METHODS OF SOIL SLOPE STABILITY. CONSTRAINTS OF THE LIMIT EQUILIBRIUM METHODS FOR NATURAL SLOPES

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Summary

Limit equilibrium methods are still very useful on the stability study of soil slopes, notwithstanding the relatively recent advances on the analysis of equilibrium and deformations of these geotechnical structures. Nevertheless it is well recognized that some constraints on the application of the limit equilibrium methods exist and in this work the relative importance of some (the most important) of these constraints is analysed. Application of the limit states approach to the safety quantification of material soil slopes is briefly tackled.

1. INTRODUCTION

Where reference is made to the slope stability calculation methods it is normally related with the safety analysis against ultimate limit states(1) (ULS), namely the loss of equilibrium of the slope considered as a rigid body (loss of overall stability)(2).

According to the Chapter 9 of the Eurocode on Geotechnics (EC7)(3) dedicated to embankments and slopes "in analysing the stability of a slope all possible failure mechanisms should be considered. The mass of soil bounded by the failure surface is normally treated as a rigid body or as several rigid bodies moving simultaneously.

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(1) Limit states are states beyond which the structure (the slope in the present case) no longer satisfies the design performance criteria (strength, stability, serviceability, durability, etc). Ultimate limit states (ULS) are those associated with collapse, instability or with any forms of failure which may endanger the safety of people.

(2) Loss of overall stability of soil and rocks slopes is just one of a set of ULS to be taken into account: large deformations, internal erosion, hydraulic uplift and rockfalls.

(3) Published for instance in Geotechnik, nº 1, 1990, pp. 1-40.

Failure surfaces or interfaces between rigid bodies may have a variety of shapes including planar, circular and more complicated shapes. Where soil is relatively homogeneous or isotropic in strength, it will usually be adequate to assure circular failure surfaces. For slopes in layered soils with considerable variations of shear strength, special attention should be paid to the layers of smaller shear strength. This may require analysis of non-circular failure surfaces.

The equilibrium of the body bounded by any possible failure surface should be verified when the actions and the shear strength parameters of the soil are assigned their design values[1].

In soils which do not exhibit marked strength anisotropy, the method of slices is recommended. As a minimum the method should verify the overall moment and vertical stability of sliding mass. If horizontal equilibrium is not checked, interslice forces should be assumed to be horizontal".

The above quotation of Eurocode 7 makes reference only to limit equilibrium methods. In fact other deterministic approaches are possible (among them the stress analysis facilitated by the development of finite element method deserves mention) but they are out of the scope of this lecture. Besides limit equilibrium methods are by far the most linked to the engineering practice and Eurocode 7 is intended to be applied only to Geotechnical Categories 1 and 2. Geotechnical categories were introduced in the Eurocode in order to establish minimum requirements for the extent and quality of geotechnical investigations, calculations, construction and control checks. The referred requirements are obviously related with the difficulty and complexity of each geotechnical design and consequently must be clearly identified. Three geotechnical categories (1, 2 and 3) are then defined number 3 corresponding to the higher degree of complexity.

Normally the large longitudinal extent of the slope and the cylindrical shape of the failure surface validate the plane strain condition admitted in the analysis. On the contrary if those conditions don't prevail, the problem is clearly of a three-dimensional nature and adequate methods of limit-equilibrium analysis must be employed.

Typically the safety of the slope declines at a rate which depends on the drainage conditions. It is therefore possible to employ long-term (effective stress) or short-term (total stress) stability analysis, depending on the real slope conditions: drained or undrained, respectively.

On the subsequent sections the main factors conditioning the applications of limit equilibrium methods in the analysis of slope stability are focussed: relative accuracy of the different methods of calculation, mechanical characterization of the soils, soil pore water pressure, failure mechanism, and choice of total or effective stress analysis.

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(1) The values of actions, properties of soils, geometrical data and constraints entered in the calculations are called design values.

Finally some brief considerations will be made about the use of the limit states approach to the safety analysis of natural soil slopes.

2. LIMIT EQUILIBRIUM ANALYSIS

Methods of limit equilibrium are largely used in the stability evaluation of natural slopes and are numerous the developed techniques. Those methods are based on the theory of plasticity making partially use of each one of the upper and the lower bound theorems of the theory(1).

