Some Remarks on Slope Stability Analysis Methods

Reflexions sur lés Méthodes d'Analyse de la Stabilité des Talus

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SYNOPSIS In this paper some theoretical concepts referring to stability analysis methods are discussed. These concepts cain explain some disagreements between calculations and structural behaviour.

1. INTRODUCTION

It has been frequently reported in the technical literature, that in some cases the usual methods for evaluating the stability of slopes render values that are not in agreement with the stability conditions observed in the field. It seems to be that this disagreement might partially find its cause in a deficient way to apply the fundamental concepts on which the stability analysis methods are based. Two misapplications are discussed further on, together with the basic concepts to which they are related.

2. THE USE OF THE COHESIVE AND THE FRICTIONAL SHEAR RESISTANCE CONCEPTS.

From the shear resistence point of view, there are two different ways to stablish the safety factor for a soil structure, each leading to a different definition of this concept, one applied to frictional soils and the other to cohesive soils.

Fig. 1. shows schematically the classical problem of shear failure in Mechanics. In part 1.1 of the figure, there appears the case where the shear strength is not a function of the normal force applied and thus it has a constant value C. The maximum force P that can be applied before the relative movement starts is Pmax = C. If a smaller force P is applied, no movement will occur, and how far from movement the stability conditions are, can be estimated through the factor of safety, expressed as:

$$F_s = \frac{C}{P} \tag{1}$$

which is the traditional way to express it in problems of fixed shear strength, that is, as the relationship between the available shear force (or strength)and the applied shear force (or stress) is constant. This definition of the safety factor has been traditionally used for cohesive soils in Soil Mechanics.

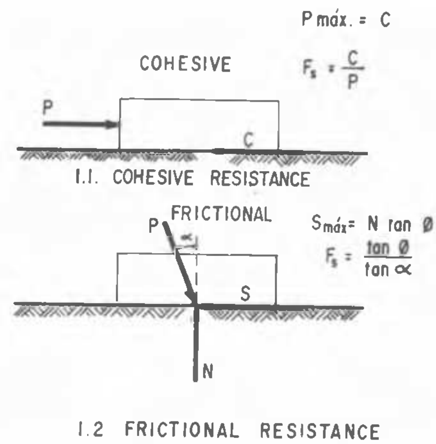


FIG. 1. SAFETY FACTORS (F_s) AS DEFINED IN COHESIVE AND FRICTIONAL SOILS.

The second part of Fig. 1 shows the case where the available shear resistance force is a linear function of the normal force applied. This is the case of a purely frictional behaviour. Now, if force P is applied with an angle $\alpha < \phi$ (where ϕ is the angle of friction) relative movement will never take place, no matter how large P might be, and to evaluate the stability conditions, the factor of safety must be stablished in terms of the sloping angle of the applied force P, as compared with the angle of friction, that is:

$$F_s = \frac{P \cos\alpha \ \tan\phi}{P \ \text{sen}\,\alpha} \tag{2}$$

which leads to:

$$F_s = \frac{\tan\phi}{\tan\alpha} \tag{3}$$

This means that the factor of safety is not a function of the magnitude of the applied force P, but only of its direction α and the friction angle ϕ.

Thus, two conceptually different approaches are necessary, one for cohesive and one for frictional soils. In practice, the use of this two different approaches has not been considered appropriate, and to ma ke them compatible, the safety factor for frictional -- soils has been applied using equation (2) rather than - equation(3) (exception made of slopes in clean sand) - which apparently defines the safety factor for frictional soil in terms of force intensities, instead of in - terms of angles. Using this approach the safety factor apparently becomes equally defined in frictional and -- cohesive soils, for if the shear resistance is constant for a given normal stress, then equations (1) and (2) - conceptually coincide.

However, by defining the safety factor in - - frictional soil as a relation of force intensities and constantly using this approach , the Soil Mechanics - - engineers came to forget that frictional behaviour - -- does not depend on force intensities but on angle values.

In practice, the safety factor is used as an indicator of how far from failure a given soil structure is. If when evaluating the stability of a structure, the designer implicity thinks as if the normal force in equation (2) were constant and the shear force -- were the only one that can vary, then a safety factor - of, for example two, is interpreted as if the shear forces can be duplicated before reaching failure. This -- represents a transformation of frictional soils stability problems, that are governed by frictional mechanics, into cohesive soils stability problems with constant -- shear strength. Such procedure might end in misleading values in practice.

