Chapter 2 : Basic mechanics of slope failure

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Continuum mechanics approach to slope stability

A question which frequently arises in discussions on slope stability is how high and how steep can a rock slope be cut. One approach to this problem, which has been adopted by a number of investigators[11-15], is to assume that the rock mass behaves as an elastic continuum. The success which has been achieved by the application of techniques such as photoelastic stress analysis or finite element methods in the design of underground excavations has tempted many research workers to apply the same techniques to slopes. Indeed, from the research point of view, the results have been very interesting but in terms of practical rock slope engineering, these methods have limited usefulness. These limitations arise because our knowledge of the mechanical properties of rock masses is so inadequate that the choice of material properties for use in the analysis becomes a matter of pure guesswork. For example, if one attempts to calculate the limiting vertical height of a slope in a very soft limestone on the basis of its intact strength, a value in excess of 3500 feet is obtained[16]. Clearly, this height bears very little relation to reality and one would have to reduce the strength properties by a factor of at least 10 in order to arrive at a reasonable slope height.

It is appropriate to quote from a paper by Terzaghi[17] where, in discussing the problem of foundation and slope stability, he said "..... natural conditions may preclude the possibility of securing all the data required for predicting the performance of a real foundation material by analytical or any other methods. If a stability computation is required under these conditions, it is necessarily based on assumptions which have little in common with reality. Such computations do more harm than good because they divert the designer's attention from the inevitable but important gaps in his knowledge....".

Muller[13] and his co-workers in Europe have emphasised for many years the fact that a rock mass is not a continuum and that its behaviour is dominated by discontinuities such as faults, joints and bedding planes. Most practical rock slope designs are currently based upon this discontinuum approach and this will be the approach adopted in all the techniques presented in this book. However, before leaving the question of the continuum mechanics approach, the authors wish to emphasise that they are not opposed in principle to its application and indeed, when one is concerned with overall displacement or groundwater flow patterns, the results obtained from a numerical method such as the finite element technique can be very useful. Developments in numerical methods such as those reported by Goodman et al[19] and Cundall[20] show that the gap between the idealised elastic continuum and the real discontinuum is gradually being bridged and the authors are optimistic that the techniques which are currently interesting research methods will eventually become useful engineering design tools.

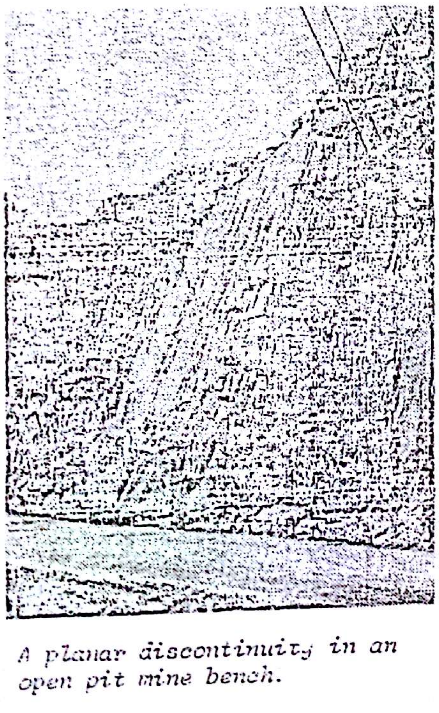
Maximum slope height - slope angle relationship for excavated slopes

Even if one accepts that the stability of a rock mass is dominated by geological discontinuities, there must be situations where the orientation and inclination of these discontinuities is such that simple sliding of slabs, blocks or wedges is not possible. Failure in these slopes will involve a combination of movement on discontinuities and failure of intact rock material and one would anticipate that, in such cases, higher and steeper slopes than average could be excavated. What practical evidence is there that this is a reasonable assumption?

A very important collection of data on excavated slopes was compiled by Kley and Lutton[21] and additional data have been obtained by Ross-Brown[22]. The information refers to slopes in opencast mines, quarries, dam foundation excavations and highway cuts. The slope heights and corresponding slope angles for the slopes in materials classified as hard rock have been plotted in Figure 7 which includes both stable and unstable slopes. Ignoring, for the moment, the unstable slopes, this plot shows that the highest and steepest slopes which have been successfully excavated, as far as is known from this collection of data, fall along a fairly clear line shown dashed in Figure 7. This line gives a useful practical guide to the highest and steepest slopes which can be contemplated for normal open pit mine planning. In some exceptional circumstances, higher or steeper slopes may be feasible but these could only be justified if a very comprehensive stability study had shown that there was no risk of inducing a massive slope failure.

Role of discontinuities in slope failure

Figure 7 shows that, while many slopes are stable at steep angles and at heights of several hundreds of feet, many flat slopes fail at heights of only tens of feet. This difference is due to the fact that the stability of rock slopes varies with inclination of discontinuity surfaces, such as faults, joints and bedding planes, within the rock mass. When these discontinuities are vertical or horizontal, simple sliding cannot take place and the slope failure will involve fracture of intact blocks of rock as well as movement along some of the discontinuities. On the other hand, when the rock mass contains discontinuity surfaces dipping towards the slope face at angles of between 30° and 70°, simple sliding can occur and the stability of these slopes is significantly lower than those in which only horizontal and vertical discontinuities are present.



A planar discontinuity in an open pit mine bench.

The influence of the inclination of a failure plane on the stability of a slope is strikingly illustrated in Figure 8 in which the critical height of a dry rock slope is plotted against discontinuity angle. In deriving this curve, it has been assumed that only one set of discontinuities is present in a very hard rock mass and that one of these discontinuities "daylights" at the toe of the vertical slope as shown in the sketch in Figure 8. It will be seen that the critical vertical height H decreases from a value in excess of 200 feet, for vertical and horizontal discontinuities, to about 70 feet for a discontinuity inclination of 55°.