Through the use of those methods in the design, the magnitude of a safety factor is obtained. By definition, when the slope is on the verge of failure its factor of safety is unity, and the analysis can be used to estimate the average shearing resistance along the failure surface or part of failure surface if the shearing resistance is assumed to be known along the remainder.

As already said, the techniques available to apply limit equilibrium analysis are countless. But some principles are common to all methods(2):

a) a slip mechanism is postulated which is feasible and sensible. Normally the sliding surfaces are planar or cylindric but when uniformity conditions don't exist more complex shapes must be assumed and analysis have been developed to handle surfaces of arbitrary shape;

b) the shearing resistance required to equilibrate the assumed slip mechanism is obtained by applying static principles. Two physical concepts are used: the potential slip mass is in a state of limit equilibrium (plastified) and the soil failure criterium is verified in all points of the admitted surfaces;

   The various methods differ in the degree to which the conditions equilibrium are satisfied, and in fact, the most common methods of analysis violate conditions of static equilibrium. Sometimes this can affect significantly the accuracy of the method;

c) the shear resistance required for equilibrium is compared with the available shear strength. The comparison is made in terms of a safety factor commonly defined as that factor by which the shear strength parameters (cohesion, c', and tangent of the angle of internal friction, tang ϕ) must be reduced in order to bring the slope into a state of limiting equilibrium along a given slip surface[(1)];

d) the mechanism corresponding to the lowest safety factor is found by an iterative process. When the position of the slip surface is dictated by a dominant weakness other trials are not necessary.

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(1) Other definitions of safety factor are used in geotechnical engineering and comparisons between numerical values of safety factors defined in a different way must be avoided.

As already said, there are a large number of limit equilibrium methods to make the analysis of the soil slope stability and it is out of the scope of this lecture to describe them here.

As previously referred to, they mainly differ on the ingenuity which they use to introduce additional hypothesis (arbitrary but with different degrees of nearness to the reality) to comply with the fundamentals of static[1] and the simplicity of the procedures adopted. The optimization on the combination of those two goals will have definitive consequences on the usefulness of the method.

Among the most popular of the "approximate" methods (not satisfying all the requirements of the static) are the simplified Bishop(3) for circular sliding surfaces and the Sarma(4), the Janbu(5) and the wedge (sliding block) methods(6) for slip surfaces of any shape.

On the other hand the "accurate" methods satisfy completely the principles of static but it is necessary to solve two non-linear equations to obtain the factor of safety instead of only one equation of the same type for the case of the "approximate" methods. Morgenstern and Price(7), Spencer(8) and Fredlund et al.(9) presented "accurate" procedures which nevertheless imply larger computational effort (and higher probability of numerical divergence). Trying to avoid those inconvenients Correia(10) specified a particular shape of the interslice shear force distribution[2]. Consequently lowering the computational work but keeping the accuracy i.e. satisfying all the conditions of static equilibrium certainly the proposed interslice shear force function deserves further research but the results obtained on the application to the stability analysis of many slopes are very encouraging.

The main aims of this lecture are the limitations of the application of analytical approaches to the soil slopes, notwithstanding the more or less recent progresses of calculation methods. These limitations are mainly related with the mechanical characterization of the slope materials, the pore water pressures, and the geometrical data. In the next sections the interferences of these factors on the quantitative evaluation of the stability of the natural soil slopes will be dealt with.

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(1) There are three types of assumptions to render the problem statically determinated:
- about the distribution of normal stresses along the slip surface;
- concerning the localization of the line of thrust (defined by the set of the interslice forces);
- concerning the distribution of interslice forces.

(2) Morgenstern and Price (op. cit.) assumed that interslice shear and normal forces are related through a specified function and a supplementary unknown parameter.

3. MAIN CONSTRAINTS OF THE LIMIT EQUILIBRIUM METHODS FOR NATURAL SOIL SLOPES

Obviously there are many limitations to the practical application of the limit equilibrium methods, but only those who are really meaningful will be tackled hereafter: the mechanical characterization of the soil, the pore water pressure and the mechanism of possible failure.

3.1. The relative accuracy of the different calculation methods

The difficulties on the quantification of the shear strength are well known. Nevertheless comparisons about the accuracy of the different methods for safety evaluation concerning slope stability are far from being uncommon. And those analysis became, in a certain sense, meaningless if the real scatter on the shear strength measurement is not taken into account.