In fact, when equation (2) is expressed in -- terms of effective stresses in Soil Mechanics, the following usual expression is obtained:

$$F_s = \frac{\bar{\sigma} \tan \phi'}{\tau} \qquad (4)$$

where $\bar{\sigma} \tan \phi'$ represents the average shear strength - in the sliding surface,based on the strength parameter ϕ' obtained from a drained test; τ represents the mean shear stress applied on the same sliding surface. Equation (4) is used in practice as if $\bar{\sigma}$ would remain - -- constant as τ increases. A direct shear test can exemplify the usual procedure to evaluate the safety factor in this case. A normally consolidated clay sample is - sheared under normal total stress σ , whithout drainage. Let ABC (Fig. 2) be the effective stress path - -- troughout the test. Following the stress path, there - occurs an ever increasing shear stress up to the value S_3 and an ever decreasing normal effective stress as -- a consequence of the pore water pressure increase due - to the shear distortion. When the sample is subjeted - to the conditions represented by point B, by applying - equation (4) the safety factor obtained would be

$$F_s = \frac{S_2}{\tau_2}$$

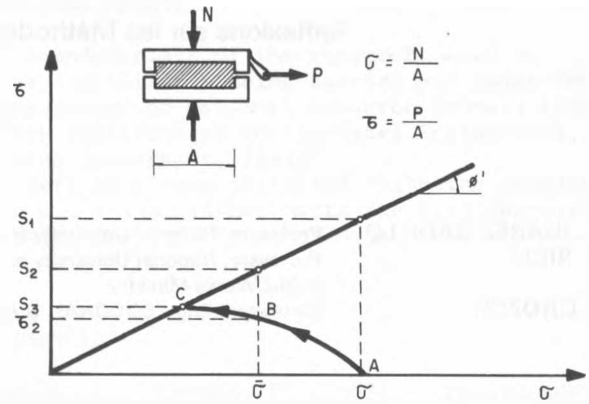


FIG. 2 STRESS PATH IN A CONSOLIDATED UNDRAINED DIRECT SHEAR TEST.

However, as the applied stresses reach failure, the final available shear strength will be S_3, which in this example is smaller than the S_2 previously considered, and the already calculated safety factor shows to be in error, on the unsafe side in this example. So, by the usual procedures, a safety that is not in agreement with the real one can be obtained. In the previous - -- example, a more realistic safety factor would be

$$F_s = \frac{S_3}{\tau_2}$$

which in practice can be easily evaluated.

However, in this last expression, the idea of a safety factor as the relation between forces is still present, even though in figure 2 it can easily be seen - that the clay sample equilibrium (if it shows a frictional behaviour with c = o) depends only on the orientation of the vector from the origin to any point of the - stress path, which is the vector sum of effective normal and shear stresses, and not on the value of the shear -- stress alone as the normal stresses remains constant.

For clay slopes, when analyzed in terms of - - effective stresses, the shear strength law used can be - expressed as

$$s = \bar{\sigma} \tan \phi'$$

which corresponds to a purely frictional behaviour. In these cases the factor of safety must be defined in - -- terms of the slope angle as compared with the available angle of internal friction. If the clay mass has no - - excess pore water pressure, the stability analysis can - be run in the same way as is usual for clean sands and - such method should be considered recomendable. The slope angle governs in this case the stability of the soil structure, because at the slope surface occurs the most unfavorable combination of normal and shear stresses.

The condition that no excess pore water pressu re be present corresponds in practice to slopes with no water flow, for long term stability conditions (drained conditions) and as for as the excess pore water pressure due to shear deformation can be disipated. All this are the usual hypothesis in the long term analysis for - slopes in normally consolidated clays and with no water flow.

For preconsolidated clay slopes with no water flow, if analyzed for long term condition, that is, considering that excess pore water preassure (tension) is disipated, then their behaviour will be nearly frictional too. The difference between this behaviour and the purely frictional one (Fig. 3) is usually attributed to the stored energy at failure due to preconsolidation -- and secondary consolidation effects. In Fig. 3 the --- straight line envelope represents the purely frictio--- nal behaviour of normally consolidated clays, and the - slightly curved upper envolope corresponds to preconsolidated clays. If such small difference is disregar -- ded, then the preconsolidated clay slopes with no excess pore water pressure, should be analyzed as a frictional phenomena and its factor of safety stablished in terms of the slope angle too

Also in Fig. 3, the dash broken line corres-- ponds to the strength envelope of a clay, when tested - in an unconsolidated undrained test and reported in - - terms of total stresses. This envolope defines a constant shear strength (cohesion). The intersection of -- this envelope with the consolidated drained envelope -- has been marked as point A. If the state of stresses - ($\bar{\sigma}, \tau$) induced by the external loads is represented - by point B, to the left of point A, and above the drained strength law, but below the undrained one, during - shear distortion pore water tension will built up for - making the slope stable in the short term conditions, - but as water tension disipates the slope can become - - unstable. Again, it can be seen that for long term conditions the stability depends on the orientation of vector OB and not only the value of τ . If the state of - internal stress is represented by point C, to the right of point A, above the undrained strength law and under the drained one, during shear pore water pressure will grow high enough to turn the slope unstable in the - -- short term conditions, but when excess pore water pressure disipates, in the long run, the slope probably -- will become stable.

