie undetected between investigation positions.

In particularly difficult strata conditions, e.g. variable glacial drift, additional investigation may be required during the contract period at locations ahead of the construction (see 3.8.1).

Where rock is encountered during the investigation, the depth of penetration into this stratum should not be less than 5 m to ensure distinction between boulders and bedrock. In general, however, the encountering of rockhead at a reasonable level will imply the adoption of a rock rather than a soil anchorage. It will then be necessary to extend the investigation to depths sufficient to prove rock of the quality necessary to give adequate rock/grout bond strengths (see 4.2).

3.3.2 Methods of investigation. The most economical and effective method of obtaining the required data on ground conditions will depend on factors relating to the type of structure, the geological conditions and the information gained from the initial desk and field study. Thus, the best technique could range from an investigation by surface mapping in the case of shallow anchorages (rock bolts) into exposed rock, to boreholes by percussion and/or rotary drilling for deep soil and rock anchorages. On occasion, trial excavations or probing techniques may provide the most satisfactory solution.

The available techniques are detailed in BS 5930 and the particular methods relevant to ground anchorages are set out in appendix D.

3.3.3 Sampling (see clauses 19 and 20 of BS 5930 : 1981). Whilst the available sampling techniques are described in BS 5930, particular emphasis should be placed on obtaining samples that can identify the fabric/structure of the stratum in which the fixed anchor length may be installed. In view of the relatively small diameter of most anchorage systems, minor differences in ground characteristics can be of significant influence in the behaviour of the anchorage/ground complex (see 3.1, 3.3.1, 3.7.1).

In soils, samples of a size and type suitable for examination and laboratory tests relevant to the nature of the soil should be taken from each stratum and at maximum intervals of 1.5 m in thick strata. Intermediate disturbed samples, suitable for simple classification tests, should also be obtained, so providing a specimen of the ground at a maximum of 0.75 m intervals of depth. In variable strata, continuous undisturbed samples may be necessary in the probable vicinity of the fixed anchor zone.

In rocks, emphasis should be placed on obtaining maximum continuous core recovery. In most rocks, this generally implies the use of proper equipment to obtain cores of not less than N size (55 mm) and, in weaker rocks, of larger diameter (see 19.8 of BS 5930 : 1981).

Samples of ground water should always be obtained in clean containers for subsequent chemical analysis (see 20.3 of BS 5930 : 1981).

3.3.4 Ground water (see clause 20 of BS 5930 : 1981). Determination of the ground water conditions on the site will be essential for the overall design of the project, particularly where excavations are proposed, and is of equal importance in the design of anchorage systems. This area of investigation, particularly the recording of long-term ground water conditions, is too often given scant attention during routine site investigation works.

It has to be appreciated that the speed of boring, together with the addition of water to stabilize a borehole or as circulation fluid during drilling, usually precludes the measurement of equilibrium ground water conditions during the investigation period. Nevertheless, all observations of water conditions during boring or drilling should be carefully recorded throughout the progress of the investigation, as these often permit an initial assessment of true ground water conditions. For example, the percentage of circulation water return during rotary drilling may assist in a first appreciation of ground water level in a rock.

The long-term ground water conditions can only be measured satisfactorily by the installation of standpipes

or piezometers (see 20.2 of BS 5930 : 1981). Where ground water may be contained in several aquifers separated by impervious strata, it may be necessary to install piezometers at different levels to record the head in each aquifer.

3.4 Field testing

3.4.1 Tests in boreholes

3.4.1.1 Standard penetration test (see clause 21 of BS 5930 : 1981). Standard penetration tests should be made at regular intervals of depth in all granular strata in order to obtain an indication of their in situ state of compaction. In the probable zone of the fixed anchor length, tests should be made at least at 1 m intervals of depth in each borehole.

The test is also of value in obtaining a relative measure of the in situ quality of weak rocks. Although experience is required in the interpretation of such tests, they have been found to be of value in materials that are too weakly cemented to allow the sampling of rotary drill cores. In such tests, it is valuable to modify the standard procedure to allow the recording of the penetration of the probe under the influence of, for example, four sets of 20 blows. This technique is to be preferred to the measurement of penetration under a single set of, for example, 50 blows, since it allows the disturbance created by previous chiselling or other boring/drilling operations to be identified.

3.4.1.2 Radial load tests

(see 21.7 of BS 5930 : 1981).

The radial stress/strain characteristics of the ground mass in situ are often important in anchorage design. These characteristics can be obtained in granular and cohesive soils, as well as in soft rocks, by a pressuremeter test either within a borehole or by driving the pressuremeter into the ground inside a slotted casing (Sol Soils 1975). Recent developments in pressuremeter design include a self-boring probe (Windle & Wroth 1977) and (Hughes et al 1977).

For strong rocks, the Goodman jack may be appropriate (Goodman 1968), but the loaded area is small and correction procedures may be necessary. If results are difficult to obtain, it may be necessary to resort to estimating the in situ characteristics from the results of tests on core samples that relate only to the properties of the intact rock material.

3.4.1.3 Permeability tests

(see 21.4 of BS 5930 : 1981).

If anchorages are to be grouted into place, it is of value to determine the permeability of the ground mass in order to assess groutability. This may be done by injecting and measuring water flow via a single or double packer.

In fractured ground masses, a knowledge of the size and frequency of fractures may be more important than mass permeability, since the former may dictate the need for pregrouting (Littlejohn 1975) and the choice of grout.

Permeability may also determine the appropriate corrosion protection (see 6.2.1).

3.4.1.4 Remote sensing. Although at the time of publication of this Draft for Development remote sensing techniques cannot provide a substitute for the good core logging of rock, additional valuable information on the in situ conditions of the rock can sometimes be obtained (Burwell & Nesbitt 1954) and (Krebs 1967).

Provided that clear vision is not obscured by ground water or drilling fluid, features on the sides of boreholes may be viewed by television or photographic camera. The nature of the materials in the borehole may be determined less directly by measuring the diameter of the borehole by caliper logging. Such measurements may show variations in borehole diameter due to preferential erosion of weaker rocks by the drilling fluid. Successive measurements over a period of time may reveal deformation of the hole due to swelling. Impression packers are also available for borehole wall logging (Hinds 1974).

In-hole geophysical logging by electrical methods, i.e. resistivity, radioactive methods or seismic methods may be undertaken (Carroll 1966). Of these, probably seismic methods are the most useful (see D.4).

3.4.2 Tests in excavations (see clauses 29 and 30 of BS 5930 : 1981). Tests in boreholes may be of limited advantage because of the small sample size. Plate bearing tests allow larger samples to be tested but care is needed in the test technique if useful results are to be obtained. Moreover, interpretation of the results should take into account the relative geometry of the test conditions and the proposed anchorage system.

A form of test appropriate for ground anchorages in rocks is that in which the reaction to loading is provided by a grouted anchorage whose tendon passes through the centre of the plate (Zienkiewicz & Stagg 1966) and (Stagg & James 1966). This had the advantage of recovery of data concerning the anchorage problem at the same time as the more fundamental properties of the ground. Rock bolts may also be used as a reaction with similar advantages.

In situ shear tests may be arranged to test the strength of individual planes in a rock. These are generally time consuming and expensive and only warranted where a specific critical plane influences the stability of the structure.

Subsurface monitoring by extensometers is recommended where load transfer mechanisms and overall stability are under investigation.

3.5 Laboratory testing

3.5.1 Soils

3.5.1.1 Classification. Tests for the classification of soils are described in BS 1377. It is recommended that the grading of granular soils and the liquid and plastic limits of cohesive soils be determined for every stratum encountered in the investigation.

The determination of low plasticity indices in a cohesive soil, even in very localized zones, can influence both the type of anchorage and the method of drilling. For example, the drilling technique adopted may have to be that used for sands rather than clays, i.e. cased as opposed to uncased.

3.5.1.2 Shear strength. For granular soils, it is only possible to estimate the in situ shear strength characteristics on the basis of in situ tests, combined with grading and particle shape assessments from laboratory classification tests.

In granular soils of mixed grading, it may be justifiable to determine the peak shear strengths of samples prepared to a series of densities from loose to dense. The in situ strength may then be inferred by interpolation at the

density estimated from field tests. It should be noted that the shear strength is dependent on stress level. Hence, all laboratory tests should be made at a stress level approximating to that in situ.

For cohesive soils, the shear strengths should be obtained by triaxial compression tests. The type of test to be employed will depend on the design method, the mass permeability of the soil and the probable rate of stressing of the anchorage.

Where stressing of the anchorage is rapid, the undrained shear parameters are relevant in the necessary total stress analysis. For a slow rate of stressing or where the long term strength is required, the shear strength in terms of effective stresses can be obtained from a consolidated drained test. Alternatively, and particularly where the behaviour of the pore-water pressure during shear is significant, a consolidated undrained test with pore-water pressure measurement should be made. The rate of strain employed should be sufficiently slow to ensure equalization of pore-water pressure throughout the specimen (Bishop & Henkel 1962).

Where the soils under investigation possess a high mass permeability, e.g. silts, clays possessing a permeable fabric, chalk and marl, both undrained and effective shear parameters should be determined to permit a study of the influence of the relatively high permeability under different loading conditions of the anchorage.

In all tests, the stress/strain characteristics of the soil during shear should be recorded. In soils exhibiting a marked structure or fabric, e.g. laminated clays, it may be necessary to study the influence of these features on the shear strength of the soil in the plane of shear induced around the fixed anchor length.

3.5.1.3 Compressibility. For systems that will apply a high average stress to a clay lying between the fixed anchor length and the structure, the compressibility characteristics should be determined by testing in a suitable type of oedometer. The rate of consolidation determined by such tests will also assist in assessing the mass permeability of the soil, particularly where tests can be made on large diameter specimens in a hydraulic oedometer (Rowe 1972). Such data may influence the choice of design method, i.e. whether a total or effective strength analysis is relevant, and also provide guidance as to the probable loss of prestress through case history comparisons.

3.5.2 Rocks

3.5.2.1 Index tests

3.5.2.1.1 General. Index tests are cheap and easily undertaken tests that correlate approximately with other parameters and in themselves provide indications of rock quality, which are useful for general classification.

3.5.2.1.2 Point load tests. Point load tests are a form of strength test that may be conducted on cores or irregular fragments of rock. The results may be related to unconfined compressive strength (Franklin et al 1971), provided that the size of specimen is not less than 25 mm. The test apparatus is portable and can be used in the field as an aid to core logging.

3.5.2.1.3 Schmidt hammer tests. The apparatus used was developed for concrete testing. The L design hammer is often considered most appropriate to rock testing. Results

may be correlated to compressive strength (Martin & Miller 1974) but these are best conducted on outcrops or large blocks of rock rather than cores.

3.5.2.1.4 Sonic velocity. The velocity of ultrasonic pulses is measured usually with instruments developed for concrete testing. The velocities measured may be related to strength and deformability properties of the rock (ISRM 1977a). Tests are most easily conducted across core diameters. Such tests are necessary if velocity index on rock masses is to be established (see D.4.3).

3.5.2.2 Strength and deformability tests

3.5.2.2.1 Shear strength. The shear strength of a rock mass is commonly estimated from a detailed study of joint geometry and roughness together with a knowledge of the material characteristics (Hoek & Bray 1977) and (Hoek & Brown 1981). The design of ground anchorages is, however, usually based on borehole information and is often dependent upon a confining stress system local to the fixed anchor. The influence of joint systems may be less marked than, for example, in a problem involving rock slope stability. The results of laboratory tests on intact rock cores can therefore be useful. Shear strength may also be determined by shear box testing, conducted either on intact weathered material or, if required, on an existing discontinuity. Soils shear box testing apparatus is suitable only for soft rocks. Shear boxes capable of testing larger specimens are relatively rare. A portable field shear box has been developed (Franklin et al 1971).

3.5.2.2.2 Unconfined compressive strength. This is determined on specially prepared test cylinders using laboratory compression test apparatus (ISRM 1978a).

3.5.2.2.3 Unconfined tensile strength. This may be determined by direct pull, which involves quite complex test arrangements, or indirectly by the Brazilian test or the pierced disc test (ISRM 1977b).

3.5.2.2.4 Deformability. This is determined commonly by measuring stress/strain relationships in the unconfined compression test. Poisson's ratio may be measured if suitable strain gauges are installed. It should be noted that such tests provide a measure of the deformability characteristics of the rock material. The characteristics of the rock mass can only be determined by in situ tests.

3.5.2.3 Durability

3.5.2.3.1 Slake durability. The slake durability test apparatus (ISRM 1978b) assesses the susceptibility of rock to weathering.

3.5.2.3.2 Microscopic examination of minerals. Microscopic examination of the rock will determine the nature of the minerals of which it is composed (Shelley 1975). When this is known, the possibility of mineral reaction with grout or ground water may be assessed.

3.5.2.3.3 Swelling. The results of swelling tests are pertinent in determining the influence of anchorage construction methods such as flushing.

3.5.2.3.4 Drillability. A guide to drillability may be obtained by assessing in situ strength and structure (Blindheim 1979). Wear on drilling bits may be related to quartz content of the rock, which may be determined by microscopic examination (see 3.5.2.3.2).

3.6 Chemical testing

3.6.1 Aim of chemical analysis. The purpose of any chemical analysis of the ground is to determine its aggressivity to the constituents of the anchorage system and in particular the tendon and its grout or concrete surround (see 6.1.3).

3.6.2 Aggressivity towards metals

3.6.2.1 General. Some generalized measure of redox potential and soil resistivity can be applied to the assessment of potential ground corrosiveness to embedded metals (see 6.1.3.1).

3.6.2.2 Redox potential. Redox potential determination (King 1977) and (Constanzo & McVey 1957) also requires measurement of pH of the ground (BRE/CP 2 : 1979) (see BS 1377). The quantitative assessment of redox potential provides guidance on the risk of microbiological corrosion, frequently characterized by pitting attack and most commonly encountered in heavy clay soils (see J.2.4).

