THE STABILITY OF SLOPES CUT INTO NATURAL ROCK

STABILITE DES PENTES EN ROCHE NATURELLE

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It has been recognised by Terzaghi (1962), Muller (1964), John (1968) and others that the stability of a slope cut into natural rock is a problem in engineering geology, combining the arts of estimation and judgment which are features of both disciplines The mechanisms of slope failure are similar to those found with soil with this basic difference - in soils the surface of failure will follow a path of minimum strength, whereas in rock slopes, where the strength of the material is relatively high, the surface of failure will follow preferred planes of weakness which are determined by geological features such as faults, contacts, bedding planes, cross joints, random joints, all of which, for generality in this paper, will be referred to as joints. Water pressures in the joints, and to a lesser extent in the pores of the intact rock, will play the same important role as do 'porewater' pressures in soils.

GEOLOGICAL PROPOSITIONS

If the stability of a rock slope is to be described quantitatively, certain propositions which will permit the definition of properties in a numerical form must be made as follows:

The Proposition of Structural Regions in the Rock Mass

Any naturally occurring rock mass of large size can be divided into structural regions within which the jointing patterns will be similar in a statistical sense, i.e. in a region the joints can be grouped into a limited number of joint sets and all joints in any particular joint set will be indentical within a range which can be defined by statistical limits. By identical is meant that the joints are the same with respect to the 10 joint factors described later.

The Proposition of an Ability to Describe Joints Quantitatively

The joints will be described by:

(a) *The Joint Naming Factors*, i.e. the direction of dip (DD) and the angle of dip (δ).

(b) *The Strength Factors*, i.e. the consistency or hardness of the rock, which is a function of its compressive strength; the roughness of the surfaces of the joint; the nature of the gouge within the joint, i.e. its thickness and its strength or hardness.

(c) *The Factors Affecting Shear Along the Joint*, i.e. the apparent dip of the joint with respect to the slope (α), the continuity of the joint (k) and its waviness.

The Proposition of the Prediction of the Presence of an Unseen Joint

The boundaries of a structural region may be defined by a survey of the joints which have already been exposed. The fact that joints

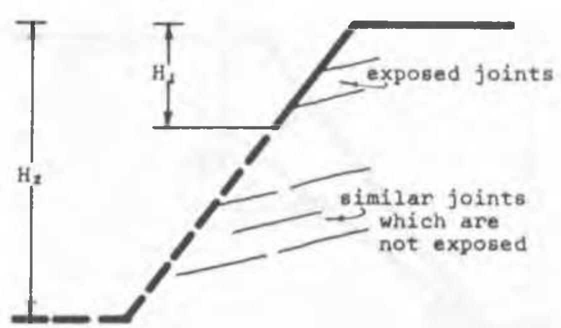

Fig. 1 Prediction of the Presence of Unseen Joints

of a particular type are visible when the excavation has reached depth H_1 is an indication that other similar joints exist in the remaining unseen volume of the exposed structural region. Such joints may be of no concern when the depth is H_1 but they may cause failure when the depth reaches H_2. The relationship between the Factor of Safety and the height of the slope is approximately hyperbolic.

The theory of slope stability in rock will depend upon man's ability to define the joint factors in a numerical way. Further, attention must also be given to the fact that natural variations will exist and that the numbers of joints which are involved will usually be very large. Two most important questions to be answered are: how far should one go in performing laboratory tests and to what extent can one rely on simpler and coarser field observations in determining the numerical quantities necessary for design calculations. In answering these questions, the great volume of joint data required is a factor of prime consideration.