Clearly the presence, or absence, of discontinuities has a very important influence upon the stability of rock slopes and the detection of these geological features is one of the most critical parts of a stability investigation. Techniques for dealing with this problem are discussed in later chapters of this book.

Friction, cohesion and unit weight

The material properties which are most relevant to the discussion on slope stability presented in this book are the angle of friction, the cohesive strength and the unit weight of the rock and soil masses.

Friction and cohesion are best defined in terms of the plot of shear versus normal stress given in Figure 9. This plot is a simplified version of the results which would be obtained if a rock specimen containing a geological discontinuity such as a joint is subjected to a loading system which causes sliding along the discontinuity. The shear stress τ required to cause sliding increases with increasing normal stress σ. The slope of the line relating shear to normal stress defines the angle of friction ϕ. If the discontinuity surface is initially cemented or if it is rough, a finite value of shear stress τ will be required to cause sliding when the normal stress level is zero. This initial value of shear strength defines the cohesive strength c of the surface.

The relationship between shear and normal stresses for a typical rock surface or for a soil sample can be expressed as :

$$\boxed{\tau = c + \sigma \tan\phi} \qquad (1)$$

c = cohesive strength
τ = shear stress
ϕ = Friction angle
σ = Normal stress

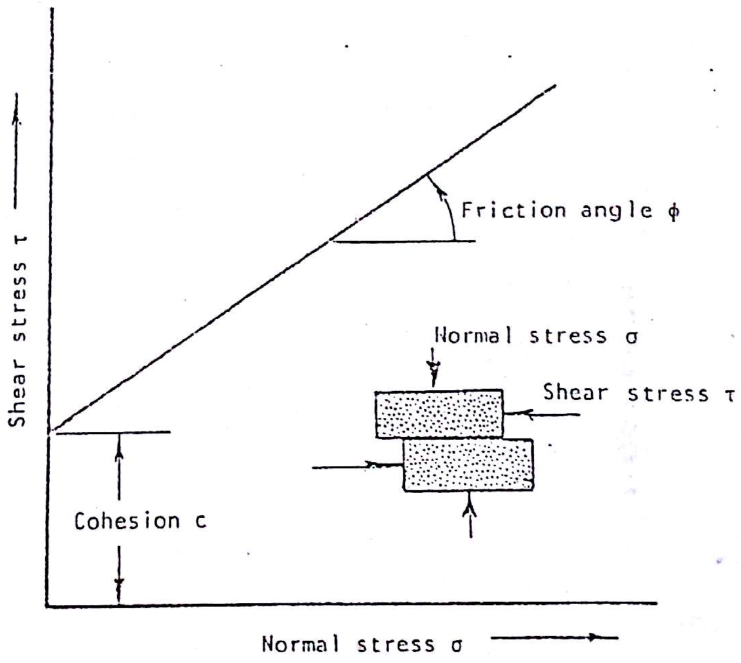


Figure 9 : Relationship between the shear stress τ required to cause sliding along a discontinuity and the normal stress σ acting across it.

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TABLE 1 - TYPICAL SOIL AND ROCK PROPERTIES

Description			Unit weight (Saturated/dry)		Friction angle	Cohesion	
Type		*Material*	*lb/ft*^{3}	*kN/m*^{3}	*degrees*	*lb/ft*^{2}	*kPa*
Cohesionless	*Sand*	Loose sand, uniform grain size	118/90	19/14	28-34*		
Cohesionless	*Sand*	Dense sand, uniform grain size	130/109	21/17	32-40*		
Cohesionless	*Sand*	Loose sand, mixed grain size	124/99	20/16	34-40*		
Cohesionless	*Sand*	Dense sand, mixed grain size	135/116	21/18	38-46*		
Cohesionless	*Gravel*	Gravel, uniform grain size	140/130	22/20	34-37*		
Cohesionless	*Gravel*	Sand and gravel, mixed grain size	120/110	19/17	48-45*		
Cohesionless	*Blasted/broken rock*	Basalt	140/110	22/17	40-50*		
Cohesionless	*Blasted/broken rock*	Chalk	80/62	13/10	30-40*		
Cohesionless	*Blasted/broken rock*	Granite	125/110	20/17	45-50*		
Cohesionless	*Blasted/broken rock*	Limestone	120/100	19/16	35-40*		
Cohesionless	*Blasted/broken rock*	Sandstone	110/80	17/13	35-45*		
Cohesionless	*Blasted/broken rock*	Shale	125/100	20/16	30-35*		
Cohesive	*Clay*	Soft bentonite	80/30	13/6	7-13	200-400	10-20
Cohesive	*Clay*	Very soft organic clay	90/40	14/6	12-16	200-600	10-30
Cohesive	*Clay*	Soft, slightly organic clay	100/60	16/10	22-27	400-1000	20-50
Cohesive	*Clay*	Soft glacial clay	110/76	17/12	27-32	600-1500	30-70
Cohesive	*Clay*	Stiff glacial clay	130/105	20/17	30-32	1500-3000	70-150
Cohesive	*Clay*	Glacial till, mixed grain size	145/130	23/20	32-35	3000-5000	150-250
Cohesive	*Rock*	Hard igneous rocks - granite, basalt, porphyry	** 160 to 190	25 to 30	35-45	720000-1150000	35000-55000
Cohesive	*Rock*	Metamorphic rocks - quartzite, gneiss, slate	160 to 180	25 to 28	30-40	400000-800000	20000-40000
Cohesive	*Rock*	Hard sedimentary rocks - limestone, dolomite, sandstone	150 to 180	23 to 28	35-45	200000-600000	10000-30000
Cohesive	*Rock*	Soft sedimentary rock - sandstone, coal, chalk, shale	110 to 150	17 to 23	25-35	20000-400000	1000-20000

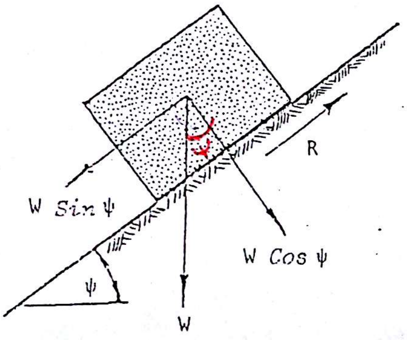
* Higher friction angles in cohesionless materials occur at ~~low confining or~~ normal stresses as discussed in Chapter 5.