3.6.2.3 Resistivity. Methods of measurement vary but all involve passing a known current through the ground and measuring the voltage drop along the line of current flow (Palmer 1974). For soils that are well graded and homogeneous, the resistivity values should adequately predict the corrosion hazard. Non-cohesive soils are the most reliable to measure, i.e. there is less scatter of results (see D.4.2).

3.6.3 Aggressivity towards cementitious grout or concrete.

Ground aggressivity towards cement based grout or concrete is partly dependent upon the sulphate content of the ground and should be determined chemically (BRE Digest 250, 1981) (BRE/CP 2 : 1979). Soil aggressivity may be classified according to such determination (see 6.1.3.2).

3.7 Presentation of data

3.7.1 Description of strata (see clauses 41 and 44 of BS 5930 : 1981). All strata should be described in accordance with BS 5930 and the Working Party Report on the logging of rock cores for engineering purposes (QJEG 1970).

Of particular importance in investigations for soil anchorages is the recording of the structure or fabric of the soil. Relatively minor features in the strata can have a significant influence on anchorage behaviour in view of the limited volume of the stressed zone. For example, the presence of thin, even single-grain, partings of silt or sand within a clay can have a marked effect on the behaviour of the soil and, in particular, on the action of drilling water on the soil prior to grouting. This, in turn, can severely limit the load that can be placed on clay anchorages, particularly of the underreamed type. Similarly, the presence of even small pockets of granular material within a generally cohesive glacial drift can have a major influence on the installation technique that it is necessary to employ and, in extreme cases, may preclude the economic use of an anchorage system. Certainly such minor features within an apparently uniform soil can seriously affect the cost of a project.

In the case of rock anchorages, knowledge of the continuity of geological structures is important. Discontinuity orientation data is also valuable and may be recorded in various ways, but the most convenient is by use of the stereographic projection. This information can be vital in determining the size and shape of a rock mass liable to fail in service and is therefore critical in any overall stability analysis. The presentation of data in this way may facilitate analysis of slope stability problems (Hoek & Bray 1977). Discontinuity frequency data may be given by reference to various parameters such as rock quality designation, which may be obtained from the examination of rock cores. The frequency of fractures, for example, can be invaluable when back-analysing water test data to determine the need for pregrouting (see 8.2).

The symbology for the geological features observed in trial pits, trenches and adits in rock is best standardized. Basic symbology is given in clause 45 of BS 5930 : 1981. Further symbols are offered in the report on the preparation of maps and plans in terms of engineering geology (QJEG 1972).

These careful fabric/structure observations are rarely made in routine investigations for the overall project. This recommendation will therefore frequently provide the major justification for a detailed investigation during the feasibility study for an anchorage system.

3.7.2 Field data (see clause 39 and 40.2.6 to 40.2.8 of BS 5930 : 1981). All field data should be presented in a clear form following the recommendations of BS 5930.

3.7.3 Laboratory test results. All laboratory test results should be reported in accordance with BS 1377 or, for other tests, in such a form as to ensure that complete data are available for consideration during anchorage design.

3.8 Investigation during construction

3.8.1 Supplementary investigation. Where the initial and any detailed investigations show that ground conditions are liable to random variations, e.g. glacial drift, then it is desirable to infill the borehole information early in the contract period with probes or in situ tests. This work should provide immediate data at site level without the necessity of any but routine laboratory tests, which can conveniently be carried out in a site laboratory. Whilst the best techniques will depend on circumstances, the necessity for in situ results suggests the use of penetrometers or pressuremeters for the strength/deformation characteristics of soils. A simple boring/probing technique can be employed where the problem involves general variations in lithology, e.g. rotary percussive drilling to establish rockhead.

3.8.2 Field records. During the course of construction of the anchorage installation, further evidence on the nature of the ground conditions is revealed by excavation, by clearing the site or in the boreholes of the individual anchorages. This information should be recorded and appraised as an extension of the engineering geology work that formed part of the earlier investigation. No site investigation can explore ground conditions as frequently as the installation process can and any major variations in the ground conditions from those anticipated have to be recorded and their significance assessed. It should be emphasized, however, that production drilling associated

with anchorage installation is not geared to investigate the ground in detail. For example, production drilling cannot be expected to highlight local zones where silt or sand lenses occur in clay or variations in density in gravelly sand.

It is proposed that all ground data obtained during anchorage drilling be recorded on suitable proformas and subjected to daily analysis. Such a system can act as an early-warning device, should variation in strata levels or ground type require changes in design or installation method.

3.8.3 Anchorage behaviour during tests. The behaviour of anchorages subjected to proving or on-site suitability tests (see 9.2 and 9.3) should be considered as part of the ground investigation and carefully compared with all field and laboratory data. In particular, the behaviour of an anchorage in a semi-permeable soil, e.g. clay possessing a permeable fabric, silt, chalk and some marls, will be dependent on the ratio between the rate of dissipation of pore-water pressure and the rate of loading. Thus, it is possible that the available anchorage load may well be increased if stressing can be applied slowly or in a series of stage increments. Similarly, the behaviour of a clay soil between the fixed anchor length and the anchor head may well depend on the stressing procedure.

On a major project, where the costs can be justified by possible design economies, anchorages of specific types can be installed and tested as part of the field testing stage of a precontract investigation programme. Whilst much valuable data can be obtained from such a programme, care should be taken that data determined from the behaviour of a specific type of anchorage is not unjustifiably applied to other systems of different design or installation technique (see 9.2 and 9.3).

3.8.4 Adjacent activities. Any activities on the site that may influence anchorage behaviour should be carefully monitored, recorded and their possible influence assessed at an early stage in the work. Such activities include local excavation, ground water lowering, piling, blasting, freezing and mining subsidence.

3.9 Health hazards and safety. Consideration should be given to any environmental hazard that may be encountered when working in and around existing factory or chemical premises, or where industrial waste has been accumulated. Liaison is essential with the owners or occupiers of such premises or with other competent persons to determine the hazards and the type of protection needed.

4. Design considerations for fixed anchor

4.1 Overall stability

4.1.1 General. Assessment of overall stability should consider the interaction between the ground, structure and anchorages as a complete system.

Deformations of this system are generally the most important consideration.

Anchorages are used in five main ways that require overall stability to be considered:

- (a) restraint for deep excavations;
- (b) stability of slopes in soil or rock;

- (c) stabilization of permanent underground excavations in rock;
- (d) stabilization of new and existing structures in soil or rock;
- (e) security of structures subject to vertical or transitory loads.

4.1.2 Restraint for deep vertical excavations in soil

4.1.2.1 Fundamental considerations. General guidance is given in table 1 after Lambe (1970). This may be augmented by reference to South African Institution of Civil Engineers (1967), Zinn (1968), Lacroix (1969), Peck (1969), Puller (1975), Institution of Structural Engineers (1975), Milligan (1977) and 4.3 of CP 2004 : 1972.

Prediction of movement caused by excavation and dewatering (see step 6 in table 1) is difficult. Figure 2 indicates the general prediction process and examples are described by Burland & Hancock (1977), Creasy & Zinn (1977) and Burland et al (1979).

The essential point to come from this work is that no excavation, however well supported, can be made without causing ground movement.

Failures of excavations also continue to occur due to lack of understanding of the site, future loads or site changes, despite advances in analytical design methods (Sowers 1975).

4.1.2.2 Methods of analysis. A planar failure surface may be assumed for a wall supported by one level of anchorages. Figure 3 shows an empirical method for the approximate location of fixed anchor zone with a planar failure surface. The design procedure is shown in figure 4 and table 2 gives details of case histories using this assumption.

Further guidance is given in Section 3.2.2.1 of Civil Engineering Code of Practice No 2 (1951).

The sliding block method (see figure 5) may be used as an alternative method of analysis with cohesionless soils only. A circular failure surface of the type shown in figure 6 is strongly preferred for a wall supported by more than one level of anchorages e.g. the Bishop simplified method of slices (Bishop 1954), in view of the lack of experimental data to confirm the validity of a sliding block method at the time of publication of this Draft for Development.

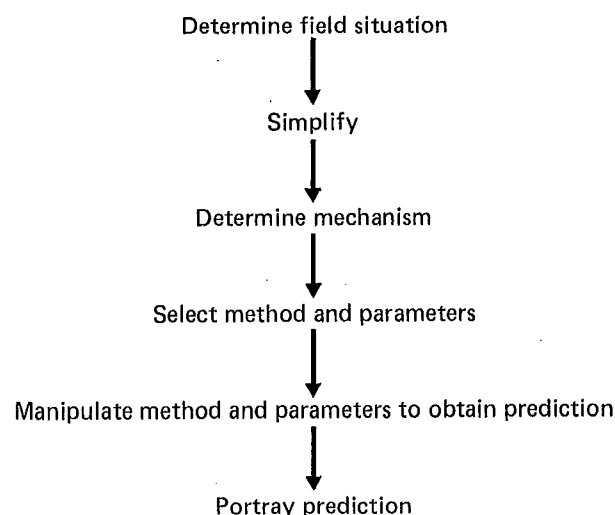


Figure 2. Prediction process (after Lambe (1973))

Table 1. Engineering an excavation (after Lambe (1970))

Step no.	Activity	Considerations/comments
1	Select dimensions of excavation	Structure, size, depth requirements, depth to good soil, depth to float structure, stability requirements
2	Investigate ground conditions, outside site/ inclined borings if necessary (see 3.3.1)	Decide on appropriate parameters and use effective stresses
3	Survey adjacent structures and services	Size, type, age, location, condition
4	Establish permissible movements	See E.1.3
5	Select propping/anchoring (in needed) and construction scheme	Local experience, cost, time available, depth of wall, type of wall, type and spacing of props/anchorages, dewater excavation sequence, prestress of propping needed?
6	Predict movements caused by excavation and dewatering	Assess stability of base of excavation and risk of local failure
7	Compare predicted with permissible movements	
8	Alter propping/anchoring and construction scheme if needed	
9	Determine division of responsibility between design and supervision of construction	An essential step, e.g. consultant: design and supervision contractor: construction
10	Instrument and monitor construction and alter bracing as needed	