MODES OF FAILURE IN ROCK SLOPES

Failure occurring on preferred planes of weakness implies that the failure surfaces will be plane or combinations of planes. The angle β is defined as the dip angle of a mean surface of failure. It should be noted that while the angle β derives from α, the apparent angle of dip with respect to the dip direction of the slope, β is not necessarily equal to α. It is important to understand that displacements during failure will take place in the α-direction and not in the β-direction where this is different from α. Four modes of failure are recognised, three of them relating to two-dimensional slices through the slope and one to a three-dimensional wedge failure. A depth of cracking z_o at the top of the slope is also accepted. The recognised failure modes are:

(a) Plane Failure Mode

In Fig. 2 the block ABDE slides out on the β-plane. The angles β and α may be the same, involving a number of α-joints

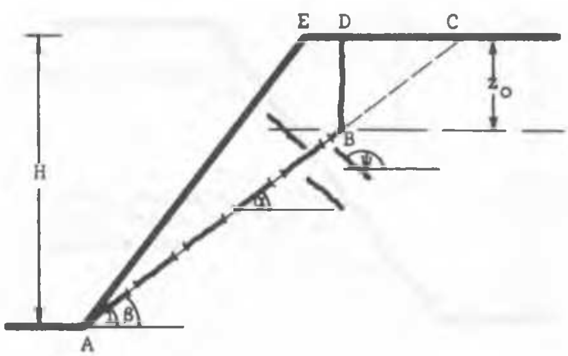

Fig. 2 Plane Failure Mode

lying in one plane (the ordered system) or it may involve stepping from joint to joint where these have mean lengths $L_{j\alpha}$ and mean spacings $d_{m\alpha}$ (statistical system). Such stepping is considerably assisted if another set of joints exists with an apparent dip angle ψ.

(b) Conjugate Planes - Zone Failure Mode

These sets of joints defined by apparent dip angles α_α, α_1 and α_2 exist and these give rise to potential failure surfaces β_α, β_1 and β_2. Below the cracking depth z_o, failure must occur on all planes simultaneously. The block ABKFG moves out, the block KCDF is a zone of vertical tension failure and the block BCK is a region of shear failure, simultaneously on the β_1 and β_2 planes. The solution

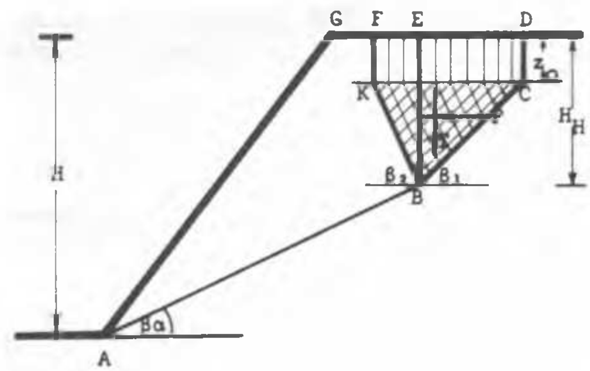

Fig. 3 Conjugate Planes - Zone Failure Mode

is found by considering the horizontal and tangential forces (P and T) developed on the surface BE, and the analysis is then carried out as for plane failure *(a)* above with P and T acting on the face of depth H_H.

(c) Conjugate Planes - Block Failure Mode

Again, three sets of joints exist but in this case the shear strength on the β_2 planes is so great that failure does not occur in this plane. As the block moves out on the β_α plane, an overhang developes

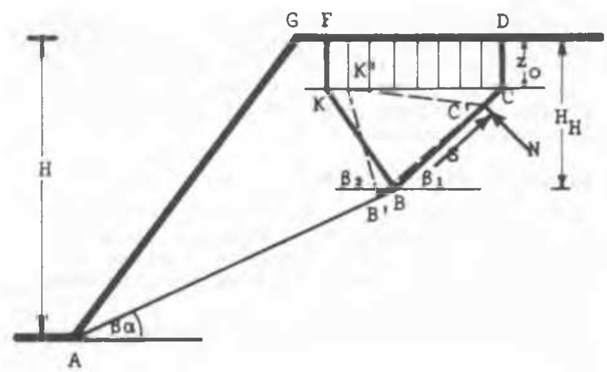

Fig. 4 Conjugate Planes - Block Failure Mode

to the right of point B. Failure in tension takes place, either on the β_2 plane, as shown, or more probably, on the vertical plane through B. An intact block such as BCK rotates as shown by B'C'K', exerting a force at B' down the AB plane on the failing block ABKFG.