** For intact rock, the unit weight of the material does not vary significantly between saturated and dry states with the exception of materials such as porous sandstones.

Typical values for the angle of friction and cohesion which are found in shear tests on a range of rocks and soils are listed in Table 1 together with unit weights for these materials. The values quoted in this table are intended to give the reader some idea of the magnitudes which can be expected and they should only be used for obtaining preliminary estimates of the stability of a slope.

There are many factors which cause the shear strength of a rock or soil to deviate from the simple linear dependence upon normal stress illustrated in Figure 9. These variations, together with methods of shear testing, are discussed in Chapter 5.

Sliding due to gravitational loading



Consider a block of weight W resting on a plane surface which is inclined at an angle ψ to the horizontal. The block is acted upon by gravity only and hence the weight W acts vertically downwards as shown in the margin sketch. The resolved part of W which acts down the plane and which tends to cause the block to slide is W Sin ψ. The component of W which acts across the plane and which tends to stabilise the slope is W Cos ψ.

The normal stress σ which acts across the potential sliding surface is given by

$$\boxed{\sigma = (W \cos\psi)/A} \qquad (2)$$

where A is the base area of the block.

Assuming that the shear strength of this surface is defined by equation (1) and substituting for the normal stress from equation (2)

$$\tau = c + \frac{W \cos\psi}{A} \cdot \tan\phi$$

$$\text{or} \quad R = cA + W \cos\psi \cdot \tan\phi \qquad (3)$$

Resisting F

where $\boxed{R = \tau A}$ is the shear *force* which resists sliding down the plane.

R = shear force

The block will be just on the point of sliding or in a condition of *limiting equilibrium* when the disturbing force acting down the plane is exactly equal to the resisting force :

Disturbing F = Resisting F

$$W \sin\psi = cA + W \cos\psi \cdot \tan\phi \qquad (4)$$

If the cohesion c = 0, the condition of limiting equilibrium defined by equation (4) simplifies to

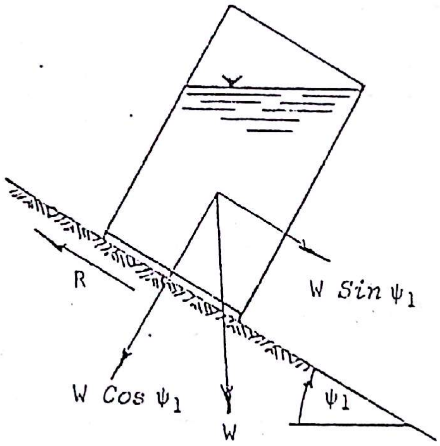
$$\boxed{\psi = \phi} \qquad (5)$$

Influence of water pressure on shear strength

The influence of water pressure upon the shear strength of two surfaces in contact can most effectively be demonstrated by the beer can experiment.

An opened beer can filled with water rests on an inclined piece of wood as shown in the margin sketch on the next page.

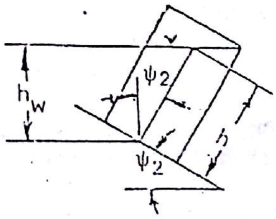
The forces which act in this case are precisely the same as those acting on the block of rock as shown in the diagram on the previous page. For simplicity the cohesion between the beer can base and the wood is assumed to be zero. According to equation (5) the can with its contents of water will slide down the plank when $\psi_1 = \phi$.



The base of the can is now punctured so that water can enter the gap between the base and the plank, giving rise to a water pressure u or to an uplift force U = uA, where A is the base area of the can.

The normal force W Cos ψ_2 is now reduced by this uplift force U and the resistance to sliding is now

$$R = (W \text{ Cos } \psi_2 - U) \text{ Tan } \phi \qquad (6)$$



If the weight per unit volume of the can plus water is defined as γ_t while the weight per unit volume of the water is γ_w, then $W = \gamma_t . h . A$ and $U = \gamma_w . h_w . A$, where h and h_w are the heights defined in the small sketch. From this sketch it will be seen that $h_w = h . \text{Cos } \psi_2$ and hence

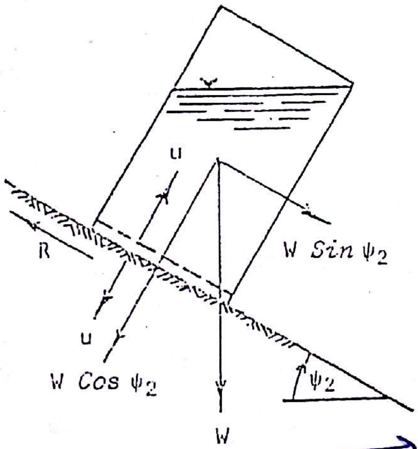
$$U = {}^{\gamma_w}/_{\gamma_t} . W \text{ Cos } \psi_2 \qquad (7)$$

Substituting in (6)

$$R = W \text{ Cos } \psi_2 \; (1 - {}^{\gamma_w}/_{\gamma_t}) \text{ Tan } \phi \qquad (8)$$

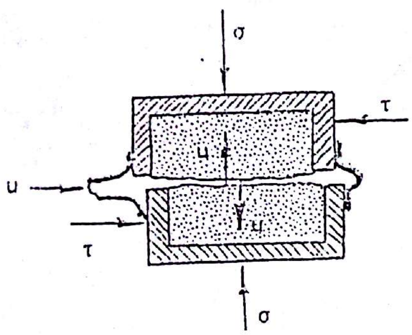
and the condition for limiting equilibrium defined in equation (4) becomes

$$\text{Tan } \psi_2 = (1 - {}^{\gamma_w}/_{\gamma_t}) \text{ Tan } \phi \qquad (9)$$



Assuming the friction angle of the can/wood interface is 30°, the unpunctured can will slide when the plane is inclined at $\psi_1 = 30°$ (from equation (5)). On the other hand, the punctured can will slide at a much smaller inclination because the uplift force U has reduced the normal force and hence reduced the frictional resistance to sliding. The total weight of the can plus water is only slightly greater than the weight of the water. Assuming $\gamma w/\gamma t = 0.9$ and $\phi = 30°$, equation (9) shows that the punctured can will slide when the plane is inclined at $\psi_2 = 3° \, 18'$.