Fig. 1 presents the geometrical, physical, mechanical and geohydrological characteristics of a slope used by Fredlund e Krahn(11) to compare the results of the different methods employed in the analysis of its stability. Various combinations of geometry, soil and groundwater conditions were then considered and the obtained results showed that, with the exception of the ordinary method (resultant of interslice forces on each slice is assumed to be parallel to its base), the range of factors of safety is less than 4% for all the six cases examined using six different methods (ordinary, simplified Bishop, Spencer, simplified Janbu, rigorous Janbu and Morgenstern-Price).

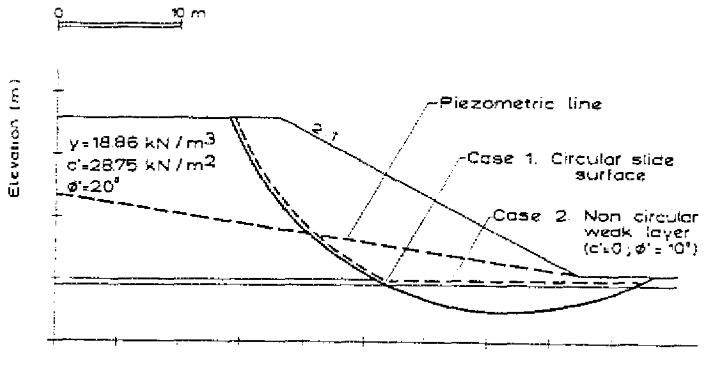

Fig. 1 - Example used by Fredlund and Krahn (11) to compare values of safety factors obtained through different methods(1).

According to Nash(1) this study of Fredlund and Krahn (op. cit.) as well as similar works by other researchers allow the conclusion that all methods that satisfy all conditions of equilibrium give accurate results ($\pm$ 5%) for the analysis of the slopes. Bishop's method which only satisfy moment equilibrium also gives accurate results except where the slip surface is steeply inclined at the toe. The other methods (ordinary and force-equilibrium) which do not satisfy all the equilibrium conditions, may be highly inaccurate.

3.2. The mechanical characterization of the soil

Shear strength is the mechanical characteristic needed to use limit equilibrium methods and cohesion (c) and angle of internal friction (ϕ) are the relevant parameters.

Independently of the techniques to obtain reliable field and laboratory measurements of c and ϕ, a certain scatter cannot be avoided. It is well known that coefficient of variation[(1)], V, of the cohesion is higher than the frictional angle. Values of 40% and 10% can be found in the literature for the coefficient of variation of c and ϕ, respectively Harr(12). As a matter of fact the information is not about cohesion but unconfined compression strength. Nevertheless this parameter is the double of undrained shear strength or cohesion in terms of total stresses.

Another example of the scatter on the shear strength parameters measurement can be seen in Fig. 2(13) where failure effective stresses obtained from twenty seven triaxial tests (consolidated-undrained) collected in the embankment of Odivelas Dam are presented. The soil was unusually homogeneous and all the samples were obtained from the same embankment layer and in a small area. Notwithstanding the extremely favourable conditions, scatter of the shear strength parameters, namely effective cohesion, is appreciable. The ϕ' and c' values presented in the Fig. 2 were obtained by the least square method.

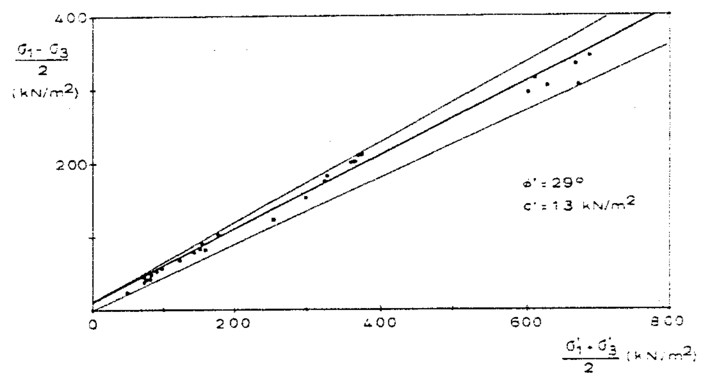

Fig. 2 - Failure effective stresses in triaxial tests performed on twenty seven samples collected in the Odivelas Dam embankment (13).