(d) Three-Dimensional Wedge Failure Mode

If two intersecting planes are such that the line of intersection between them has a component of dip down the slope, the wedge thus formed may slide out, either as a rigid body or as a broken mass, if failure also takes place within the wedge. In Fig. 5 ABD represents the plan of a joint with strike AB and dip δ_1; BCD that of a joint with strike BC and dip δ_2; the intersection of the two planes is given

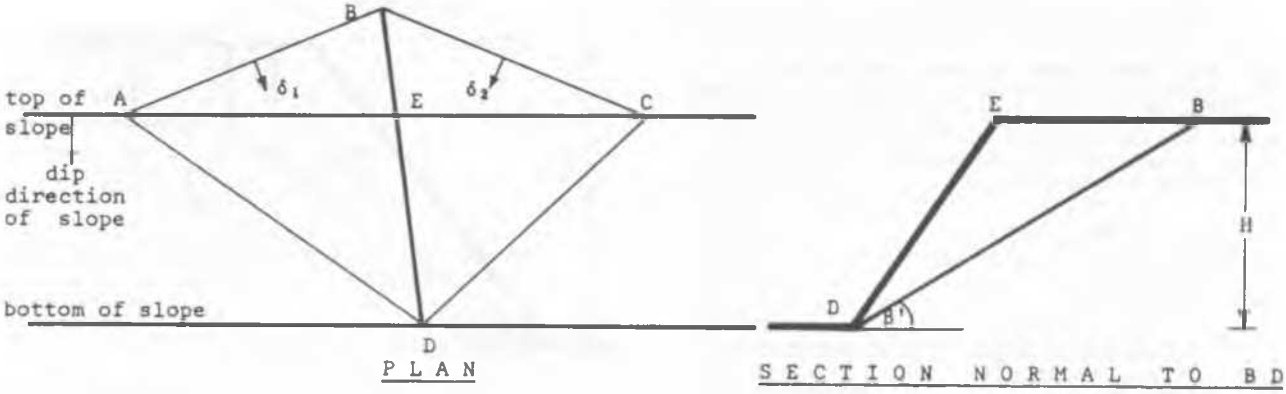

Fig. 5 Three-Dimensional Wedge Failure Mode

in plan by the line BD, having an apparent dip β'. If the block remains intact during sliding out and if no separation takes place on either joint surface then the vector of movement is in the direction BD and the resisting forces on the δ_1 and δ_2 planes are in the direction DB. The analysis is carried out by three-dimensional resolution of forces.

If the Factor of Safety, F, is taken on the shear strength, in the way commonly used in slope stability analysis, the necessary equations for the four modes of failure may be set up. They become somewhat involved if considerations of water pressures and combinations of surcharge and z_o are included. Their solution requires the use of a highspeed computer, particularly when combinations of β's and possible events are taken into account. The equations are not given here and will be published elsewhere.

The Strength Parameters applying to the β-Planes

The analysis leading to a factor of safety can only be carried out if we have a knowledge of the strength parameters applying to failure about the mean β-plane. Fig. 6 shows the joints and elemental forces acting on them for the surface of failure defined by the β-plane. If σ_n is the stress normal to the α-plane; if it be assumed that Mohr-Coulomb relationships apply to shear failure through the intact rock and along joint surfaces; if c_m, ϕ_m and c_j, ϕ_j are the cohesion and friction parameters applying to the intact rock and to the joints, respectively, and, finally, if t_m is the tensile strength of the intact rock then:

s_m = shear strength of the intact rock for failure in the α-direction

$$= c_m + \sigma_n . \tan\phi_m \quad \dots (1)$$

and

s_j = shear strength along the joint surfaces for failure in the α-direction

$$= c_j + \sigma_n . \tan\phi_j \quad \dots (2)$$

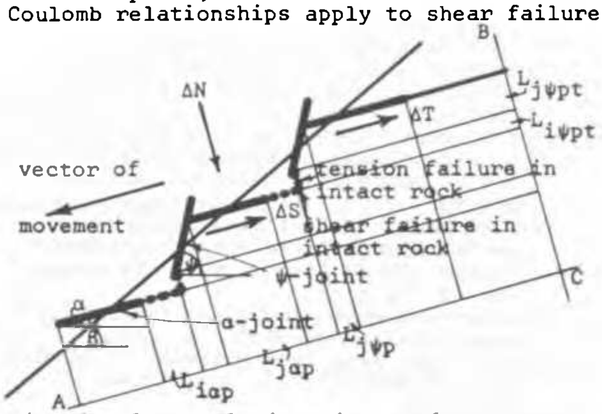

Fig. 6 Elemental Dimensions and Forces on the β-Plane

In Fig. 6, taking unit widths normal to the paper and considering the whole length of the β-plane, R.F., the force resisting sliding on the β-plane (in the vector direction α) is:

$$RF = \Sigma\Delta S + \Sigma\Delta T \quad \dots (3)$$

As movement takes place, separation occurs on the ψ-joints passing the Δ(ΔN) forces which previously acted across them onto intact rock and joints of the α-direction. As a conservative assumption it is taken that the elemental forces are transferred only to the α-joint surfaces. The elemental normal force then acting on an α-joint surface is $\sigma_n(L_{j\alpha p}+L_{j\psi p})$ where $L_{j\alpha p}$ and $L_{j\psi p}$ are the projections of the α and ψ joints onto AC, parallel to the α-plane, as shown in Fig. 6:

$$\Sigma\Delta S = \{c_m \Sigma(L_{i\alpha p}) + \sigma_n \Sigma(L_{i\alpha p}) \tan\phi_m\} + \{c_j \Sigma(L_{j\alpha p}) + \sigma_n \Sigma(L_{j\alpha p} + L_{j\psi p}) \tan\phi_j\} \dots (4)$$

Clean joints without gouge are considered to have $c_j = o$ and, if a joint has any cohesion, this is due to the cohesion of the gouge material filling the joint. This gouge will normally have a strength considerably lower than the intact rock. Hence the term $c_j \Sigma(L_{j\psi p})$ tends to be small in comparison with the other terms and, without loss of accuracy, equation (4) may be rewritten:

$$\Sigma\Delta S = \Sigma(L_{i\alpha p})(c_m + \sigma_n \tan\phi_m) +$$

$$\Sigma(L_{j\alpha p}+L_{j\psi p})(c_j+\sigma_n\tan\phi_j) \quad \ldots (5)$$

Dividing both sides of the equation by $(\Sigma L_{i\alpha p}+\Sigma L_{j\alpha p}+\Sigma L_{j\psi p})$ which is the total length of the surface of shearing, AC, and defining $k_{\alpha\beta\psi}$, the coefficient of continuity of the α and ψ joints with respect to shearing in the α-direction, gives:

$$k_{\alpha\beta\psi} = \frac{\Sigma L_{j\alpha p}+\Sigma L_{j\psi p}}{\Sigma L_{j\alpha p}+\Sigma L_{j\psi p}+\Sigma L_{i\alpha p}} \quad \ldots (6)$$

then

$s_{\alpha\beta}$ = shear per unit length for shearing on the β-plane with movements in the α-direction

$$= (1-k_{\alpha\beta\psi})(c_m+\sigma_n\tan\phi_m)+ k_{\alpha\beta\psi}(c_j+\sigma_n\tan\phi_j) \quad \ldots (7)$$

writing

s_m = strength of the intact rock when the normal stress across the shearing plane is σ_n, equation (7) becomes:

$$s = \{(1-k_{\alpha\beta\psi})s_m+k_{\alpha\beta\psi}.c_j\}+\sigma_n\{k_{\alpha\beta\psi}.\tan\phi_j\} \quad \ldots (8)$$