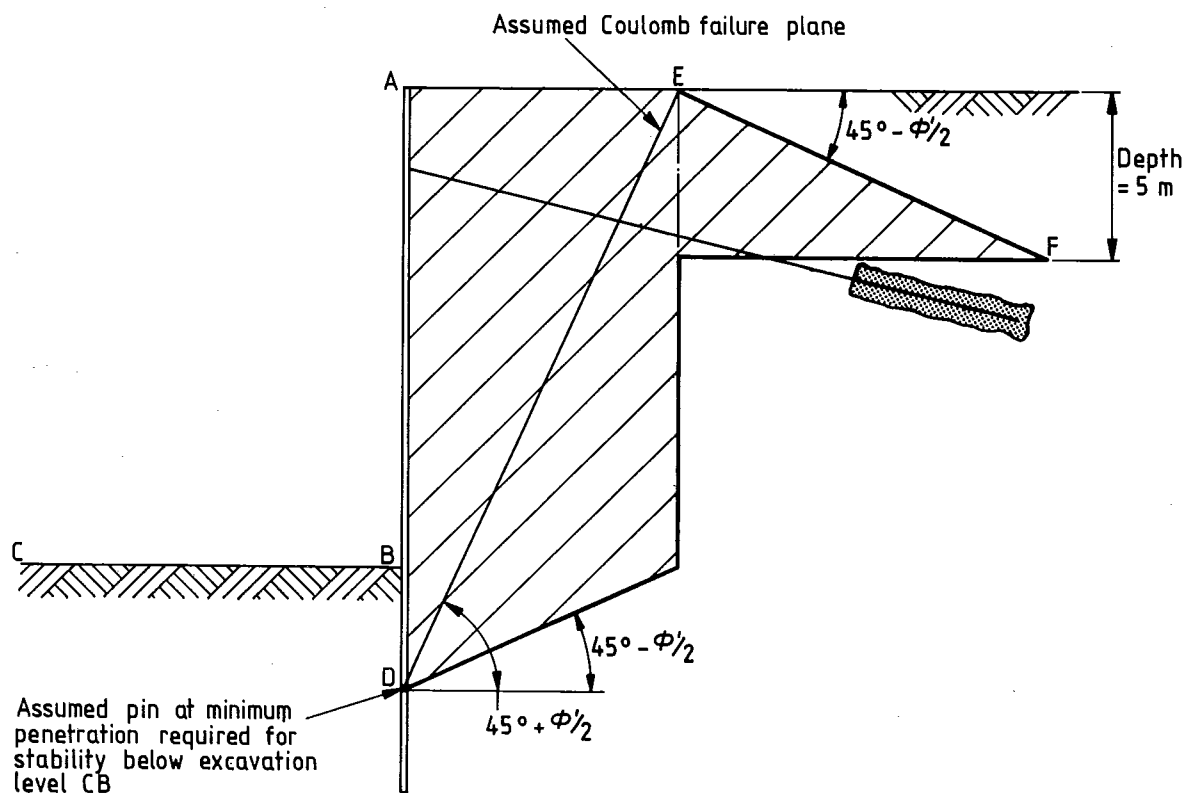


Figure 3. Empirical method for approximate location of fixed anchor zone

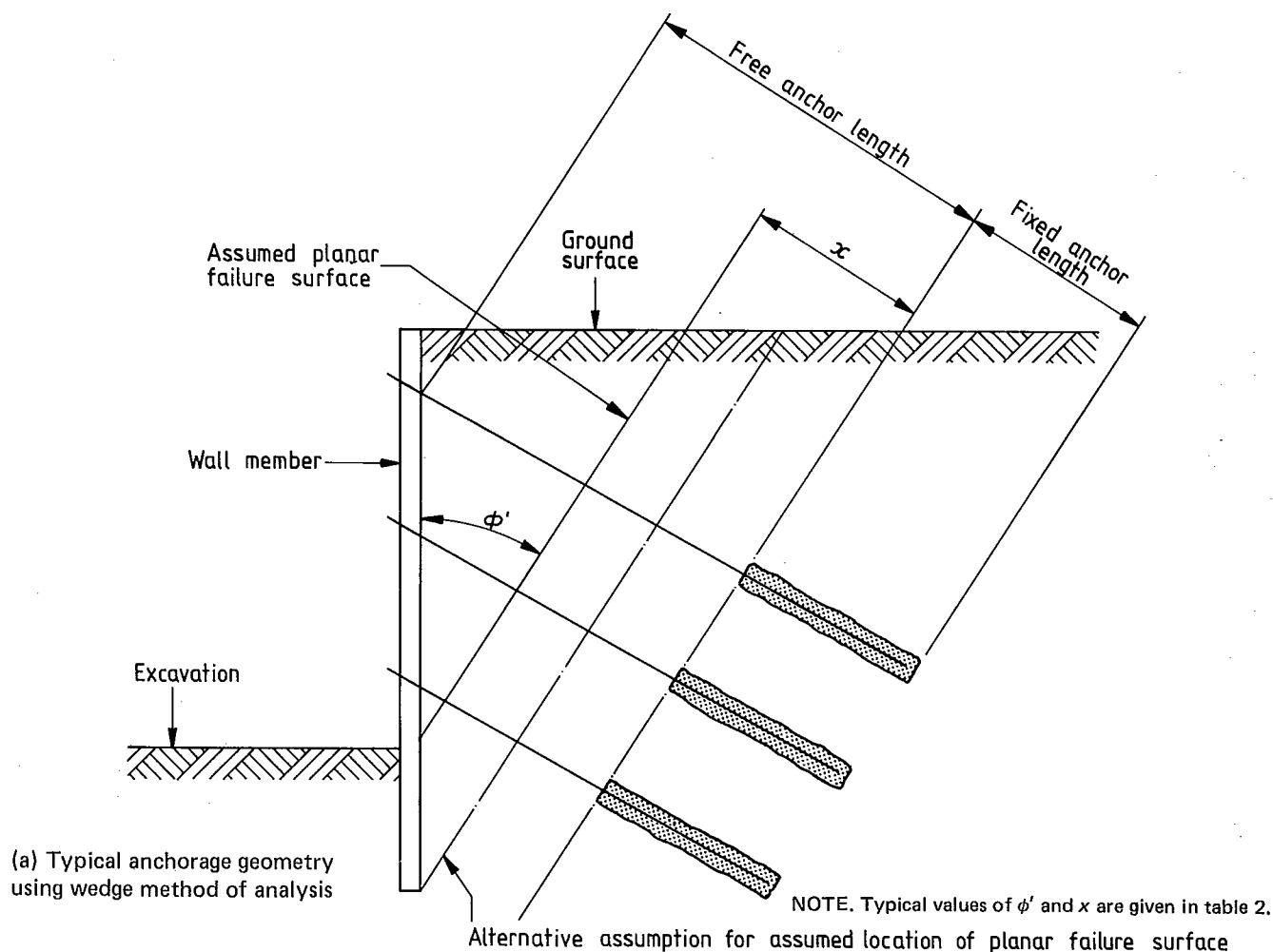
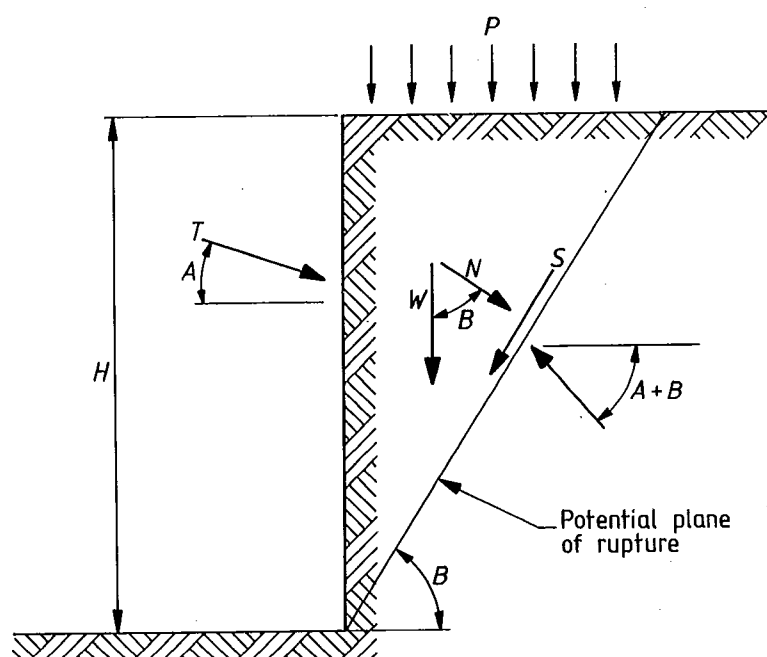


Figure 4. Wedge method of analysis

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where:

P is the surcharge;
 W is the weight of sliding wedge $= \frac{1}{2} \gamma H^2 \cot B$;
 γ is the density of wedge;
 H is the depth of excavation;

Known $\left\{ \begin{array}{l} S \text{ is the shear resistance of retained material along plane of rupture} \\ (S = (c'H/\sin B) + N \tan \phi'); \\ A \text{ is the angle of inclination of anchorages;} \\ \phi' \text{ is the angle of shearing resistance of ground;} \\ c' \text{ is the cohesion of retained material;} \end{array} \right.$

Unknown $\left\{ \begin{array}{l} B \text{ is the angle of inclination of potential plane of rupture} \\ (B \text{ should be varied and plotted against values of } T); \\ T \text{ is the anchorage force.} \end{array} \right.$

$$T = \frac{(P + \gamma H/2) H \cos B (F - \cot B \tan \phi') - c'H/\sin B}{\sin (A + B) \tan \phi' + F \cos (A + B)}$$

$$S_f = \frac{c'H/\sin B + [(P + \gamma H/2) H \cos B \cot B + T \sin (A + B)] \tan \phi'}{(P + \gamma H/2) H \cos B - T \cos (A + B)}$$

where

S_f is the factor of safety required;
 N is the normal force on wedge $= (P + \gamma H/2) H \cos B \cot B + T \sin (A + B)$;
 T should not exceed the working stress of the tendon.

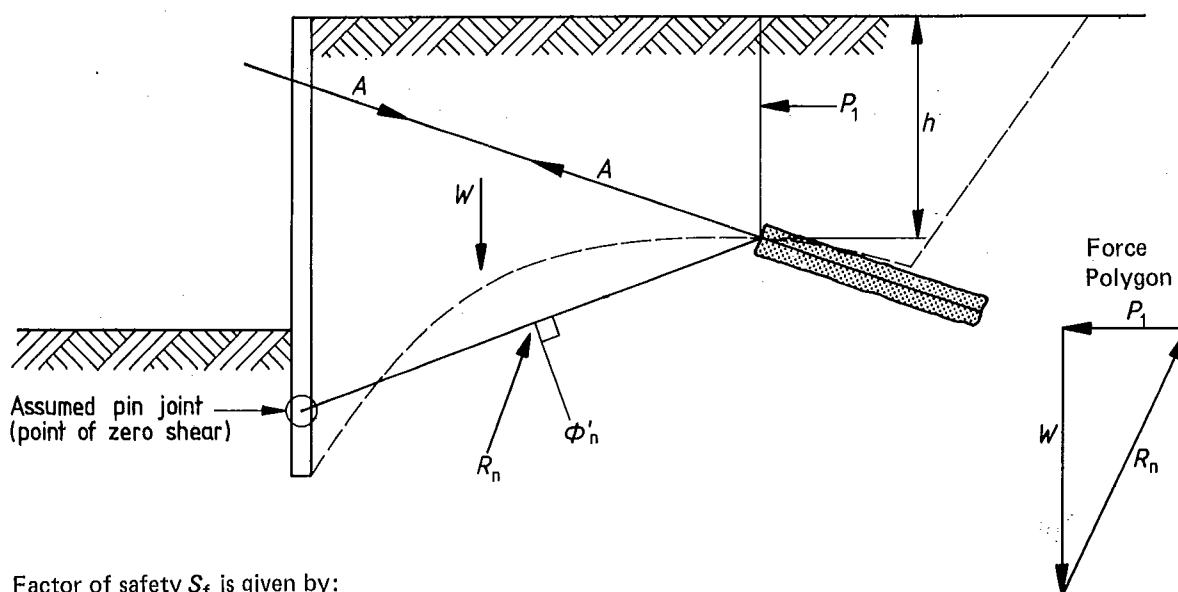
(b) Forces acting on wedge mechanism of failure

Figure 4. (concluded)

Table 2. Examples of deep excavations where overall stability has been judged by wedge method

Reference	Depth of excavation	No. of anchorage levels	Anchor loads	Soil type	Groundwater conditions	Angle of failure plane to vertical ϕ'	x	Remarks
Mansur & Alizadeh (1970)	m 10.0 to 14.0	2	kN 450 (upper) 315 (lower)	Silty clay and shaley clay	m Below base of dig	45° (base of dig)	m 3.4 to underream	Excavation underlain by limestone (king-post and poling board wall)
Hanna & Seeton (1967)	14.0	4	900 (three upper) 500 (lower)	Clayey then sandy till	1.0 below ground level	not specified (base of dig)	Zero	Excavation underlain by shale (king-post and poling board wall)
Clough & Tsui (1974)	15.0	4	—	Clay	—	45° (base of dig)	—	Parametric study of effects of anchor/wall stiffness
Malijan & Van Beveren (a) (1974)	21.0 to 27.0	10	410	Alluvium and shale	4.0 to 9.0 below ground level	35° (base of dig)	Zero	Excavation underlain by shale (king-post and poling board wall)
	21.0 to 34.0	9	360	Firm clays and silty sands	Below base of dig	35° (base of dig)	Zero	King-post and poling board wall
Liu & Dugan (1972)	16.5	7	300	Firm clays and dense sands	Below base of dig	35° (base of dig)	Zero	King-post and poling board wall (fully instrumented)
Larson, Willette, Hall & Gnaedinger (1972)	15.0	4	400	Dense to very dense sands and gravels	Below base of dig	45° — $\phi'/2$ (base of dig)	1.2 to 1.5	King-post and poling board wall
Wosser & Darragh (1970)	12.0 to 20.0	4	200 to 400	Cemented sands and silt clays	10.0 below ground level	30° (base of pile)	—	King-post and poling board wall underlain by bedrock
Booth (1966)	12.0	3 to 5	300 to 600	Silt sands and clays	Below base of dig	45° (base of dig)	3 to rear of underream	King-post and poling board wall
Dietrich et al (1971)	15.0	2 to 4	450	Sands and fills	Below base of dig	27° (base of pile)	Zero	King-post and poling board wall, flows into excavation from water mains
Ware et al (1973)	12.0 to 15.0	4	400 450	Silty sands and clays	Below base of dig	40° (base of dig)	Zero	King-post and poling board wall
White (1970)	—	—	—	—	—	35° (base of dig)	4.6 to 6.1 and to single bell anchor	General design procedure
Clough et al (1972)	18.3	4 to 5	556 to 890	Fill and silty clays	Below base of dig	30° (1.5 m below base of dig)	—	King-post wall
Private communication (1977)	8.0	4	400	Stiff fissured clay	—	35° base of wall	Zero	Diaphragm wall. (Neasden)
Milligan (1977)	—	—	—	—	—	40°	—	Average value for base of excavation

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Factor of safety S_f is given by:

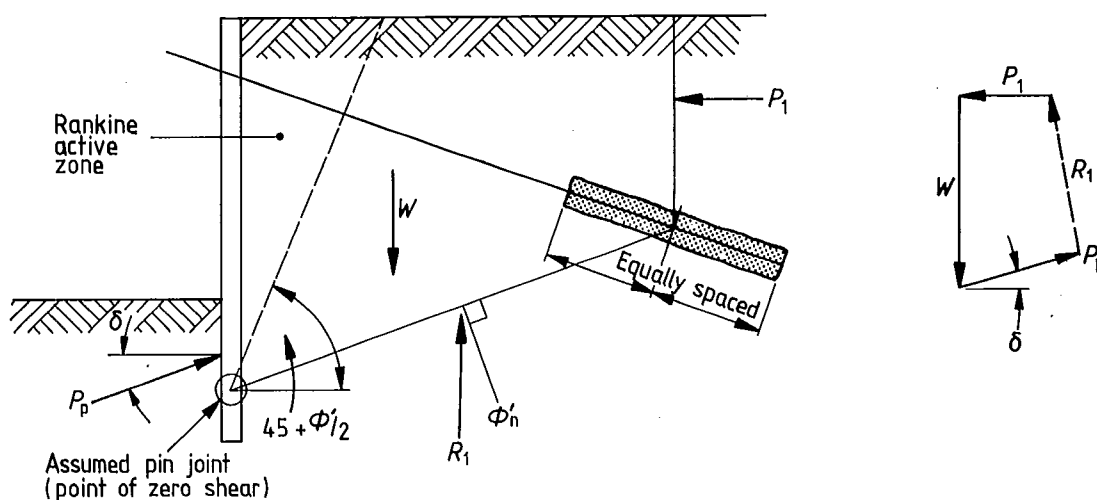
$$S_f = \frac{\tan \phi'}{\tan \phi'_n}$$

where

ϕ'_n is nominal or mobilized angle of shearing resistance (in degrees).

NOTE. If ϕ'_n has been correctly assumed, the weight W and the forces R , and P_1 are in equilibrium. If this is not the case ϕ'_n has to be altered.

(a) Modified by Locher (1969) and Littlejohn (1977)



Factor of safety S_f is given by:

$$S_f = \frac{\tan \phi'}{\tan \phi'_n} \geq 1.2$$

where

ϕ'_n is the nominal or mobilized angle of shearing resistance of the soil (in degrees).

(b) Modified by Ostermayer (1977)

Figure 5. Sliding block method of analysis

Alternatively, the method as modified by Janbu (1956 and 1957) may be used for circular and non-circular failure surfaces (see figure 6).

The fixed anchor length should be positioned beyond the critical failure surface giving the appropriate factor of safety. On the basis that the anchorage forces have a prime role in holding the soil block together during sliding, the forces are regarded as part of the internal force system and no allowance should therefore be made for the effect of the anchorage forces on a potential slip surface (Ranke & Ostermayer 1968), (Littlejohn 1970) and (Malijian & Van Beveren 1974).