The effective stress law



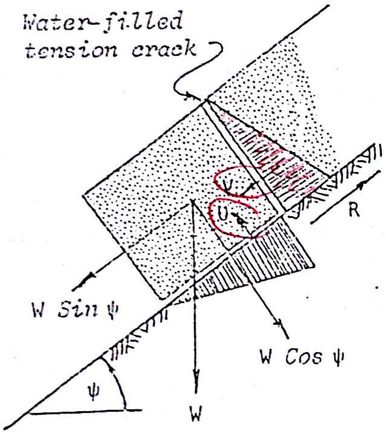
The effect of water pressure on the base of the punctured beer can is the same as the influence of water pressure acting on the surfaces of a shear specimen as illustrated in the margin sketch. The normal stress σ acting across the failure surface is reduced to the *effective stress* (σ - u) by the water pressure u. The relationship between shear strength and normal strength defined by equation (1) now becomes

$$\boxed{\tau = c + (\sigma - u) \text{ Tan } \phi} \qquad (10)$$

In most hard rocks and in many sandy soils and gravels, the cohesive and frictional properties (c and ϕ) of the materials are not significantly altered by the presence of water and hence, reduction in shear strength of these materials is due, almost entirely to the reduction of normal stress across failure surfaces. Consequently, it is *water pressure* rather than *moisture content* which is important in defining the strength characteristics of hard rocks, sands and gravels. In terms of the stability of slopes in these materials, the presence of a small volume of water at high pressure, trapped within the rock mass, is more important than a large volume of water discharging from a free draining aquifer.

In the case of soft rocks such as mudstones and shales and also in the case of clays, both cohesion and friction can change markedly with changes in moisture content and it is necessary, when testing these materials, to ensure that the moisture content of the material during test is as close as possible to that which exists in the field. Note that the effective stress law defined in equation (10) still applies to these materials but that, in addition, c and ϕ change.

The effect of water pressure in a tension crack



Consider the case of the block resting on the inclined plane but, in this instance, assume that the block is split by a tension crack which is filled with water. The water pressure in the tension crack increases linearly with depth and a total force V, due to this water pressure acting on the rear face of the block, acts down the inclined plane. Assuming that the water pressure is transmitted across the intersection of the tension crack and the base of the block, the water pressure distribution illustrated in the margin sketch occurs along the base of the block. This water pressure distribution results in an uplift force U which reduces the normal force acting across this surface.

The condition of limiting equilibrium for this case of a block acted upon by water forces V and U in addition to its own weight W is defined by

$$\boxed{W \sin\psi + V = cA + (W \cos\psi - U)\tan\phi} \qquad (11)$$

V = disturbing force
U = ~~Restoring force~~ reducing weight

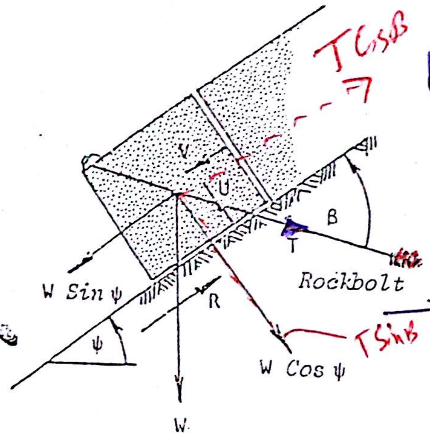
From this equation it will be seen that the disturbing force tending to induce sliding down the plane is increased and the frictional force resisting sliding is decreased and hence, both V and U result in decreases in stability. Although the water *pressures* involved are relatively small, these pressures act over large areas and hence the water *forces* can be very large. In many of the practical examples considered in later chapters, the presence of water in the slope giving rise to uplift forces and water forces in tension cracks is found to be critical in controlling the stability of slopes.

Reinforcement to prevent sliding.

One of the most effective means of stabilising blocks or slabs of rock which are likely to slide down inclined discontinuity surfaces is to install tensioned rockbolts or cables. Consider the block resting on the inclined plane and acted upon by the uplift force U and the force V due to water pressure in the tension crack. A rockbolt, tensioned to a load T is installed at an angle β to the plane as shown. The resolved component of the bolt tension T acting parallel to the plane is T Cos β while the component acting across the surface upon which the block rests is T Sin β. The condition of limiting equilibrium for this case is defined by

Disturbing = Resisting

$$W \sin\psi + V - T\cos\beta = cA + (W\cos\psi - U + T\sin\beta)\tan\phi \qquad (12)$$



This equation shows that the bolt tension reduces the disturbing force acting down the plane and increases the normal force and hence the frictional resistance between the base of the block and the plane.

Factor of safety of a slope

All the equations defining the stability of a block on an inclined plane have been presented for the condition of *limiting equilibrium*, i.e. the condition at which the forces tending to induce sliding are exactly balanced by those resisting sliding. In order to compare the stability of slopes under conditions other than those of limiting equilibrium, some form of index is required and the most commonly used index is the *Factor of Safety*. This can be defined as the ratio of the total force available to resist sliding to the total force tending to induce sliding. Considering the case of the block acted upon by water forces and stabilised by a tensioned rockbolt (equation 12), the factor of safety is given by

$$F = \frac{\text{Resisting } F}{\text{Disturbing } F}$$

$$F = \frac{cA + (W\cos\psi - U + T\sin\beta)\tan\phi}{W\sin\psi + V - T\cos\beta} \qquad (13)$$

When the slope is on the point of failure, a condition of limiting equilibrium exists in which the resisting and disturbing forces are equal, as defined by equation 12, and the factor of safety F = 1. When the slope is stable, the resisting forces are greater than the disturbing forces and the value of the factor of safety will be greater than unity.