If hypothetical or apparent parameters c_a and ϕ_a apply to shearing on the β-plane with movements in the α-direction, then:

$$s = c_a+\sigma_n\tan\phi_a \quad \ldots (9)$$

Since our main interest is in $k_{\alpha\beta\psi}$ near 1.0 we may write:

$$c_a = \{(1-k_{\alpha\beta\psi})s_m+k_{\alpha\beta\psi}.c_j\} \quad \ldots (10)$$

and

$$\tan\phi_a = k_{\alpha\beta\psi}\tan\phi_j \quad \ldots (11)$$

By similar reasoning, applied to the $\Sigma\Delta T$ term of equation (3)

$$k_t = k_{\alpha\beta\psi t} = \frac{\Sigma L_{j\psi pt}}{\Sigma L_{j\psi pt}+\Sigma L_{i\psi pt}} \quad \ldots (12)$$

and t_a = apparent tensile strength

$$= (1-k_t).t_m \quad \ldots (13)$$

Equations (10), (11) and (13) allow the setting up of equations for the disturbing and resisting forces for failure on the mean β-plane. Referring to Fig. 7, if W_S is the weight of surcharge over the sliding wedge of weight W_W, then

$$DF = (W_S+W_W)\sin\alpha \quad \ldots (14)$$

$$(RF) = \frac{1}{F}\{c_a.(H-z_o)\operatorname{cosec}\beta.\cos(\beta-\alpha) +(W_S+W_W)\cos\alpha.\tan\phi_a +t_a(H-z_o)\operatorname{cosec}\beta.\sin(\beta-\alpha)\} \quad \ldots (15)$$

$$(RF) = DF \quad \ldots (16)$$

The solution requires knowledge of the shear strength of the intact rock (s_m or c_m, ϕ_m and σ_n); the shear strength on the joints

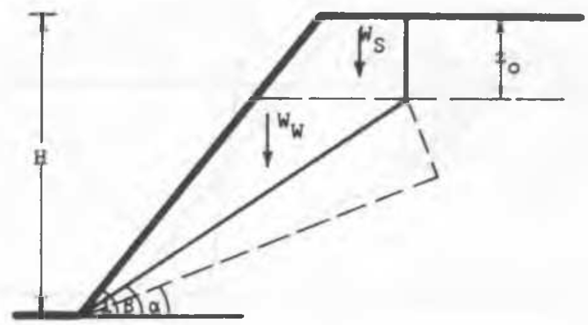

Fig. 7 Wedge Failure of a Slope

(c_j, ϕ_j); the tensile strength of the rock (t_m); the apparent dip angle of the joints (α or ψ); and the coefficients of continuity ($k_{\alpha\beta\psi}$, k_t). The investigation and, in particular, the field survey of the joints must be so arranged as to provide this necessary information.

THE JOINT SURVEY AND THE ASSIGNMENT OF VALUES TO THE PARAMETERS NEEDED IN THE ANALYSIS

This survey should aim to measure a sufficient number of exposed joints so that the data can be viewed statistically. It has been found that where sufficient exposure of rock face exists a line survey is quicker to conduct and the results from it are simpler to analyse. It is preferable that a line survey be carried out, i.e. every joint along the selected line should be observed for the following:

(a) Position of the centre of the joint along the line: this should allow the co-ordinates x, y, z to be fixed.

(b) Dip angle, δ.

(c) Dip Direction, angle DD.

(d) Hardness of the rock to a scale similar to that used for the consistency of soils i.e. use simple field tests to divide the rock into five categories:

- H1 very soft rock - can be peeled with a knife, material crumbles under firm blows with sharp end of a geological pick;
- H2 soft rock - can just be scraped with a knife - indentations 1/16 in. to 1/8 in. with firm blows of the pick point;
- H3 hard rock - cannot be scraped or peeled with a knife - hand specimen breaks with a firm blow of the hammer end of the pick;
- H4 very hard rock - specimen breaks with more than one blow of the pick;
- H5 very very hard rock - breaks only with great difficulty and many blows of the pick.

(e) Roughness of the surfaces of the joint is assessed using five categories:

slickensided; smooth; defined ridges; small steps and very rough.

(f) Gouge thickness is recorded, again in five categories:

no gouge at all; gouge thickness 0-½ in; thickness ½-1 in; thickness 1-2 in; and, finally, greater than 2 in.

All filling in between joints is defined as gouge - it might be a fault breccia, a clay, or even a calcite deposition in the joint.