3.2.1. Slope in a dry frictional soil

It will be examined the sensivity of the conventional safety factor (F) with respect to the angle of friction,

$$F = \frac{tang\ \phi}{tang\ \beta} \tag{1}$$

(1) $V = (s/\mu)\ 100$ (percent), where s is the standard deviation and μ the arithmetic mean. It expresses a measure of the reliability of the central tendency.

According to Harr (op. cit).

$$V\,[F] = \left[\frac{1}{tang\ \beta}\right]^2 V\,[tang\ \phi] \tag{2}$$

that is, the variance of F is proportional to the variance of tang ϕ.

As the expected value (mean) of F is

$$\bar{F} = \left[\frac{1}{tang\ \beta}\right] \overline{tang\ \phi} \tag{3}$$

the coefficient of variation of F is that of the tang ϕ. So a value of 10% can be assigned to the coefficient of variation of F, i.e., a range of 20%. This is the double of the range of 10% (F = ± 5%) obtained when comparing results of different limit equilibrium methods.

3.2.2. <u>Slope in a pure cohesive soil</u>

Using the trial wedge method (and a planar slip surface) Harr (op. cit.) also demonstrated that the coefficient of variation of the critical height of the slope, H_c, (maximum height at which the slope ß can rise) is equal to the coefficient of variation of the cohesion

$$V[H_c] = V[C] \tag{4}$$

As for H_c the condition F = 1 (failure) is attained,

$$V[F] = V[C] \tag{5}$$

Recalling that V[C] ˜ 50%, an order of the magnitude of the coefficient of variation is obtained.

3.2.2. <u>Slope in cohesive and frictional soil</u>

In this case it can be stated that

$$V[H_c] \geq V[C] \tag{5}$$

The conclusion is that, mainly in soils with cohesive behaviour, the influence of the values of c and tag ϕ in the coefficient of variation of the conventional safety factor is significantly higher than that due to the use of different limit equilibrium methods (simplified method of slices excluded).

This doesn't mean that those methods do not need research or are not useful. When the problem has peculiar distribution of interslice forces rigorous methods are absolutely necessary. But this is not the case of many soil slopes and it is not surprising the current and adequate utilization of the simplified Bishop method.

3.3. The soil pore water pressure

A trial on the sensitivity of F to the pore water pressure can be made without difficulty. In this case the shear strength parameters must be considered only in terms of effective stresses (c' and tang ϕ).

It is well known that F and the pore water pressure are related through(14).

$$F = m - r_u\, n \tag{7}$$

where m and n (named stability coefficients) are constant for a given slope and r_u, as well as N (adimensional quantities related to the pore water pressure and effective cohesion, c', respectively), are defined in Fig. 3. The relationship between F and the pore pressure (through r_u) is linear and in Fig. 4 graphical representations for different shear strength parameters in the same slope are presented. It is apparent that, for frictional materials, the higher the angle of friction, the higher the repercussion of the error in the quantification of the pore pressure (consequently on the value of the safety factor). On the other hand cohesion has no influence on the relations just described. Trying to go further on the quantification and assuming a coefficient of variation of 50%, for $r_u = 0.1$, an error of $\pm$ 0.1 can be associated with the determination of F ($\phi = 30°$ and $c' = 5\ kN/m^2$). This must be considered a very rough indication as only a particular slope is analysed. But as the range of the coefficient of variation of pore water pressure is normally very high the error introduced on the evaluation of the safety factor will be certainly more important than the error associated with the calculation models.

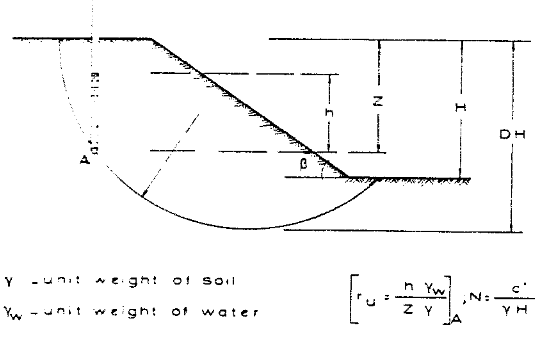

Fig. 3 - Definition of the symbols used in the charts for the application of the method of the stability coefficients.

Frequently the pore water pressures are lower than the atmospheric pressure (suctions). In this case the theoretical context for the interpretation of test data on shear strength characterization is much more complex than for saturated soils (positive pore water pressures).