Suppose that, in a practical mining situation, the observed behaviour of a slope suggests that it is on the point of failure and it is decided to attempt to stabilise the slope. Equation 14 shows that the value of the factor of safety can be increased by reducing both U and V, by drainage, or by increasing the value of T by installing rockbolts or tensioned cables. It is also possible to change the weight W of the failing mass but the influence of this change on the factor of safety must be carefully evaluated since both the disturbing and resisting forces are decreased by a decrease in W.

The bolt tension required to provide a specified factor of safety of F is a minimum when the angle β satisfies the equation

$$\boxed{\tan\beta = \frac{1}{F}\tan\phi} \qquad (14)$$

This result is obtained by differentiating equation (13) with respect to β, and setting $\frac{dT}{d\beta} = 0$ and $\frac{dF}{d\beta} = 0$.

Practical experience suggests that, in a situation such as that described above, an increase in the factor of safety from 1.0 to 1.3 will generally be adequate for mine slopes which are not required to remain stable for long periods of time. For critical slopes adjacent to haul roads or important installations, a factor of safety of 1.5 is usually preferred.

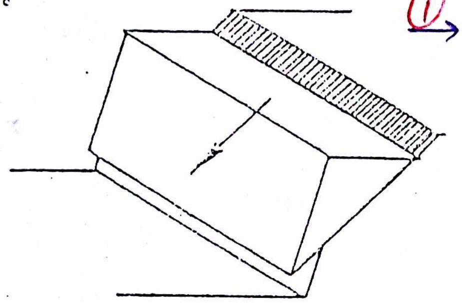
This example has been quoted because it emphasises the fact that the factor of safety is an index which is most valuable as a design tool when used on a *comparative* basis. In this case, the mine engineers and management have decided, on the basis of the observed behaviour of the slope, that a condition of instability exists and that the value of the factor of safety is 1.0. If remedial measures are taken, their effect can be measured against the condition of slope failure by calculating the increase in the factor of safety. Hoek and Londe, in a general review of rock slope and foundation design methods[23], conclude that the information which is most useful to the design engineer is that which indicates the response of the structure to changes in significant parameters. Hence, decisions on remedial measures such as drainage can be based upon the *rate of change* of the factor of safety, even if the absolute value of the calculated factor of safety cannot be relied upon with a high degree of certainty. To quote from this general review : "The function of the design engineer is not to compute accurately but to judge soundly."

In carrying out a feasibility study for a proposed open pit mine or civil engineering project, the geotechnical engineer frequently is faced with the task of designing slopes where none have previously existed. In this case there is no background experience of slope behaviour which can be used as a basis for comparison. The engineer may compute a factor of safety of 1.3 for a particular slope design, based upon the data available to him, but he has no idea whether this value represents an adequately stable slope since he has not had the opportunity of observing the behaviour of actual slopes in this particular rock mass. Under these circumstances, the engineer is well advised to exercise caution in the choice of the parameters used in the factor of safety calculation. Conservatively low values of both cohesion and friction should be used and, if the groundwater conditions in the slope are unknown, the highest anticipated groundwater levels should be used in the calculation. Sensitivity analyses of the effects of drainage and rock-bolting can still be carried out as in the previous case but, having chosen conservative rock strength parameters, the slope designer is unlikely to be faced with unpleasant surprises when the slope is excavated.

~~In later chapters of this book, a number of practical examples are given to illustrate the various types of rock slope design which are likely to be encountered by the reader. The problems of obtaining rock strength values, rock structure data and groundwater conditions for use in factor of safety calculations are discussed in these examples and guidance is given on the values of the factor of safety which are appropriate for each type of design.~~

Slope failures for which factors of safety can be calculated

In discussing the basic mechanism of slope failure, the model of a single block of rock sliding down an inclined plane has been used. This is the simplest possible model of rock slope failure and, in most practical cases, a more complex failure process has to be considered. In some cases, the methods of calculating the factor of safety, presented in this book, cannot be used because the failure process does not involve simple gravitational sliding. These cases will be discussed later in this chapter. The method of limiting equilibrium can be used in analysing the slope failures listed below.

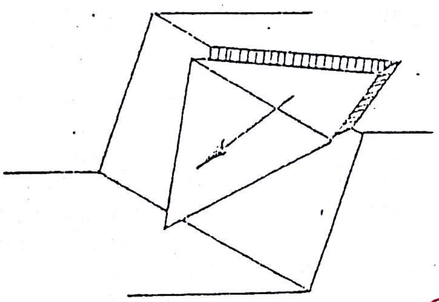


1

Plane failure

As shown in the margin sketch, plane failure occurs when a geological discontinuity, such as a bedding plane, strikes parallel to the slope face and dips into the excavation at an angle greater than the angle of friction. The calculation of the factor of safety follows precisely the same pattern as that used for the single block (equation 14). The base area A and the weight W of the sliding mass are calculated from the geometry of the slope and failure plane. A tension crack running parallel to the crest of the slope can also be included in the calculation.

A detailed discussion on the ~~analysis of plane failure is~~ given in Chapter 7.

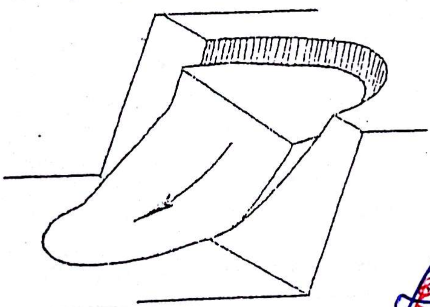


2

Wedge failure

When two discontinuities strike obliquely across the slope face and their line of intersection daylights in the slope face, the wedge of rock resting on these discontinuities will slide down the line of intersection, provided that the inclination of this line is significantly greater than the angle of friction. The calculation of the factor of safety is more complicated than that for plane failure since the base areas of both failure planes as well as the normal forces on these planes must be calculated.