(g) The gouge material is described firstly by its origin and then either as a soil or a rock, its hardness classification being noted. If the gouge is considered to be a soil, a sample is taken and the Atterberg Limits determined in the laboratory. From these limits a rough assessment of the friction angle may be made.

(h) Waviness of the exposed joint surface is measured by placing a straight edge down dip and recording the length between high points of contact, together with the offset in between them.

(i) The longest visible length of the exposed joint is recorded.

(j) Rock type.

(k) Nature and origin of the joint.

These observations are then used to make assessments as follows:

(a) The structural regions are determined by plotting dip and dip direction sequentially along the survey lines. Changes in patterns in the plots denote boundaries to the regions and these are generally found to coincide with geological features such as faults, dykes or contacts.

(b) Within each structural region the dip and dip direction are plotted on rectangular plots as proposed by Pincus (1951). The data is also corrected for sampling direction and dip angle to give equivalent numbers for sampling lines normal to the joint planes. The plots taken together permit the definition of joint sets within the region. Other factors such as roughness, joint lengths, etc. may also be isolated in the plots and, together, these data permit one to define the design joint sets.

(c) The major geological features, e.g. dykes, faults and contacts, which are likely to be continuous, are separated out and tested separately. In the design of the slope each must be considered individually.

(d) The apparent dip angle, α, is calculated using simple trigonometry.

(e) The number of joints intersected on a

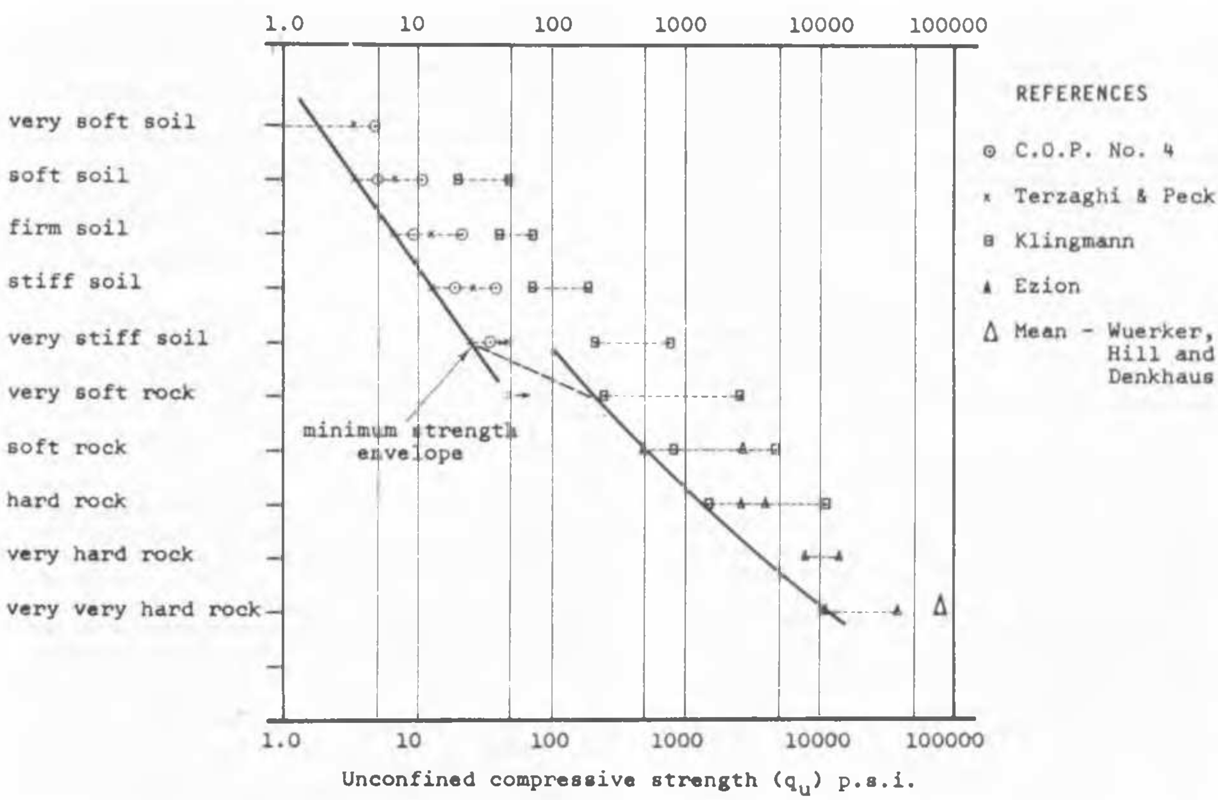

Fig. 8 Relationship Between Hardness or Consistency and Unconfined Compressive Strength