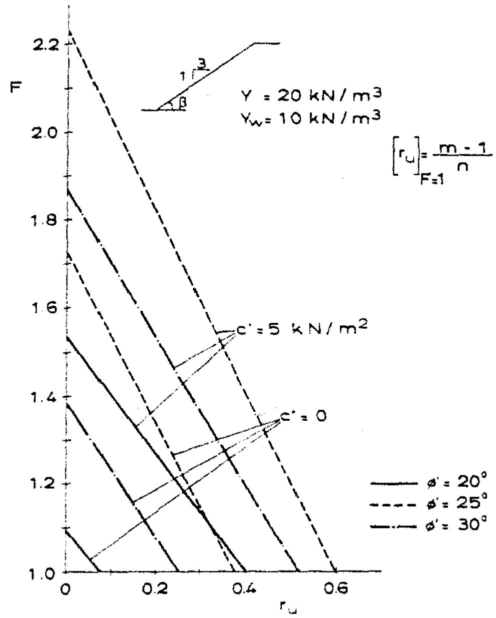

Fig. 4 - Influence of the pore water pressure on the F values.

The first important contribution on this matter was given by Bishop et al.(15) introducing the factor in the equation of effective stresses for unsaturated soils.

$$\sigma' = (\sigma - u_a) + \chi\ (u_a - u_w) \qquad (8)$$

In this equation u_w and u_a are the pressures in the pore water and pore air, respectively. The factor can assume values between 0 and 1 and is a function of many variables, mainly of the degree of saturation. More recently Fredlund et al.(16) proposed a linear shear strength equation in terms of the two independent stress state variables, the net normal stress $(\sigma-u_a)$ and the matric suction (u_a-u_w),

$$\zeta_{ff} = C' + (\sigma_f - U_a)_f\ tg\ \Phi' + (U_a - U_w)_f\ tg\ \Phi^b \qquad (9)$$

In this equation is the shear stress on the failure plane at failure, c', the effective cohesion and ϕ^b is the angle indicating the rate of change in the shear strength relative to changes in matric suction. Index f means failure. As can be seen the shear strength contributions from net normal stress and matric suction are characterized by ϕ' and ϕ^b respectively. For the case of non-linear failure envelops and at low matric suctions (saturated soils), ϕ^b ˜ ϕ'; but as desaturation commences the ϕ^b angle appears to reduce to a relatively constant value(17). Suctions (negative pore water pressures) depend on the equilibrium between the vapor pressure in the atmosphere and in the gaseous phase filling part of the pores (as a matter of fact water exchange may occur between the liquid and gaseous phase of the soil through the surface water film, evaporation or condensation, depending on the vapor pressure in the air of the pores related to the tension of the water surface). And they can also be induced by the execution of excavations. In this last case, suctions result from the decrease in the pore water pressure due to the decrease in the principal stress. These negative pore pressures have an important contribution to the stability of the cut (slope), and as they dissipate the safety factor decreases.

The time required for the pore pressures to attain equilibrium associated with the steady flownet condition is highly variable depending mainly on the soil swelling index, the coefficient of permeability, the detailed stratigraphy and the geometry of the excavation. After a cut or erosion process, the soil is unloaded and will swell with time and soil will become gradually softer and weaker. But it is important to mention that swelling takes place five to ten times faster than consolidation, so in the case of a silty-clay stationary ground water conditions may be reached in weeks or a few months(18). In a real case (Lodalen, Norway) described by this author (op. cit.) it is shown that after a cut in clay the swelling potential lead to negative pore pressures and F could have been equal to 1.5. With time an average steady state of positive pore pressures developed. For instance F value would drop by 10% when ground water level rose about 1,5 m, and fluctuations of about 3 m induce variations of 20% or more in F.

To Janbu (op. cit.) even moderate seasonal pore pressure changes are much more important than uncertainties in effective strength values. Measurement of in situ negative pore pressures in soil slopes is very difficult and not common.

3.4. The mechanism of possible failure

As already mentioned in section 2, limit equilibrium method results from the partial application of each one of the two theorems of the theory of plasticity: the upper bound and the lower bound. As stated in the first one, a collapse mechanism is postulated but without restrictions on the shape of the slip surface. This means that even for ideal material (elastic - perfectly plastic) a slip surface must be assumed. In the nature, geological materials have more or less pronounced discontinuities that influence the slip surface geometry and it is also well-known that strain-softening materials give origin to slip surfaces when they approach failure (fragile rupture).