The analysis of wedge failures is discussed in Chapter 8.



3

Circular failure

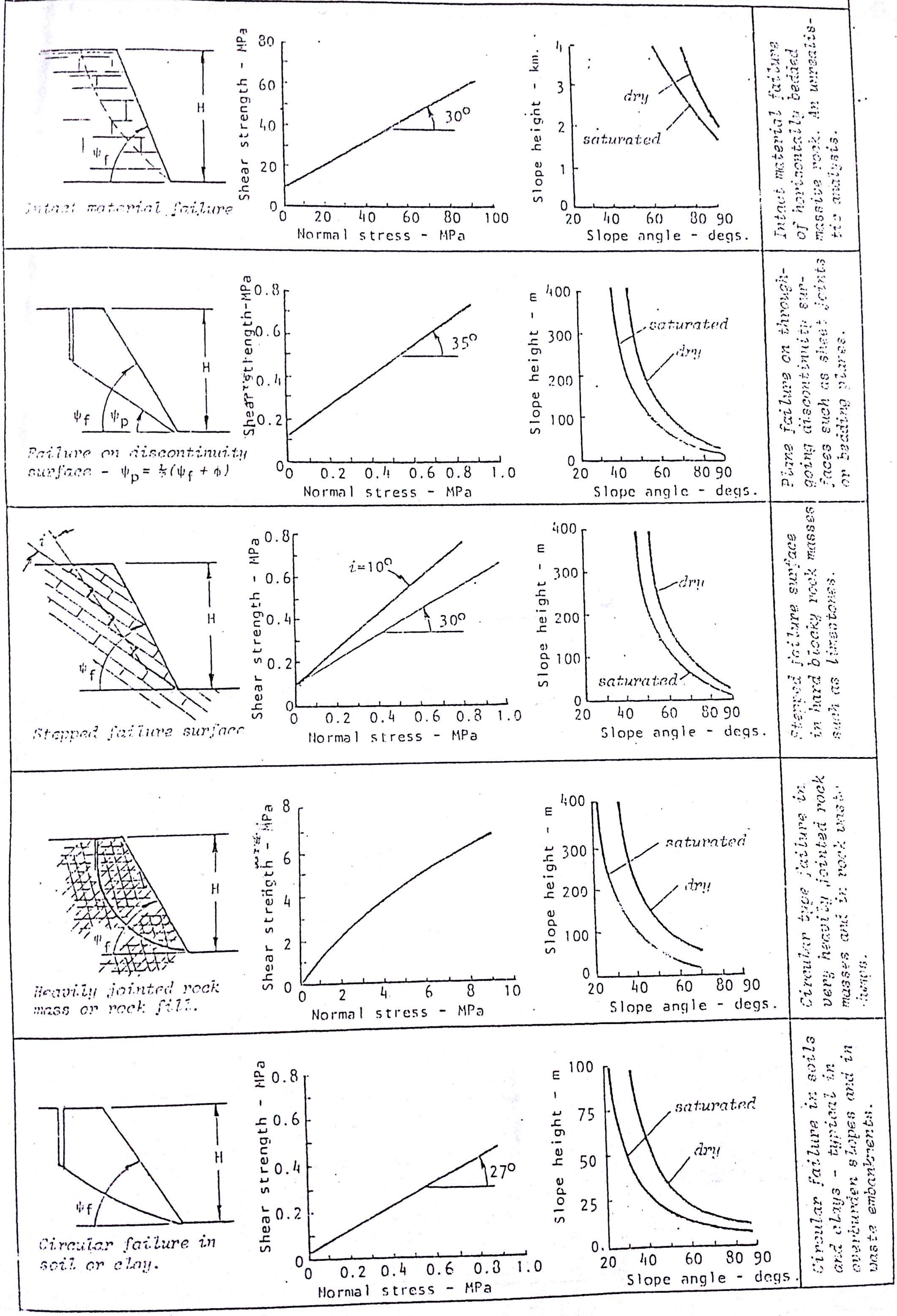
When the material is very weak, as in a soil slope, or when the rock mass is very heavily jointed or broken, as in a waste rock dump, the failure will be defined by a single discontinuity surface but will tend to follow a circular failure path. This type of failure, illustrated in the margin sketch, has been treated in exhaustive detail in many standard soil mechanics textbooks and no useful purpose would be served by repetition of these detailed discussions in this book. A set of circular failure charts is presented in Chapter 9 and a number of worked examples are included in this chapter to show how the factor of safety can be calculated for simple cases of circular failure.

4

Critical slope height versus slope angle relationships

One of the most useful forms in which slope design data can be presented is a graph showing the relationship between slope heights and slope angles for failure, e.g. the dashed line in Figure 7. A number of typical slope failure cases have been analysed and the relationships between critical slope heights and slope angles have been plotted in Figure 10. This figure is intended to give the reader an overall appreciation for the type of relationship which exists for various materials and for the role which groundwater plays in slope stability. The reader should not attempt to use

Figure 10 : Slope angle versus slope height relationships for different materials.



this figure as a basis for the design of a particular slope since the conditions may differ from those assumed in deriving the results presented in Figure 10. Individual slopes should be analysed using the methods described in Chapters 7, 8 and 9.