Finite element methods (FEM) may also be used to:

- (a) compare computed and observed behaviour;
- (b) carry out a parametric study.

The value of FEM depends principally on the degree of confidence in the quality of data for the properties of the ground, particularly stiffness (Simpson et al 1980).

4.1.2.3 Design considerations. The principal aim for the design of walls supported by ground anchorages is to minimize vertical and horizontal movements around the excavation.

To achieve this, the following should be used as a guide.

- (a) The base of the wall and excavation should be stable at all times.
- (b) King-post and poling board walls are particularly liable to failure due to instability at the base of the wall.
- (c) Wall stiffness has little effect on ground movement (Burland et al 1979).

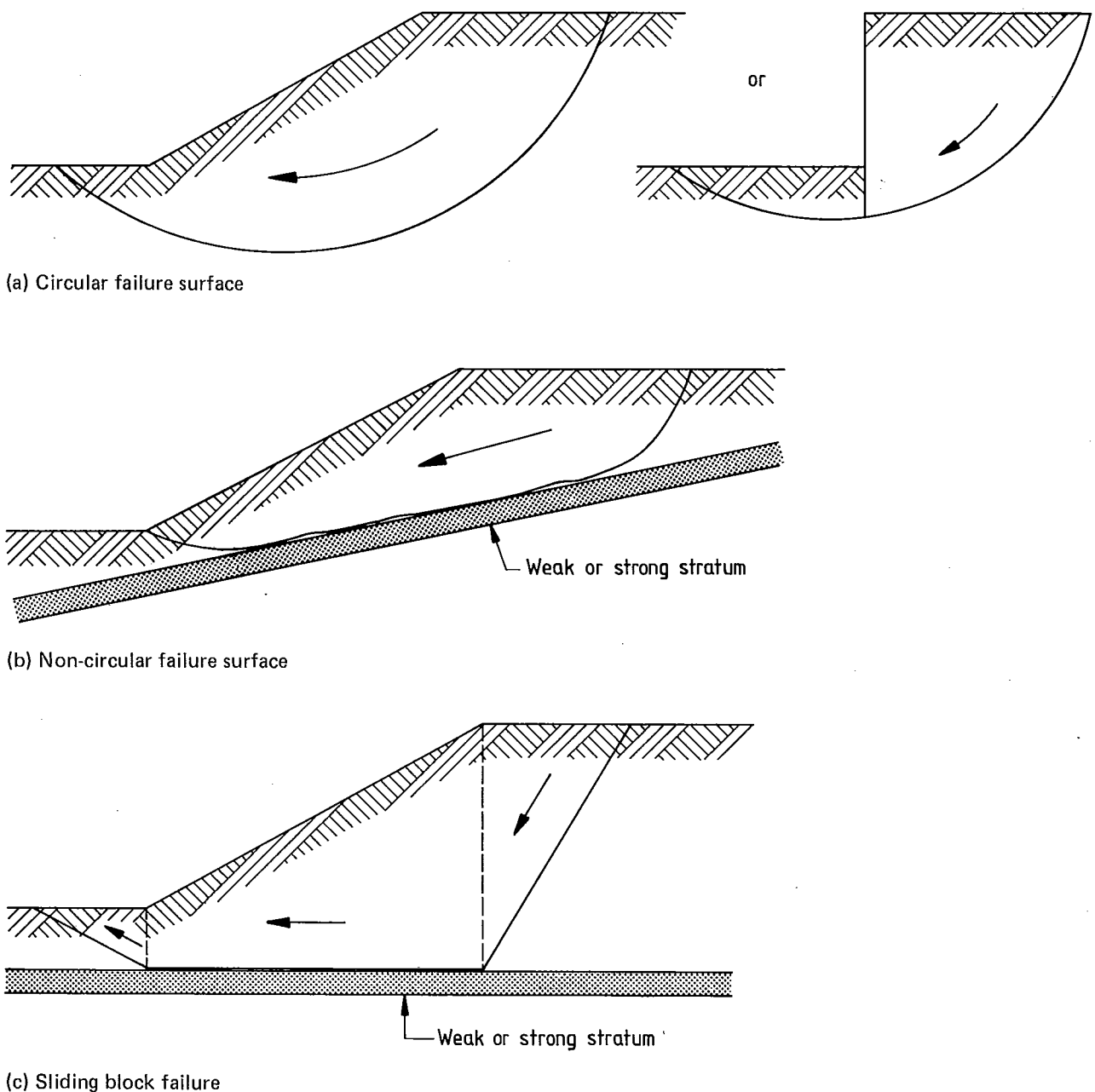


Figure 6. Types of failure surface in soils (after Lambe & Whitman (1969))

(d) Beams or props can be used effectively in conjunction with ground anchorages to control wall movements, provided that the base of the excavation is stable (Clough & Davidson 1977) and (Burland et al 1979).

(e) Continuous walings should be used for king-post and poling board walls. Poling boards should be located either just behind or in front of the front flange of the king-post (see figure 7). Walings can be omitted where the consequences of failure can be properly assessed and accepted.

(f) The location of anchorages at re-entrant corners requires careful consideration to avoid interaction (see figure 8).

(g) Table 3 and figure 9 give details of measured

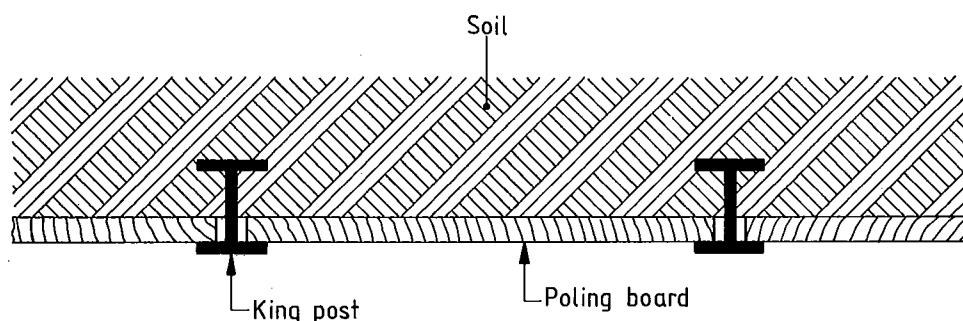
movements of the crest of anchored walls and of the settlement behind the walls.

The ratio of horizontal to vertical movement depends on the amount of support and the rigidity of the ground at the base of the excavation.

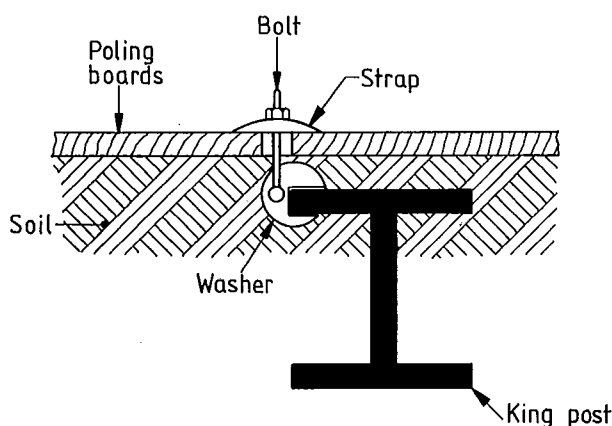
The ratio can exceed 3 in an open excavation and be less than unity for a well supported wall. The wall acting as a cantilever as shown in figure 9(b) should be taken as the most likely deflected shape of a wall supported by ground anchorages.

(h) Measurements of the movement of the wall and the ground/buildings behind the wall should always be made (see 9.5 and 10.5).

A more detailed discussion of design considerations is given by Mitchell*.



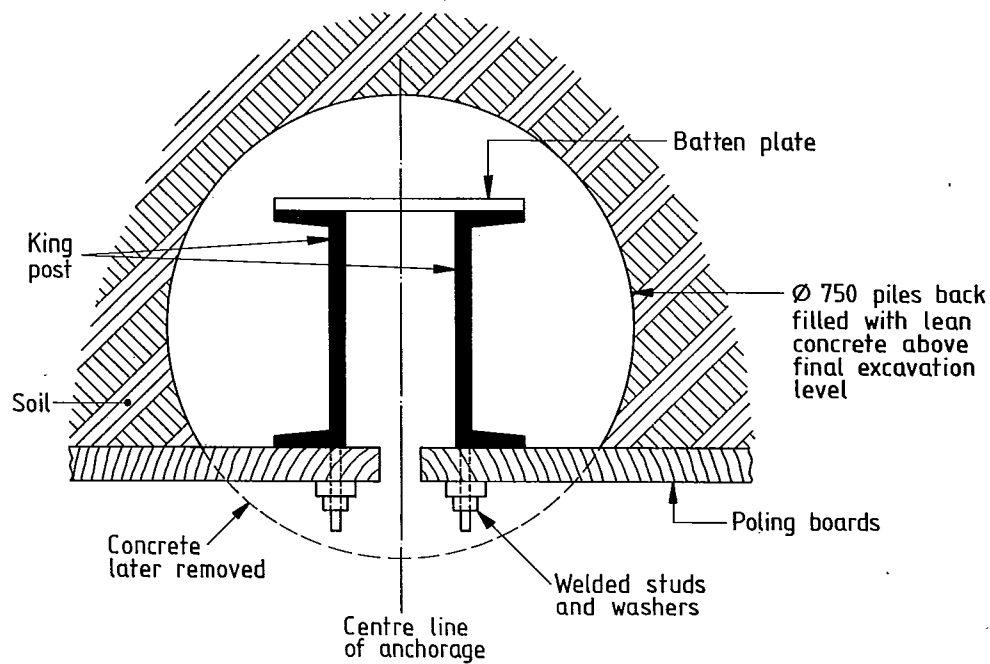
(a) Recommended location of poling boards



(b) Contact poling boards (after Hanna & Littlejohn (1969))

Figure 7. Details of poling boards for king-post/soldier pile walls

*In course of preparation.

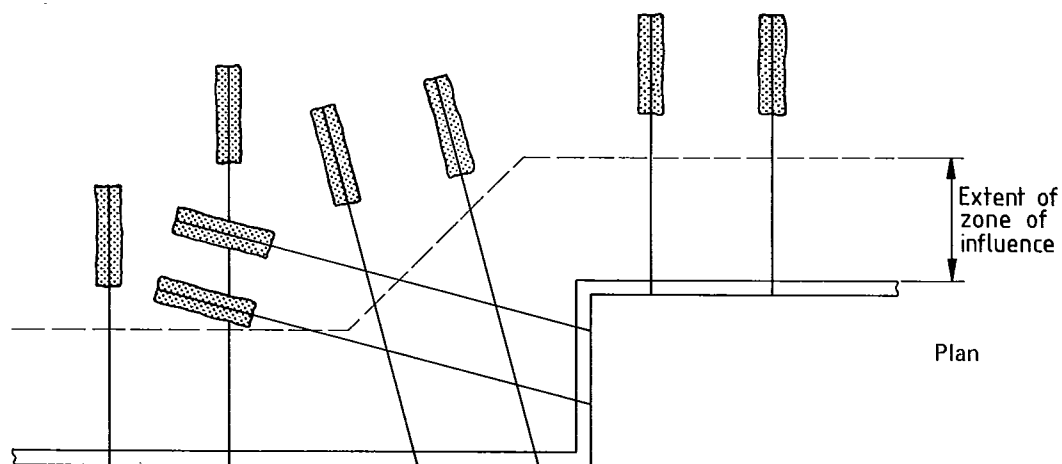


Dimensions are in millimetres.

(c) Contact poling boards (after Wosser & Darragh (1970))

NOTE. See also Peck (1969).

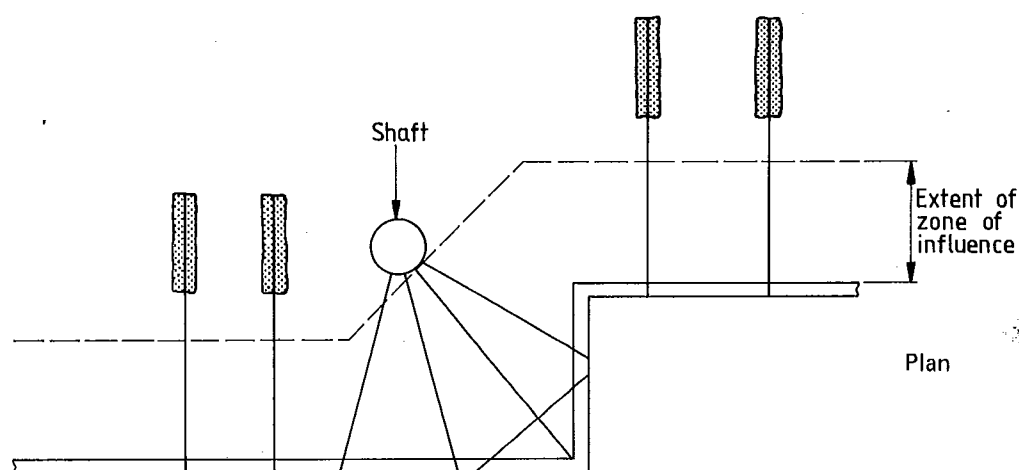
Figure 7. (concluded)



(a) Using only anchorages

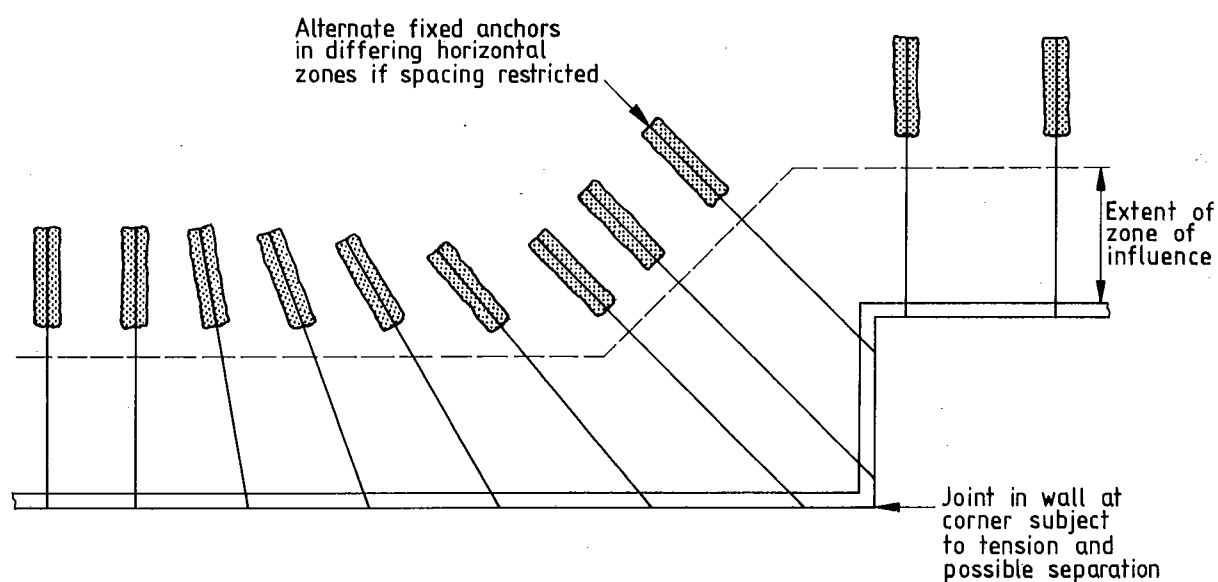
Figure 8. Anchorages at re-entrant corners

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(b) Using a shaft (after Ostermayer (1977))

NOTE. Zone of influence based on choice of method of analysis, e.g. wedge, sliding block, circular.