In principle the determination of the lowest safety factor is obtained by trial unless some kind of discontinuity (or combination of discontinuities) exists defining the slip surface. This situation is by far the most frequent in natural slopes so localization of slip surface is very important but very difficult in most of the cases and it is apparent that large errors can be introduced in the calculations of F. Even in the back-analysis of failure situations in natural soil slopes it is generally very difficult to localize the slide surface.

3.5. Choice between total and effective stress

It is well-known that the knowledge of pore water pressures commonly favours the calculation of F in terms of effective stresses.

In practice this means that for very low permeable materials and at the end of construction situations, the analysis must be done in total stresses. For long term situation the effective stress analysis is recommended. Nevertheless opinions on this matter are not always coincident. For instance Janbu, pretending that on cuts and excavations in clay the evolution of pore water pressures due to swelling can be very rapid, advises the use of effective stress analysis. But La Rochelle(19) considering our inability to predict pore water pressures of clay slopes, in short-term conditions supports the total stress analysis.

This is a matter to be tackled only by experienced geotechnical engineers and these few considerations highlight the possibility of important errors if an inadequate procedure is adopted in the stability analysis.

4. LIMIT STATES AND SAFETY OF SOIL SLOPES

As pointed out in section 1 the safety of a soil slope can be analysed in terms of limit states. The limit equilibrium methods are procedures (well recognized in practice) to verify only one of the possible ultimate limit states (ULS): the occurrence of a slide in the slope.

Limit states design implies not only the verification of the other limit states but also the use of partial safety factors which are employed to obtain the design values of the actions and of the material mechanical properties. The design load effect, S_d, and design resistance effect, R_d, can then be calculated and the design criterion can be expressed in the following way:

$$R_d \geqslant S_d \tag{10}$$

Through this procedure the impossibility (or the probability of occurrence sufficiently low) of occurrence of the ULS being analised (the slide of the slope) is verified.

When compared with the traditional calculation of the global safety factor this kind of safety analysis has some important advantages:
- more logical procedure (puts the incertainty in the right places);
- avoids the use of different safety factors for different design situations;
- makes necessary an exhaustive analysis of the problem through the consideration of all the possible serviceability and ultimate limit states.

With this method it is not intended to modify the safety levels already adopted and this raises the problem of the quantification of the partial safety factors. This matter will be dealt with in the next section

5. QUANTIFICATION OF THE PARTIAL SAFETY FACTORS

In Table 5.1 the values of partial safety factors suggested in the Eurocode on Geotecnics (EC7) are presented. Obviously this values must be calibrated.

TABLE 5.1

Partial safety factors (γ_F and γ_M) suggested in EC7 to be used in the analysis of ultimate limit states (slope slide in the present case)

Actions	γ_F (for permanent actions)
Favourable	1.0
Unfavourable	1.0

Material properties	γ_M
tg ϕ	1.2 - 1.25
c	1.5 - 1.8

It must be pointed out that this approach will be made in semi-probabilistic terms. A probabilistic treatment trying to assess the risk of landslides is being developed and defended by many authors as Chowdhury(20) and Mineiro(21) for instance. The conventional deterministic procedures are only concerned with the determination of a value for F, but it must be stressed that from the probabilistic point of view F must be regarded as a random variable being itself a function of several random variables and constants, i.e.,

$$F = f\ (c', \ \varphi', \ \gamma, \ u, \ H, \ \beta) \qquad (11)$$

and the failure probability (p_f) of the slope will depend on the probability distribution of F. It must be stressed[22] that the influence of the assumed distribution of F is only significant for low values of the probability of failure (say $p_f < 10^{-2}$) and a lack of knowledge of the real distribution of F means that absolute values of p_f are not as significant as relative values when comparing analysis under different conditions against the global safety factor, trying to maintain, as was previously stated, the same safety levels when analysing the stability of soil slopes.

Assuming the slope of Fig. 3 and the conditions D = 1 and r_u = o, it is possible to represent the infinite combinations of c and tg ϕ that bring the slope β to a certain global safety factor (F = 1 means failure). This curved lines, for certain values of γ (volumetric weight) and H, are represented in Fig. 5. Similar graphics can be obtained calculating for other values of β for H constant (variation of parameter γ is so small that it can be considered constant without great error).