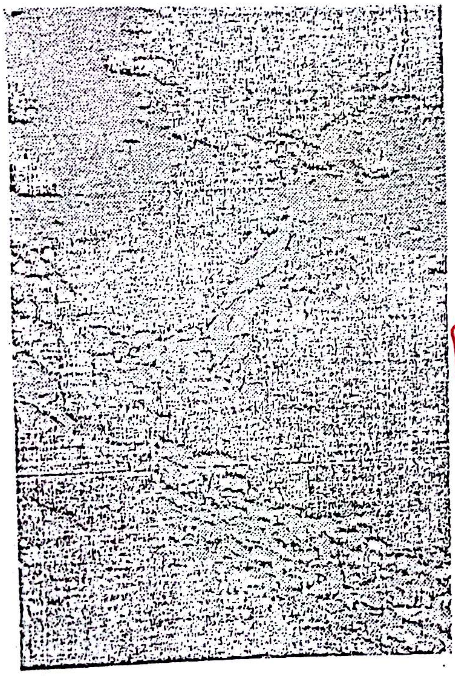
Slopes for which a factor of safety cannot be calculated

The failure modes which have been discussed so far have all involved the movement of a mass of material upon a failure surface. An analysis of failure or a calculation of the shear factor of safety for these slopes requires that the shear strength of the failure surface (defined by c and ϕ) be known. There are also a few types of slope failure which cannot be analysed by the methods already described, even if the strength parameters of the material are known, since failure does not involve simple sliding. These cases are discussed on the following pages.

1 *Toppling failure*

Consider, once again, a block of rock resting on an inclined plane as shown in Figure 11a. In this case, the dimensions of the block are defined by a height h and a base length b and it is assumed that the force resisting downward movement of the block is due to friction only, i.e. $c = 0$.

When the vector representing the weight W of the block falls within the base b, sliding of the block will occur if the inclination of the plane ψ is greater than the angle of friction ϕ. However, when the block is tall and slender ($h > b$), the weight vector W can fall outside the base b and, when this happens, the block will topple i.e. it will rotate about its lowest contact edge.



Toppling failure in a slate quarry .

The conditions for sliding and/or toppling for this single block are defined in Figure 11b. The four regions in this diagram are defined as follows :

By limiting eq.

Region 1 : $\psi < \phi$ and $b/h > \text{Tan}\ \psi$, the block is stable and will neither slide nor topple.

Region 2 : $\psi > \phi$ and $b/h > \text{Tan}\ \psi$, the block will slide but it will not topple.

Region 3 : $\psi < \phi$ and $b/h < \text{Tan}\ \psi$, the block will topple but it will not slide.

Region 4 : $\psi > \phi$ and $b/h < \text{Tan}\ \psi$, the block can slide and topple simultaneously.

cannot analyse

In analysing the stability of this block, the methods of limiting equilibrium can be used for regions 1 and 2 only. Failure involving toppling, i.e. regions 3 and 4 to the right of the curve in Figure 11b, cannot be analysed in this same way. Methods for dealing with toppling failure in slopes are discussed in Chapter 10.

2 *Ravelling slopes*

Travellers in mountain regions will be familiar with the accumulations of scree which occur at the base of steep slopes. These screes are generally small pieces of rock which have become detached from the rock mass and which have fallen as individual pieces into the accumulated pile. The cyclic expansion and contraction associated with the freezing and thawing of water in cracks and fissures in the rock mass is one of the principal causes of slope ravelling

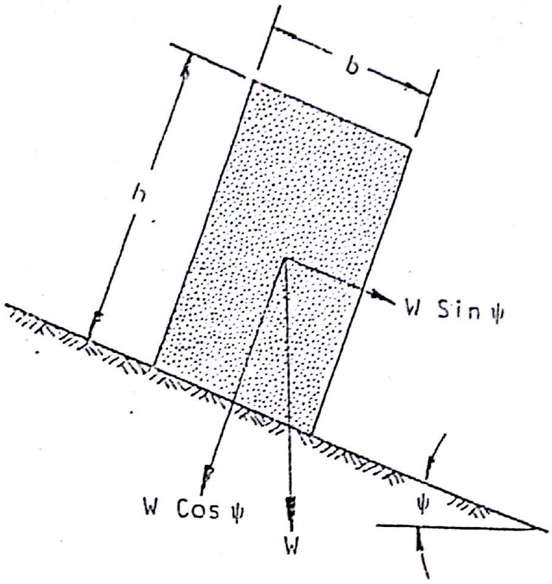


Figure 11a : Geometry of block on inclined plane.