(c) Corner without cross-over of tendons

Figure 8. (concluded)

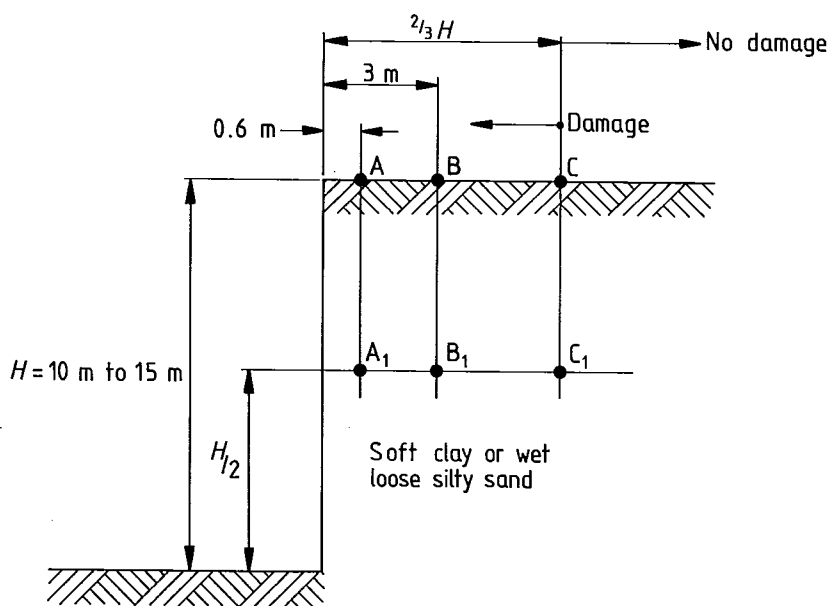
Table 3. Movements of anchored walls (soils group in accordance with Peck (1969))

Reference	Depth of excavation (H)	Crest displacement (Δ)	Δ/H	Settlement behind wall (δ)	δ/H	Wall type	Remarks
Group one. Cohesionless sands							
Shannon & Strazer (1970)	m 24.6	mm 75	% 0.30	mm 75	% 0.30	King-post and guniting	Bearing capacity failure of king-posts required emergency measures
Dietrich et al (1971)	17.4	46	0.25	—	—	King-post and poling board	King-posts settled 63 mm
Trow (1974)	10.5	25	0.25	—	—	King-post and poling board	
	10.0	25	0.25	—	—		
Saxena (1975)	17.0	67	0.39	—	—	Reinforced concrete diaphragm	Panel G21 Panel Y16 returned to original position after final anchorages installed. Bending moments measured
	19.50	146	0.75	—	—		
Jelinek & Ostermayer (1976)	20.0	140	0.7	—	—	Reinforced concrete diaphragm	
Rizzo et al (1968)	11.3	20.38	0.18 to 0.34	—	—	King-post and poling board	Active method
Group two. Cohesive granular soils (e.g. clayey sands, silty sands, sandy clays)							
Phillips & James (1973)	10.4	10	0.10	—	—	Reinforced concrete diaphragm	Bending moments also measured
Sandqvist (1972)	6.5	15	0.23	—	—	Sheet pile	Deflection due to frost heave (section 3)
Larson et al (1972)	15.2	25	0.16	25	0.16	King-post and poling board	Ravelling of soil occurred during placing of poling boards
Ware et al (1973)	14.8	50	0.34	16	0.11	King-post and poling board	Maximum street settlement 6.1 m from excavation
Malijian & Van Beveren (1974)	33.0	75	0.23	75	0.23	King-post and poling board	Cavity during anchorage installation
Corbett & Stroud (1975)	15.3	20	0.13	18	0.12	King-post and poling board	Movements behind wall were upwards (heave)
Hodgson (1975)	15.0	3	0.02	28	0.19	Reinforced concrete diaphragm	Toe of wall in stiff clay. Wall moved 25 mm when electrical vibrator used close by
St John (1975)	(first stage) 10.0	(base of excavation) 6	0.06	—	—		Excavation by mining ground floor as prop (YMCA)
Rizzo et al (1968)	6.4	2.5	0.04	—	—	King-post and poling board	Soft clay over rock 'at rest method'
Group three. Soft to medium clays							
Sandegren et al (1972)	6.7	28	0.42	—	—	Sheet pile	Deflection due to frost heave
Cunningham & Fernandez (1972)	7.0	100 to 18	1.43 to 0.27	—	—	Reinforced concrete diaphragm	After wall heated
Murphy et al (1975)	10.0	5	0.05	—	—	King-post and poling board	Varied clay underlain by dense till
Broms & Stille (1976)	6.3	50 to 150	1.60	17.5 to 4.0	0.28 to 0.06	Sheet pile	Block Manfred, Västerås settlements measured 6 m and 13 m behind wall
	4.0 to 5.0	50 to 100	1.50	—	—	Sheet pile	Blocks Hjotter & Spiran-Novrkopiny piles settled some amount as crest displacement
	4.0	510 to 610	—	410	—	Sheet pile	Wall failure, Lilla Bälstörn Gothenburg

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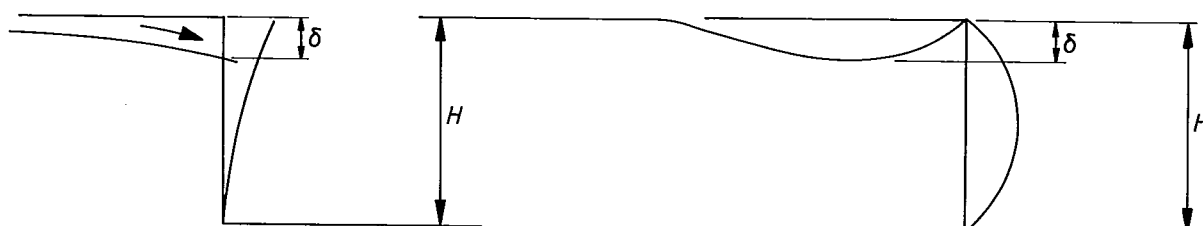
Table 3. (concluded)

Reference	Depth of excavation (H)	Crest displacement (Δ)	Δ/H	Settlement behind wall (δ)	δ/H	Wall type	Remarks
Group three. Soft to medium clays (concluded)							
da Costa Nunes & Diaz (1978)	m 20.0	mm 250	% 1.25	mm —	% —	Concrete wall	Residual soil (partially collapsible). Earth pressures measured to control design
Fenoux (1978)	17.5	50 to 60	0.28 to 0.34	—	—	Tee-shaped diaphragm	Excavation underlain by gravel and rock
Stille & Fredriksson (1979)	7.5	10	0.13	—	—	Sheet pile	Ground settled 50 mm 7.5 mm behind wall
Group four. Stiff clays							
D'Appollonia et al (1967)	8.8	60	1.80	—	—	Sheet pile	Slope in colluvium
Jennings (1969)	14.7	76	0.52	—	—	Bored piles	Damage to services in street and buildings across street. Soft jointed rock
	14.7	38	0.26	—	—	Bored piles	
	18.3	25	0.14	—	—	Concrete poling board	
Wosser & Darragh (1970)	12 to 18	6	0.04	—	—	King-post and poling board	Site underlain by rock
Mansur & Alizadeh (1970)	10.5	12.0	0.11	—	—	King-post and poling board	Shaley clay
McRostie & Schriever (1967)	12.0	90	0.40	75	0.63	Sheet pile	Building settled 75 mm at distance of 4.60 m for excavation
Liu & Dugan (1972)	18.0	15.0	0.10	15.10	0.08	King-post and poling board	Street 7.63 m settled 28 mm and king-posts settled 10 mm
Clough et al (1972)	18.3	—	0.10	30.5	0.15	King-post only	
Nelson (1973)	31.0	100.0	0.30	25	0.08	King-post and poling board	Soft siltstone: street settlements may be high
Mallijian & Van Beveren (1974)	28.0	50.0	0.15	—	—	King-post and poling board	Settlements of king-posts of 100 mm to 150 mm caused cracks in roads and broke services
Littlejohn & Macfarlane (1975)	14.5	22.0	0.15	—	—	Reinforced concrete diaphragm	Panel settled 12.2 mm. Bending moments measured. Top anchor level in gravel
Sills et al (1977)	8.0	50.0	0.63	20.0	0.25	Reinforced concrete diaphragm	Panel settled 30.0 mm. 20.00 mm settlement occurred 4.0 m behind wall
Haines et al (1980)	11.25	20	0.18	—	—	Reinforced concrete diaphragm	Gravel off. Stiff Keuper Marls. 6.8 m cantilever to first anchorage level
Sommer (1980)	15.0	5.0 to 20.0	0.03 to 0.13	—	—	Bored piles	Limestone layer at base of excavation
	16.50	35	0.21	—	—	Bored piles (piles moved 10 mm into excavation at lowest slab level)	Horizontal displacement of about 10 mm, 16 m from excavation. Ground heaved 60 mm outside excavation



(a) After Lacroix (1969)

Point	Settlement	Lateral movement
	mm	mm
A	50 to 150	25 to 50
B	25 to 75	12 to 38
C	6 to 19	3 to 12
A ₁	50	30 to 75
B ₁	12	25 to 38
C ₁	3	3



(1) Wall acts as cantilever
(horizontal movements greater than vertical)

(2) Wall propped at ground level
(horizontal movements less than vertical)

Soil type	δ/H for preliminary design	
	Milligan (1977)	Burland et al (1979)
Soft clays	2	2
Loose sands and gravels	0.2 (dense sands and tills)	0.5
Stiff clays	0.5	< 0.15

(b) After Burland et al (1979)

Figure 9. Typical wall movements outside an excavation

4.1.3 Restraint for deep vertical excavations in rock

4.1.3.1 Fundamental considerations. In general, the considerations are identical to those for slopes in rock (see 4.1.5).

4.1.3.2 Methods of analysis. The approach for assessing the stability of rock slopes recommended in 4.1.5 is applicable to rock excavations.

For weak and weathered rocks, the methods of analysis recommended in 4.1.2 for soils should apply.

4.1.3.3 Design considerations. Where deep excavations are solely in rock, the design considerations recommended in 4.1.5 should be applied. In the case where soil overlies rock at the base of the excavation or where there is a weathering profile in the rock, some special considerations may apply. These include the following:

(a) ensuring adequate support for the toe of the anchored wall;

(b) controlling spalling of the rock below the base of the wall (White 1975).

4.1.4 Stability of slopes in soil

4.1.4.1 Fundamental considerations. Section three of BS 6031 : 1981 gives helpful advice on assessment of a slope where anchorages are considered as a remedial measure. Figure 6 shows typical failure surfaces.

For such problems, a thorough understanding of the geology and ground water regime is essential. Further consideration in slope stability analysis (Janbu 1957) may be summarized as follows.

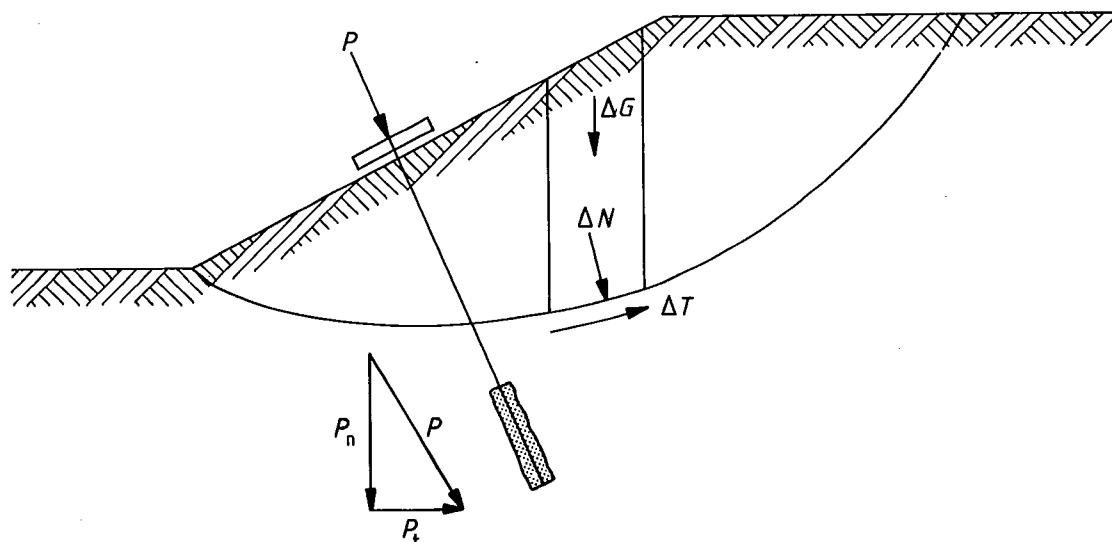
- Carry out the stability analysis in terms of effective stress, using pore pressures obtained from a flow net.
- Deformation and stress levels may also have to be estimated in producing a minimum factor of safety.
- Judgement should be exercised on reliability of soil parameters selected for design.
- Allowance should be made in the design for layered or fissured soils.
- Residual strength of soils may be applicable in design (see 4.1.4.3).