known length of survey line, is used together with the directions of the joint and of the line to calculate d_m, the mean distance between joints of the same set, measured normal to the joint planes.

(f) The joint lengths are used to calculate L_{jm}, the mean joint length, and L_{jl}, the probable maximum joint length for joints within the sets.

(g) The assessed hardness of the rock is used with the empirical curve in Fig. 8 to predict a conservative value of the compressive strength, q_u.

(h) The cohesion of the rock c_m is taken as $0.16q_u$.

(i) The friction angle of the intact rock ϕ_m is estimated from the rock type.

(j) The mean normal stress, σ_n, for the slope under consideration is then judged and used with equation (1) to obtain s_m for the intact rock of the slope.

(k) The joint parameters c_j and ϕ_j are then assessed from the roughness category. If gouge is present the gouge parameters c_{jg} and ϕ_{jg} are assessed and these are combined with the clean joint parameters c_{jc} and ϕ_{jc}, using the percentage of joints filled with gouge and the thickness of the gouge as measures of the relative influences of each. In this way revised c_j and ϕ_j values are found.

(l) The tensile strength of the intact rock is taken as $0.10q_u$.

(m) The coefficients of continuity are assessed from the lengths of the joints in relation to the probable length of the failure surface in the slope. Two models are used - one for an ordered system where an observed joint in an exposed face is taken as a linear model of the slope being designed; the other is for a statistical spatial distribution of the joints involving a concept that failure occurs by stepping from joint to joint on the β line intersecting the joints. The presence of other joints cutting across the main set, around which the β line forms, considerably affect the coefficients of continuity. These coefficients are probably the most difficult and uncertain of all of the assessed factors. Their description requires a separate technical paper which will be published elsewhere.

(n) Waviness refers to irregularities of the surface which are large and unlikely to be sheared off. Attempts have been made to determine the wave shape from offset and distance between high points of contact with a straight edge and results show that a linear variation, which implies a triangular wave form, is as good an approximation as any other shape. It is a conservative assumption. This allows a definition of ω, the angle of waviness. This is used to modify the apparent dip angle α.

WATER PRESSURES IN THE ROCK

As with slopes in soils, water pressures in the rock are a major factor causing instability. Basically, flow in the rock obeys the Laplacian Law but allowances must be made for inflow and outflow at the phreatic surface and for ratios of permeability which are well outside the ranges found with soils. The whole theory is tied back to piezometric observations and the piezometers are installed using a system of hydraulic fracturing to ensure connection with the fissures in the rock. The water pressures are used in the theory in the ordinary way applying to soils with the further assumption that c_a and ϕ_a are parameters with respect to effective stresses.

CONCLUSION

The determination of potential planes of failure in a slope is still largely a judgment process based on many field observations of joints and their properties and simple conservative rules which permit the geometry and the strength along these planes to be assessed. Briefly the process is summarised as follows:

(i) The joint data is analysed using suitable computer printouts which may be interpreted by inspection to yield structural regions, joint sets and major geological features.

(ii) Suitable sorts are performed on the joint data to determine the average joint properties for each joint set.

(iii) Using the result from (ii) above and equations of the form given, estimates are made of continuity of jointing on potential failure planes.

(iv) The remainder of the results from (ii) are used to make assessments of c_m, ϕ_m, c_j, ϕ and t_m, using Fig. 8, and values found in the literature.

(v) The various potential failure planes are used in combination, giving the four basic failure modes. The factor of safety of the slope is determined.

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