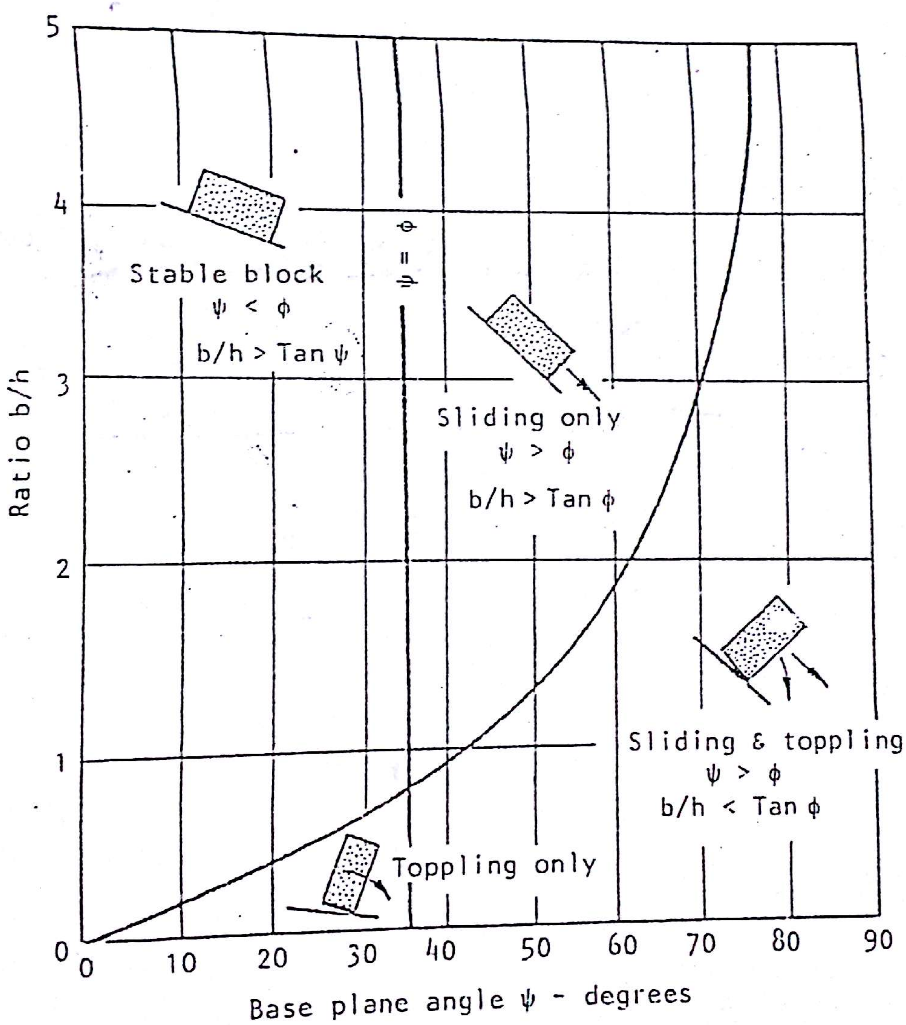


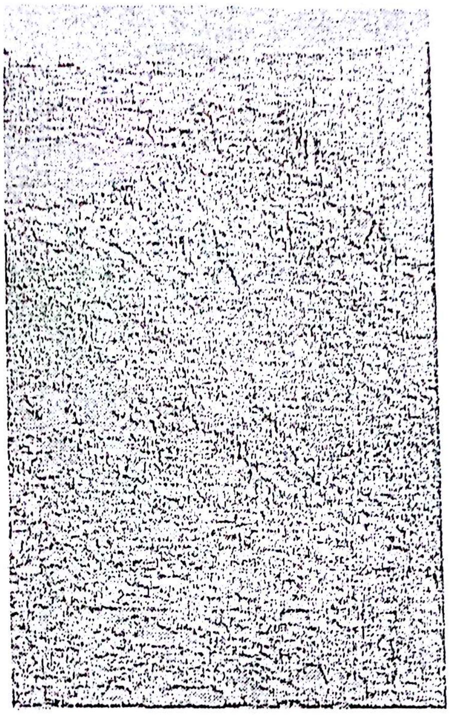
Figure 11b : Conditions for sliding and toppling of a block on an inclined plane.

but a gradual deterioration of the materials which cement the individual blocks together may also play a part in this type of slope failure.

Weathering, or the deterioration of certain types of rock on exposure, will give rise also to a loosening of a rock mass and the gradual accumulation of materials on the surface and at the base of the slope. Some of the engineering implications of weathering have been reviewed by Goodman[24] who gives a selection of useful references on the subject[25-30].

Few serious attempts have been made to analyse the process of slope failure by ravelling since the fall of small individual pieces of rock does not constitute a serious hazard. When the stability of an accumulation of scree or of weathered material is likely to be altered by the excavation of a slope in this material, the stability of the excavation can be assessed by one of the methods described in Chapters 7, 8 and 9. Generally, the method of circular failure analysis, described in Chapter 9, would be used unless the size of the excavation is such that it is likely to cut back into the undisturbed rock mass.

It is important that the slope designer should recognise the influence of weathering on the nature of the materials with which he is concerned and this subject will be discussed in greater detail in Chapter 7.



Ravelling of the weathered surface material in a slope

Probabilistic approach to slope design

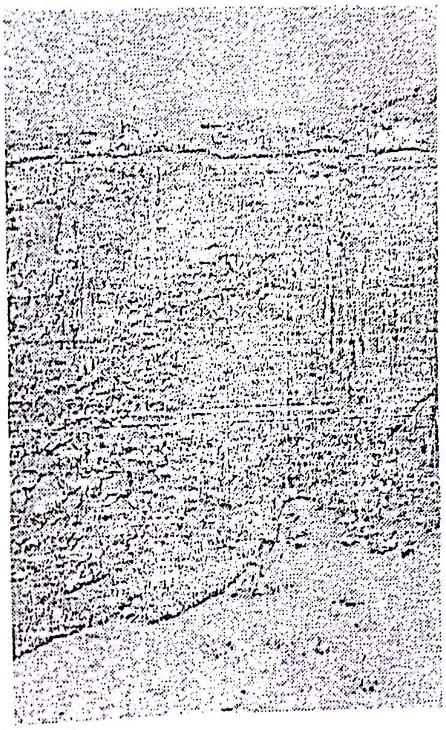
Probability theory has two distinct roles in the design of rock slopes :

a. In the analysis of populations or families of structural discontinuities to determine whether there are dominant or preferred orientations within the rock mass.

b. As a replacement for the factor of safety as an *index* of slope stability (or instability).

The first role is discussed in Chapter 3 which deals with the graphical presentation of geological data. The second role, that in which probability of failure replaces factor of safety as an index of slope stability, has been strongly advocated by McMahon[31] and has been utilised by a number of other authors[32-35].

It should clearly be understood that the use of probability theory in this latter role does not influence the other steps in a stability investigation. The collection of geological data follows the same basic pattern as that described in this book. The mechanics of failure are treated in the same way and the same limitations apply to the types of failure which can be analysed. Probability theory does not, at present, offer any particular advantages in the analysis of toppling, ravelling or buckling type failures.



Slumping of columns in vertically jointed dolerite as a result of weathering in an underlying shale layer.

The authors of this book have chosen to present all the detailed discussions on stability analysis in terms of the factor of safety. This decision has been made because it is believed that the discussion is less confusing for the non-specialist reader for whom this book is intended. The reader who feels that he has understood the basic principles of slope analysis is strongly recommended to examine the