Skempton & Hutchinson (1969) and Skempton (1977) give guidance on dealing with many of these problems. Ground water control should be considered, since in most cases anchorages are used in conjunction with additional drainage to stabilize the slope (Reti 1964) and (Camberfort 1966).

4.1.4.2 Methods of analysis. For a circular failure surface (see figure 6) the Bishop simplified method of slices may be used (Bishop 1954) with the fixed anchor length positioned beyond the critical slip surface giving the appropriate factor of safety.

Allowance for the anchorage forces on the slices should be made by adding a line load at the surface of the slope to represent an increase in the normal stresses on a potential failure surface.

Alternatively, the friction circle method (Taylor 1948) may be used with the modifications suggested by Hobst (1964). Details of the method of analysis are shown in figure 10. For the non-circular failure surface shown in figure 6, the method of slices suggested by Janbu (1957) may also be employed. Where the potential failure surface



With anchorage forces acting normally to slide area, factor of safety (S_f) is given by:

$$S_f = f \frac{(\Sigma \Delta N + P_n) + \Sigma c \Delta L}{\Sigma \Delta T \pm P_t}$$

and

$$P_n = S_f \frac{\Sigma \Delta T - \Sigma c \Delta L}{f} - \Sigma \Delta N$$

where

- ΔG is the self-weight of the sliding area strip (in kN);
- ΔN is the normal weight component (in kN);
- f is the friction coefficient ($f = \tan \phi'$);
- r is the sliding area radius (in m);
- c is the cohesion of the soil (in kN/m²);
- ΔL is the width of the sliding area strip (in m);
- ΔT is the sum of the tangential forces acting on sliding area strip (in kN);
- P_n is the total normal weight (in kN);
- P_t is the total tangential force (in kN).

NOTE. Method of analysis due to Hobst (1964), based on the friction circle method (Lambe & Whitman 1969).

(a) Definition of forces

Figure 10. Method of analysis of soil slopes

can be approximated to two or three straight lines (see figure 6), the wedge method described by Lambe & Whitman (1969) may be used.

4.1.4.3 Design considerations. A careful choice of soil parameters is required (see clause 3) because small changes in the value of cohesion, for example, will make great differences in calculated factors of safety (Brandl 1979). A parametric study should therefore be carried out using the fully softened strength of the soil in the analysis of potential slips (Skempton 1977).

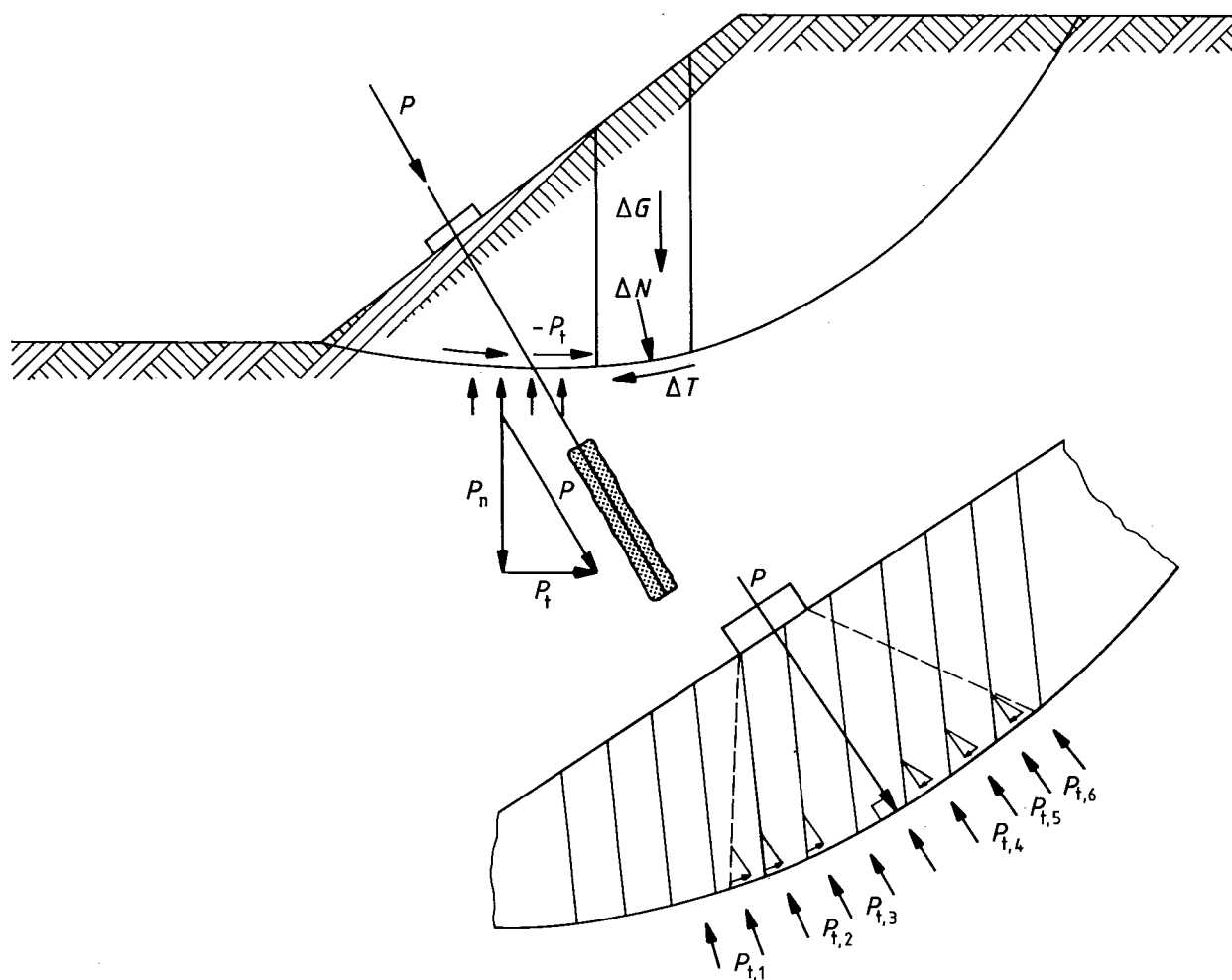
Fully softened strength is a property used to describe a completely remoulded normally consolidated clay. This is equivalent to the lower limit of strength measured on the discontinuities and may involve displacements of up to about 0.40 m that do not take place on pre-existing

shear surfaces. Residual strength (Skempton 1964) is much smaller and should be applied to natural slopes and where displacements of 1.0 m to 2.0 m have already occurred.

These recommendations apply principally to stiff clays (Simpson et al 1980).

The centroid of the anchoring forces should be placed near the likely point of intersection of the failure surface with the ground surface (see figure 6).

The bearing of the anchor heads on the ground surface should consist of a continuous concrete beam or individual concrete pads. These should be substantial enough to ensure that deflections that could cause a loss of load in the anchorages are controlled.



$$-\sum_1^3 \Delta P_t = \sum_4^6 \Delta P_t$$

NOTE. This is the design method of Hobst & Zajic (1977) for stabilizing soil slopes.

(b) Tangential forces

Figure 10. (concluded)

It is probable that a system of drainage will also be installed in conjunction with the ground anchorages (Hoek & Bray 1977). The performance of the anchored slope should be monitored (see 9.5 and M.4). A more detailed discussion of design considerations is given by Mitchell*.

4.1.5 *Stability of slopes in rock*

4.1.5.1 *Fundamental considerations.* The use of anchorages to improve the stability of existing slopes or allow steeper slopes to be excavated is well established (see figure 11). Fundamental considerations listed for soil slopes (see 4.1.4.1) generally apply in rock even though different failure modes occur (see figure 12).

Other aspects that require additional considerations in assessing the overall stability are as follows:

- (a) effect of discontinuities on slope stability;
- (b) stress/deformation behaviour of the rock mass;
- (c) influence of excavation methods on rock mass strength, e.g. blasting damage.

Hoek & Bray (1977) provide assistance in dealing with many of the above aspects and those recommended in 4.1.4.1.

4.1.5.2 *Methods of analysis.* The main types of failure likely to be encountered in rock cuts and slopes are shown in figure 12. In design, the following mechanisms are well recognized:

- (a) rotational failures through weak rock;
- (b) planar failures along a single discontinuity or set;
- (c) wedge failures along intersecting discontinuities;
- (d) failures along weak seams or shear zones;
- (e) toppling failures in rocks with steeply dipping discontinuities.

Analytical procedures and methods are typically given in figures 13 and 14 and include the following:

- (f) circular failure surfaces (Bishop simplified method of slices) or stability charts (Hoek & Bray 1977);
- (g) non-circular failure surfaces (Janbu 1957) and (Morgenstern & Price 1965);
- (h) planar and wedge analysis using stereographic projection methods (Kovari & Fritz 1976) and (Hoek & Bray 1977) or direct calculation; the method of analysis shown on figure 15 may be used when a planar surface represents the most likely failure plane.

All methods of analysis should be carried out in terms of effective stresses after a careful assessment of likely ground water conditions, with or without drainage. Field data should be used from simple permeability tests and piezometric observations (Sharp et al 1978).

The failure modes listed in (a) to (e) cover most commonly encountered situations. Rocks can, however, exhibit time dependent effects, such as weathering, or strength properties may change as a result of changing stress/deformation conditions during excavation. Progressive, i.e. time dependent, failure may also occur. Although such effects are not amenable to detailed analysis, an assessment of the potential change in properties and the influence in stability should be made.

*In course of preparation.

Under certain circumstances, it may be necessary to analyse the stress and deformation conditions in the outer slope to assess the likely changes to be induced by the anchorage system. The finite element approach is particularly suitable for this purpose, although simpler boundary element methods may often be adequate (Hocking et al 1976).

4.1.5.3 *Design considerations*

4.1.5.3.1 *Slope failures.* In order to predict the likely failure mode, a careful assessment of the geological and structural data should be presented in the form of a stereoplot as shown in figure 14. The use of stereonets requires careful interpretation of the observed structural data, particularly in relation to the continuity and spacing of the discontinuities.

The strength of the discontinuities along which failure may occur has to be determined using either data from tests that are representative with respect to location, scale and stress state, or from empirical methods, such as those developed by Barton (1971) or Ladanyi & Archambault (1970). Because the shear strength of a discontinuity will be dependent on the stress state, it is important to use appropriate strength values at the effective normal stress, determined directly from the shear strength/normal stress envelope.

If the discontinuities are filled with clay or other weathered material, residual strength parameters should be adopted. Where large displacements have already occurred, cohesion should be taken as zero and the design based only on the frictional resistance of the discontinuities (Noble 1973), (Wagner et al 1975) and (Brandl 1976).

A sensitivity analysis should be carried out to assess the influence of each factor on the overall stability of the cut or slope. Fookes et al (1977) describe an example of this approach applied to a slope in a badly contorted slate.

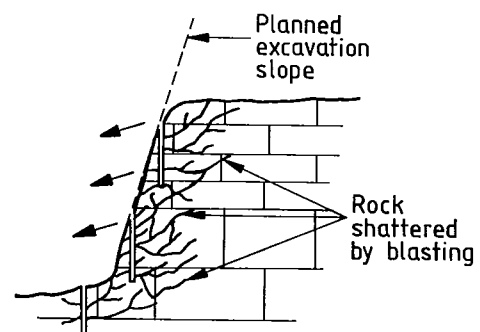
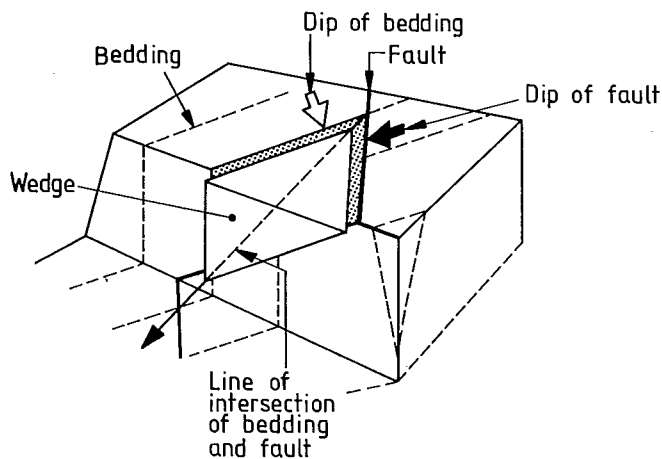
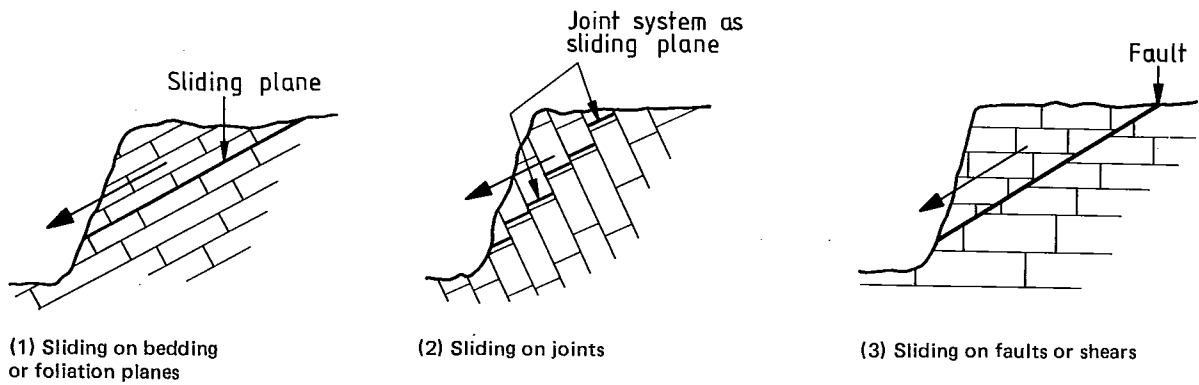
The adoption of an anchorage system as a stabilization measure should produce a structure with an adequate factor of safety. If the variability of the design parameters is significant, an extra margin of safety will be necessary.