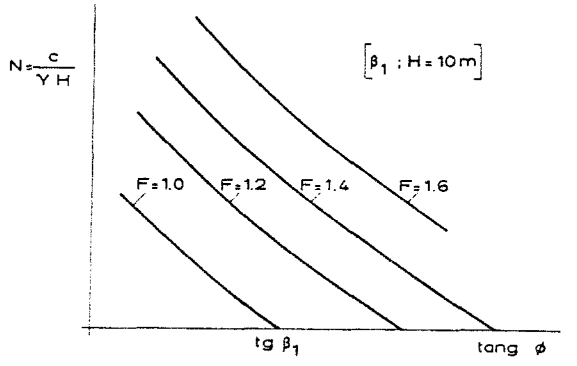

Fig. 5 - Pairs of shear strength parameters corresponding to the same safety factor.

Fig. 6 shows pairs of characteristic values of material properties (that for the slope β_1 are equivalent to a global safety factor equal to 1.4) as well as some design values taking into account numerical values of Table 5.1. Studying a certain number of characteristic values it can be obtained one area of design values (the area represented in Fig. 7). It can be concluded that if this area more or less coincides with the line F = 1, it means that the safety level with the partial safety factors adopted is equivalent, for this slope, to the global safety level of 1.4. But if the location of the area is that one presented in Fig. 7 higher partial safety factors should be used. And obviously in the case of Fig. 8 lower partial safety factors would be necessary.

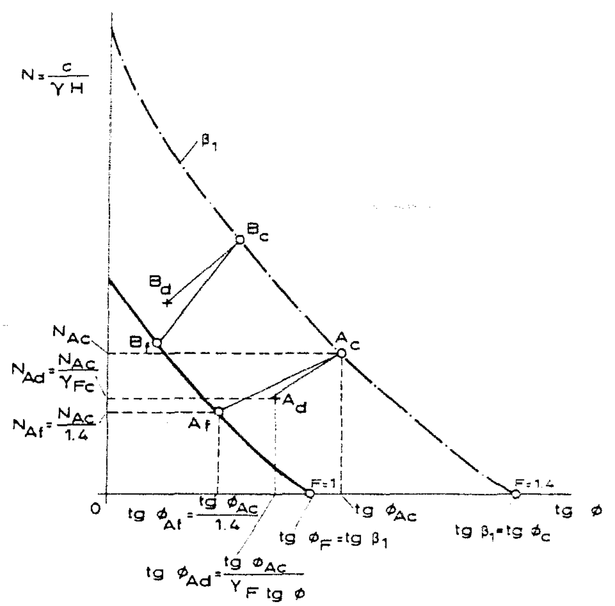

Fig. 6 - Pairs of characteristic shear strength parameters (c_c and ϕ_c) for F = 1.4 and some of the corresponding design and failure values.

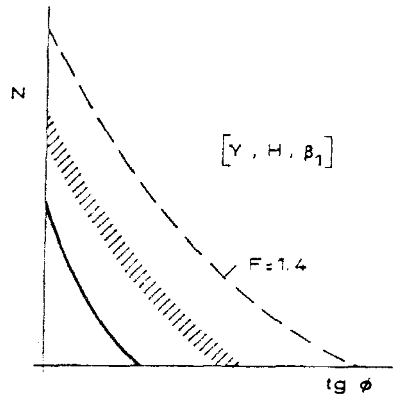

Fig. 7 - Comparison between pairs of shear strength parameters at failure and of design values (obtained from characteristic values equivalent to F = 1.4) showing a lower safety level for these last ones.

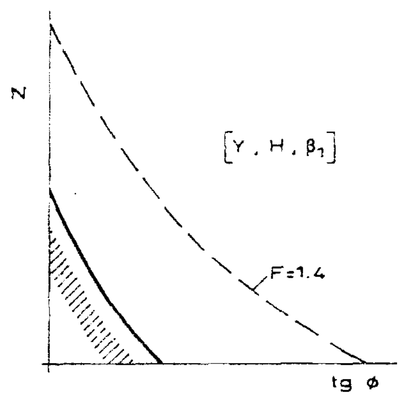

Fig. 8 - Comparison between pairs of shear strength parameters at failure and of design values (obtained from characteristic values equivalent to F = 1.4) showing a higher safety level for these last ones.