The arrangement or layout of anchorages will be based on the geological structure but a uniform distribution is generally adopted. Where a critical failure plane exists within the slope, the fixed anchor length should be located beyond this plane (see figure 16).

It is not usual to utilize a face support system other than the faceplate itself, owing to the intrinsic strength of most rocks. Shotcrete, with or without mesh reinforcement, may be used to control spalling at minor rockfalls.

It is probable that drainage facilities will be installed in conjunction with the anchorages (Rawlings 1968), (Williams & Muir 1972), (Wagner et al 1975) and (Brandl 1976 and 1979). Typical methods of draining rock slopes are described by Hoek & Bray (1977).

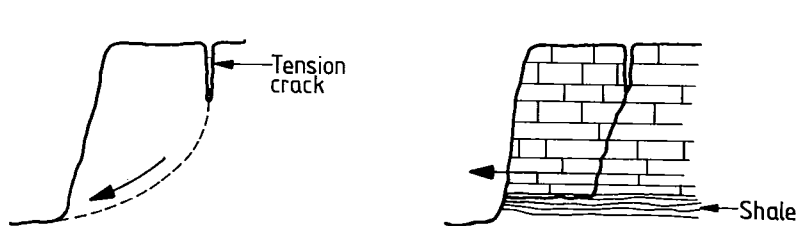
4.1.5.3.2 *Rockfalls.* In intensely jointed rocks, anchorages may be used to provide near-surface reinforcement to the rock mass and prevent ravelling. Normally, anchorages should be used with a rock retention system such as wire mesh and/or shotcrete. The depth of the anchorage may be quite shallow (2 m to 3 m), providing that overall slope stability, i.e. deeper seated failure, is not of concern.



(4) Two intersecting discontinuities may allow sliding of rock wedge

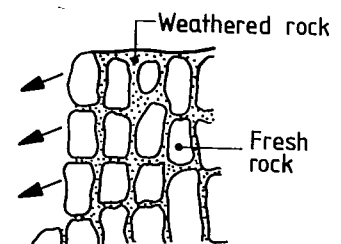
(5) Rock slope may be unstable due to fractures caused by blasting

(a) Discontinuities dipping out of a slope may allow sliding failure



(1) Rotational failure in weak rock, such as chalk

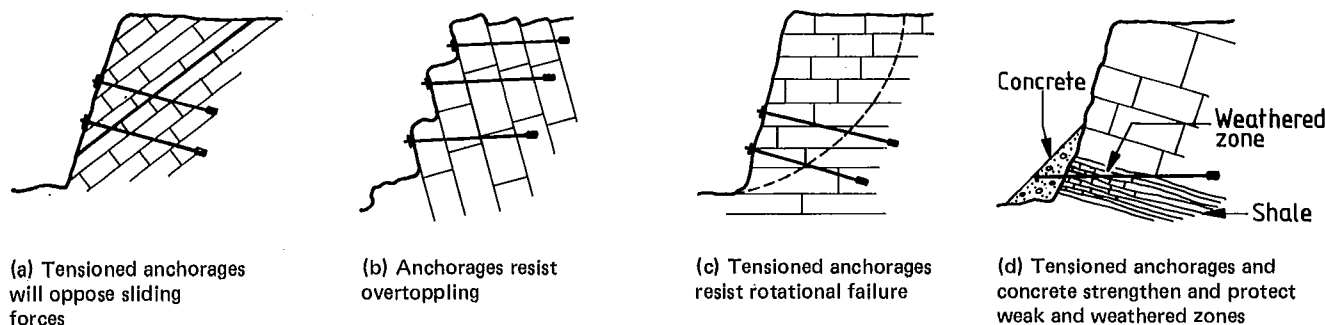
(2) Sliding failure on basal band of weak rock, such as shale



(3) Toppling failure of weathered igneous rocks, such as dolerite or granite

(b) Slopes may fail if the rock is too weak to support the weight of the slope

Figure 11. Slope instability



NOTE. Anchorages may improve slope stability.

Figure 12. Slope stabilization

The spacing of anchorages will depend on the nature of the face retention system but should typically be in the range of 1.5 m to 3.0 m. Anchorages may also be used to tie back shotcrete surfacing used to prevent weathering. Excavation methods for cuts and slopes may have a significant influence on anchorage or bolting requirements. Controlled blasting techniques should always be adopted for anchored slopes in order to prevent damage to the rock (see figure 12) and minimize the rockfall potential (Littlejohn et al 1977).

4.1.5.3.3 Tensioned and untensioned rock reinforcement.

Both tensioned (active) and untensioned fully bonded (passive) anchorages are commonly used. In judging the suitability of either type, the stress/deformation characteristics of the anchorage in relation to the rock mass strength/deformation behaviour should be carefully assessed.

In broad terms, deep anchorages will normally be tensioned and may subsequently be fully bonded for permanence, whilst short anchorages (≤ 5 m) can be either tensioned or untensioned.

The development of two speed resin systems for shorter anchorages provides the facility for permanent tensioning, as required, and full bonding.

Passive reinforcement bars should be used with faceplates to provide a degree of restraint to the rock face.

4.1.6 Permanent underground excavations in rock

4.1.6.1 Fundamental considerations. Anchorages in conjunction with shotcrete are widely used to stabilize underground openings in all classes of rocks, including weak, sheared and decomposed masses.

In tunnels and mine openings where there is a significant length of opening in a particular rock type, it is normal practice to design the anchorage system initially on the basis of the classification system (Barton 1974) and subsequently to modify the level of support based on observed conditions. The approach is thus largely empirical and observational. Often the anchorage arrangement will be temporary, based on the limited life of the opening as in mining or where there is subsequent placement of a permanent concrete lining as for water tunnels.

For large permanent underground openings, such as an underground power station or storage facility, a more specific design approach may be warranted. The site is amenable to detailed exploration, since it is of limited extent compared with tunnels and the need for long term security calls for a thorough approach, especially where rock reinforcement is used for permanent support.

Since the type of anchorage system varies widely according to the opening size, geology and stress state, detailed design methods are outside the scope of this Draft for Development but have been partially covered in references (see appendix P).

Design parameters that should be evaluated are as follows:

- geology: general distribution of rock types and degree of weathering;
- major structures: definition and investigation of faults, dykes and shears;
- discontinuities: mapping and representation of discontinuities likely to affect the stability of the underground opening;
- determination of the stress state;
- determination of the strength of the rock mass (intact and joint strengths);
- determination of rock mass deformability;
- determination of the potential ground water conditions.

4.1.6.2 Methods of analysis. Analysis of underground excavations is required for the following principal reasons:

- determination of stress conditions in relation to strength of rock mass by finite element and boundary element methods;
- determination of geometry of potential kinematic failures by stereographic projection or direct analysis based on structural mapping of openings (Croney et al 1978).

Careful appraisal of the results of such analyses in relation to the quality of the parameters is required.

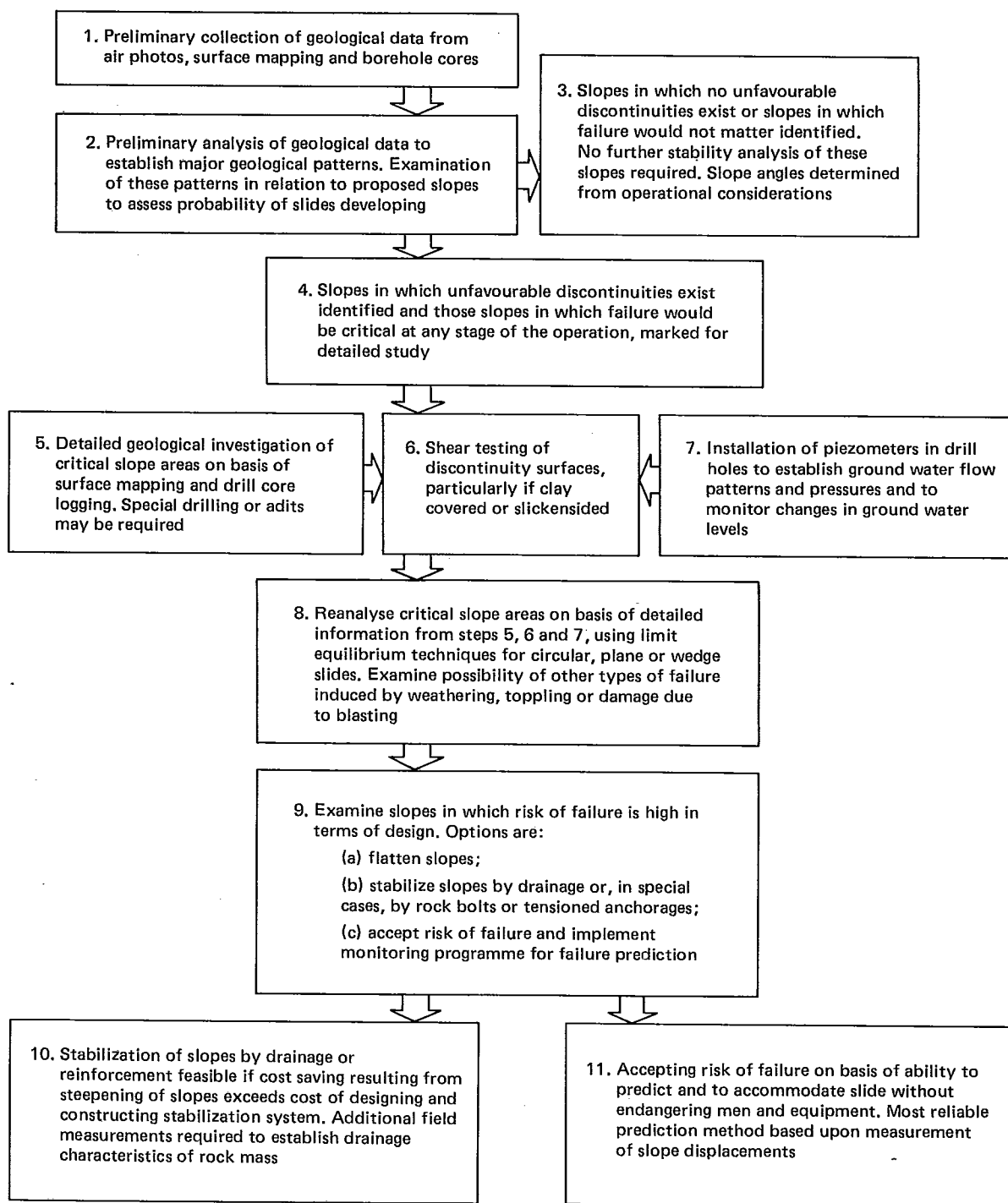
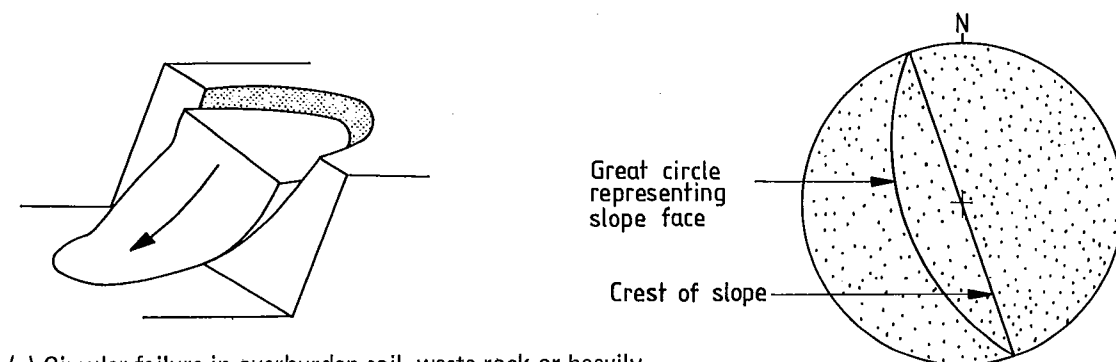
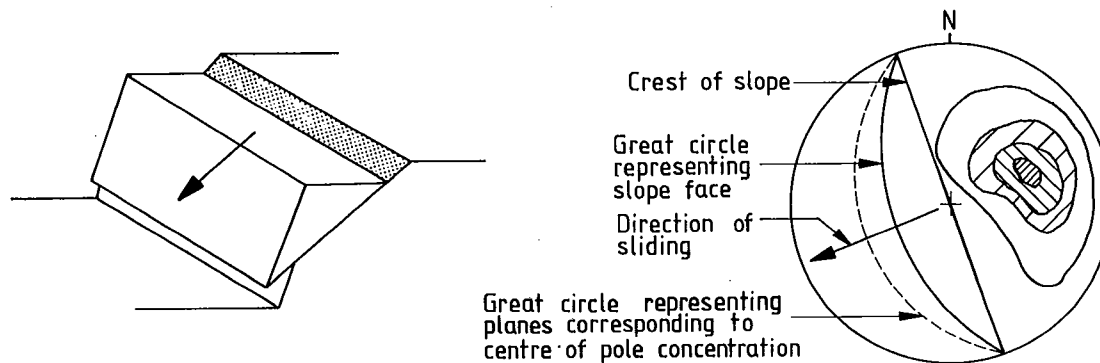


Figure 13. Procedure for analysing rock slopes (after Hoek & Bray (1977))