The calculations can be repeated for other values of β as well as for different values of H (Fig. 9) in such a way that a calibration between both methods can be done.

In certain cases pore water pressures must be taken into account. Fig. 10 shows lines of F = 1 (failure) for different r_u in the case of a certain slope (2.5H : 1.0V) with H = 10m and γ = 20 kN/m^3. These lines were obtained through the use of the stability charts of Bishop and Morgenstern(14). For the characteristic values of c = 5 kN/m^2 and tg ϕ = 0,25 (point A_f) a situation of failure is attained when r_u = 0. Point A'_f represents also a failure situation but now for the pore water pressure condition r_u = 0.1. It is then possible to evaluate two partial safety factors ($\gamma_{M\phi}$ and γ_{Mc} quantified in Fig. 10) and in this particular case, an idea of the influence of the parameter r_u on safety analysis is then obtained.

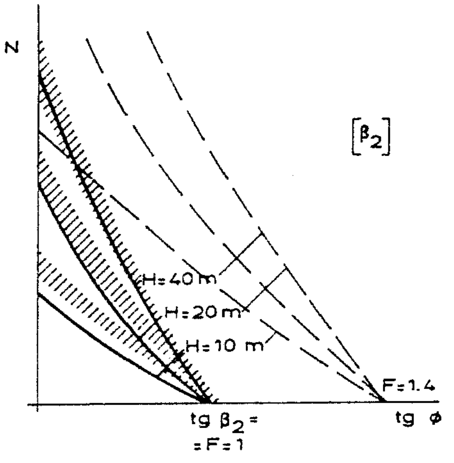

Fig. 9 - Calculations similar to those presented in Figs. 7 and 8 but for different heights H.

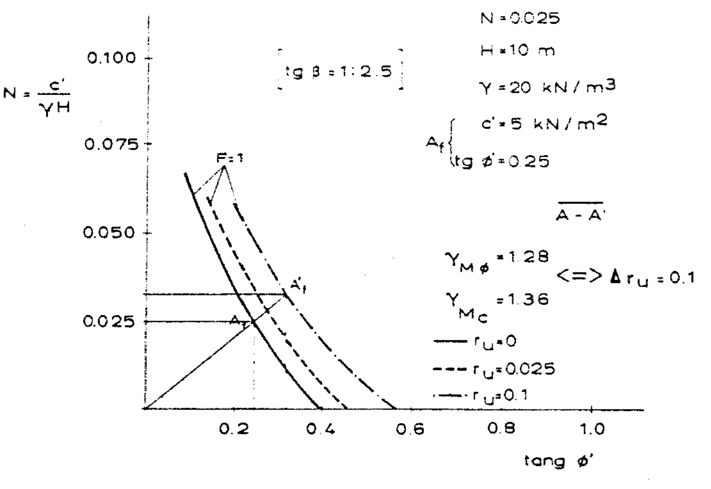

Fig. 10 - Influence of pore water pressure on limit equilibrium analysis.

CONCLUSIONS

Main constraints of the limit equilibrium methods for natural slopes are accuracy of calculation methods, mechanical characterization of the natural soils, the quantification of the pore water pressure and prediction of the real failure mechanism.

Principally in cohesive soils, the influence of the scatter on the measurements of c and ϕ in the coefficient of variation of the global safety factor is higher than the influence of the accuracy of the different calculation methods (excluding the simplified method of slices).

Even moderate seasonal pore water pressure changes are much more important than uncertainties in effective strength parameters.

Important errors can occur in the safety factor determination due to difficulties on the localization of the slip surface.

Prediction of pore water pressures is problematic and is very important an adequate choice of the procedure to be followed in the equilibrium analysis: in terms of effective stresses or in terms of total stresses. Is a kind of matter that can only be tackled by experienced geotechnical engineers.

When compared with the procedures envisaging the obtention of traditional global safety factor, the limit states approach appears as a more logical method and allows a broaden analysis of the safety of the slope. In this last sense it can be safer, notwithstanding that is not intended to modify the safety levels already adopted.

As a final note it must be kept in mind that, when tackling natural slope stability problems, it is dangerous to trust only on intuition but it is also hazardous to rely only on calculations.

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