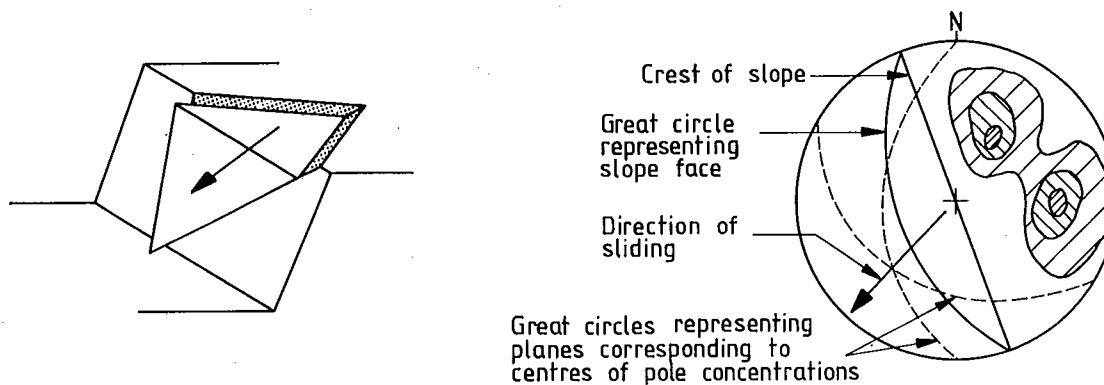
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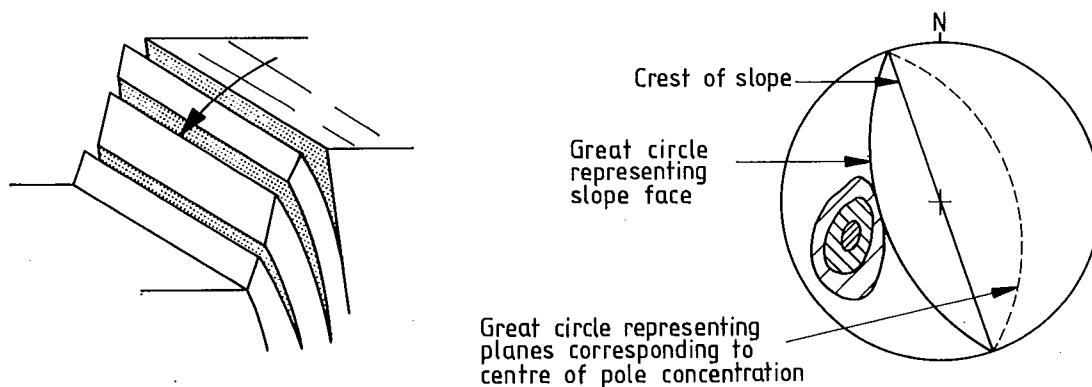
(a) Circular failure in overburden soil, waste rock or heavily fractured rock with no identifiable structural pattern



(b) Plane failure in rock with highly ordered structure such as slate

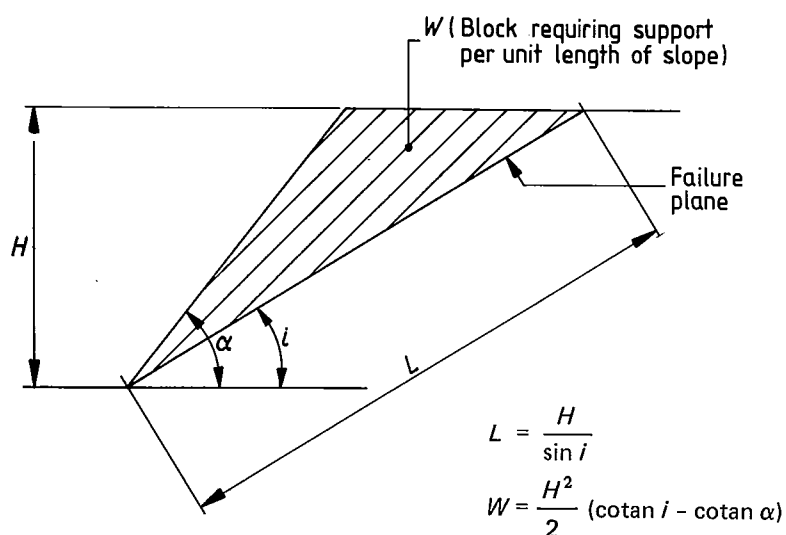


(c) Wedge failure on two intersecting discontinuities

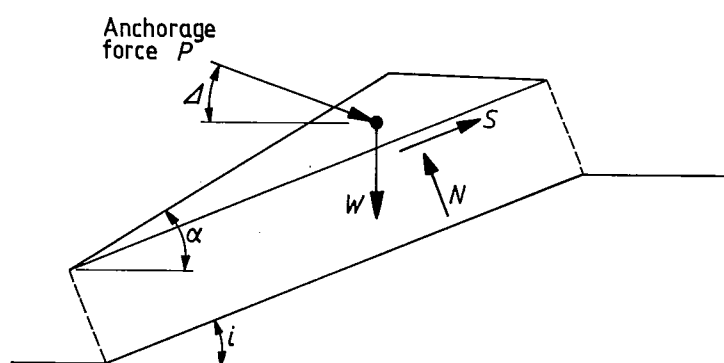


(d) Toppling failure in hard rock which can form columnar structure separated by steeply dipping discontinuities

Figure 14. Typical slope failure related to appearance of stereoplots of discontinuities
(after Hoek & Bray (1977))



(a) Block geometry



where

$$S = W \sin i - P \cos (i + \Delta)$$

$$N = W \cos i + P \sin (i + \Delta)$$

Factor of safety against sliding (S_f) is given by:

$$S_f = \frac{N \tan \phi'}{F}$$

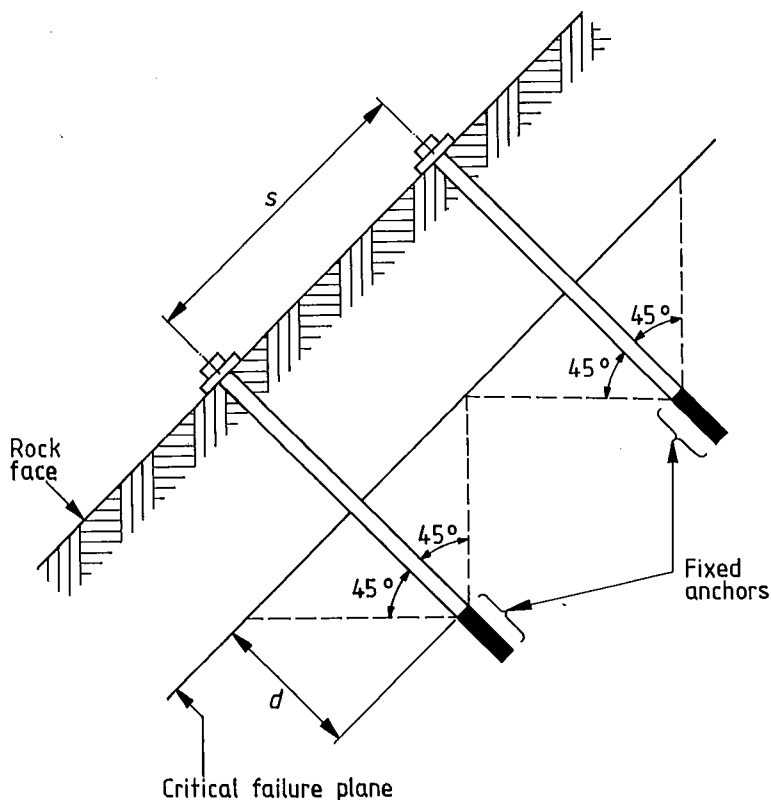
Anchorage force (P) is given by:

$$P = \frac{W (F \sin i - \cos i \tan \phi')}{F \cos (i + \Delta) + \sin (i + \Delta) \tan \phi'}$$

where

ϕ' is the angle of shearing resistance on failure plane

Figure 15. Simple stability analysis of excavations on slopes in rock (after Sharp (1973))



$$d \approx 0.5 \times s$$

Figure 16. Depth of fixed anchor below critical failure plane
(after Sharp (1973))

4.1.6.3 Design considerations. The design of anchorage systems requires the careful appraisal of each of the following elements:

- (a) review of practice and experience;
- (b) reinforcement and monitoring of trial openings, particularly in weak rock masses with limited precedent;
- (c) reinforcement of specific structurally controlled zones or blocks;
- (d) reinforcement of overstressed zones of the rock mass;
- (e) size, length and spacing of reinforcement members;
- (f) three-dimensional arrangement of openings including proximity;
- (g) most efficient excavation profile with respect to geology and structure;
- (h) intersection of openings.

The practical arrangements of anchorage installations are extremely important in underground design and the following aspects should be carefully assessed:

- (i) type of anchorage system (resin anchored rockbolts usually preferred for speed and simplicity);
- (j) use of tensioned or untensioned reinforcement (tensioned usually preferred);

(k) timing of reinforcement (usually immediately after excavation unless staged excavation with pre-reinforcement is adopted);

(l) integration of reinforcement with face support system (faceplate/mesh/shotcrete);

(m) quality control and prevention of damage by adjacent excavation stages.

Monitoring is required to provide a comprehensive check of excavation behaviour and compare observed versus predicted movements.

Where variations are significant, modifications to the anchorage system may be necessary.

4.1.7 Stabilization of new and existing structures in soil and rock

4.1.7.1 Fundamental considerations. Anchorages may be employed to strengthen existing buildings or new structures and to stabilize walls that are near collapse due to earth pressure forces that they cannot resist (Wolf & Hensen 1963), (Williams 1974) and (Anon 1977).

With regard to existing structures, it is advisable to understand how they behave before the anchorages are installed. Copies of the original drawings should be obtained, if possible, or a site investigation carried out to determine the geometry of the structure or wall.

An assessment should also be made of the strength of the existing fabric of the structure with regard to the additional loads induced by the installation of anchorages.

4.1.7.2 Methods of analysis. References should be made to 4.1.2.2, 4.1.4.2 and 4.1.8.2 for soils, and 4.1.3.2, 4.1.5.2 and 4.1.9.2 for rocks.

4.1.7.3 Design considerations. Reference should be made to 4.1.2.3, 4.1.4.3 and 4.1.8.3 for soils and 4.1.3.3, 4.1.5.3 and 4.1.9.3 for rocks.

4.1.8 Security of structures subject to vertical and transitory loads in soil

4.1.8.1 Fundamental considerations. At the time of publication of this Draft for Development, it is not possible to predict theoretically the geometry of the failure zone for vertical or inclined grouted anchorages installed in soil (see C.2).

Research using model plates or full scale test on cylinders or single underreams has suggested an empirical design method for both cohesive and cohesionless soils (Adams & Hayes 1967), (Meyerhof & Adams 1968), (Adams & Radhakrishna 1971), (Adams & Klym 1972) and (Radhakrishna & Adams 1973).

Current methods assume that a cylindrical failure surface is formed. The detailed shape of the failure surface for anchorages with several enlargements or underream is not known, although model test results using radiographs are available (Swain 1976).

Cyclic, repeated, or pulsating loadings should also be considered, but there are no accepted design proposals at the time of publication of this Draft for Development. With regard to this, research using untensioned model plates has shown that once the plate begins to yield it will do so at an increasing rate until failure (Bemben & Kupferman 1975), (Andreadis et al 1978) and (Hanna et al 1978).

However, many structures subject to such loads and restrained by tensioned grouted anchorages have performed satisfactorily (Huder 1969), (Littlejohn 1970), (Betts & Welsh 1972), (Von Soos 1972) and (Feld & White 1974).

4.1.8.2 Methods of analysis. For deep anchorages, a cylindrical failure surface local to the individual fixed anchors should be assumed and the ultimate resistance to withdrawal at the ground/grout interface should be estimated in accordance with 4.2. In this context, field experience with downward inclined anchorages indicates that general failure (see 4.2.1) does not occur for slenderness ratios (H/D) in excess of 15, where H is the depth to the top of the fixed anchor and D is the diameter of the fixed anchor. For the small diameters involved, the top of the fixed anchor is usually founded at depths in excess of 5 m (see figure 3). Figure 17 shows methods of analysis for groups of anchorages in cohesionless soil.

For shallow anchorages, e.g. deadman tying back retaining walls, reference should be made to Civil Engineering Code of Practice No 2 (1951).

Table 4. Reducing coefficient (m)
(after Trofimenkov & Mariupolskii (1965))

Soil type	Reducing coefficient (m)		
	Load increasing by steps	Pulsating load	Alternating load
Hard, very stiff and stiff clays	0.8	0.7	0.7
Medium clays	0.8	0.7	0.6
Soft clays	0.8	0.6	0.4
Very soft clays	0.8	0.5	0.3
Partly saturated sands	0.8	0.7	0.7
Saturated and dry sands	0.8	0.5	0.3

4.1.8.3 Design considerations. Allowance should be made for a reduction in anchorage load carrying capacity with time if a passive untensioned anchorage is subject to transitory loading. For passive anchorages, table 4 may be used to account for pulsating, alternating or increasing loads, by multiplying the ultimate capacity of the anchorage by the reduction coefficient m (Trofimenkov & Mariupolskii 1965).

Prestressing the anchorage appears to make the anchorage system stiff (Hanna et al 1972). The anchorage system should be adequate if no displacement takes place for each cycle of load. If displacement of the fixed anchor length does occur, then the possibility of a sudden failure exists (Hanna et al 1978).

The prestressing force in the anchorage should therefore be chosen to be greater than any likely load fluctuation (Littlejohn 1977).

4.1.9 Security of structures subject to vertical and transitory loads in rock

4.1.9.1 Fundamental considerations. At the time of publication of this Draft for Development, it is not possible to predict theoretically the geometry of the failure zone for vertical or inclined anchorages installed in rock (see C.2).

Details of empirical methods of analysis are given in 4.1.9.2, but there is little experimental or practical evidence or theoretical data to substantiate the methods (Littlejohn & Bruce 1977). In general, however, current empirical methods are conservative (Bruce 1976).

4.1.9.2 Methods of analysis. An inverted cone with the geometry shown in figure 18 should be assumed to be mobilized at failure of the anchorage.