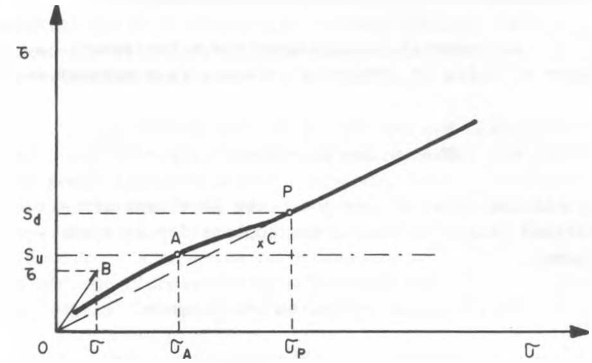


FIG. 3 SHEAR STRENGTH ENVELOPES FOR PRECONSOLIDATED CLAYS.

In all the cases so far analyzed, where no -- excess pore water pressure exists, that is, as far as - no water flow takes place and shear deformation occurs in drained conditions, a linear distribution of neutral pressure with depth exists. This pressure distribution does not interfere with the gravitational field, so the orientation of stress vector is the same as if the soil were dry.

If flow is taking place there will appears -- a new force, the seepage force, that will modified the stress vector orientation. The magnitude and direction of this seepage force varies in practice as far as it - depends on the flow field, and in general, no linearity will exist with depth. In this case, apart from the -- fact that the soil behaviour will continue to be fric-- tional, the slope behaviour will not be frictional any more, and now it is posible that a slip surface through the interior of the slope be more critical than the slope surface itself. By now, the stability condition in this last case are assesed by the limiting equilibrium method using a slip surface that might be circular or - have any other shape. The authors have no better procedure to propose, but believe that if in the future some attention is driven to this problem, it is possible - - that a better razonalization of the slope stability analysis methods can be reach.

3. FAILURE MECHANISMS

Some general remarks can also be made on the current methods to select the failure surfaces that are to be used in the stability analysis. Specially when - no circular sufaces are proposed, it can be frequently seen that the geometrical points of view prevail on the geological ones, which are neglected. Stratification, cracking, discontinuities, folding and other non homo-- genities play a much greater role on the surface posi-- tion that is usually seen in the desk analysis. Exploration and field instrumentation will help to clarify - this aspect in each specific situation.

4. SHEAR STRENGTH OF OVERCONSOLIDATED CLAYS

In normally consolidated clays the shear - -- strength in terms of effective stresses is expressed -- as:

$$s = (\sigma - u) \tan \phi \qquad (5)$$

where σ is the total stress and u the pore pressure - - (positive), built up during the application of shear -- stresses.

This process brings to the diagram

u (positive) ⟶ $\bar{\sigma}$, decreases ⟶

⟶ s (at failure) decreases

Which seemes to produce a solid mental scheme.

Nevertheless, the extension of such ideas to overconsolidated soils appears to be somewhat confuse.

In fact, the authors impression is that in - highly overconsolidated clays under undrained condi--- tions the technicians general thinking is:

u (negative) ⟶ $\bar{\sigma}$, increases ⟶

⟶ s (at failure) increases

This last scheme deserves some considera--- tion because it includes an important fallacy.

Consider as illustration the case of a isotro- pically consolidated soil subjected to a direct shear - test. Consider first the soil beeing in a normally con solidated state, with σ_1 as the vertical normal conso- lidation stress (Fig. 4). If a drained test, with - -- small shear stress increments, were performed, keeping σ_1 constant, it is accepted that the obtained shear - strength would be s_{d_1}. This value defines the strength law in normally consolidated soil as a straight - line by the origin. If a undrained consolidated test - where performed to the same soil with the same total -- normal stress σ_1, it is accepted that s_{u_1} represents -- the obtained strength and with this value a - - - strength law for normally consolidated soil in undrai-- ned consolidated condition can be obtained. This is -- also a straigth line through the origin. In Fig. 4 it can be seen the positive u value that has built up du-- ring the undrained test as well as the corresponding -- effective stress, $\bar{\sigma}_1$, for the case.

Considering total stresses, the undrained - - strength is smaller than the drained one, but of course the strength law is the same in terms of effective - -- stresses. This law is

$$s_{u_1} = (\sigma_1 - u) \tan \phi = \bar{\sigma}_1 \tan \phi$$

It can be said that the initial "potential -- shear strength" of the clay under σ_1 decreases in the - undrained test due to the positive pore pressure built - up.

The positive pore pressure built up during -- the undrained test has decreased the initial "potential strength" of the clay related to σ_1.

Consider now the same clay, but in a highly - overconsolidated condition. If this clay is subjectec to a vertical normal stress σ_2, the corresponding - - - drained shear strength would be s_{d_2}. In an undrained test on the same vertical normal stress σ_2, the - - developed pore pressure will now be negative and the -- exhibited undrained shear strength, s_{u_2}, will result to be greater than s_{d_2}. By extending to this case the - reasoning made to the normally consolidated case, it is usually said that the available initial "potential -- strength" in the clay was increased because the negati- ve pore pressure. This is not fully correct, because - as it can be seen in Fig. 4b, in drained or undrained - condition, the initial "potential shear strength" of -- tha clay, before any shear stress is applied, must - -- include an stored energy, measured by:

$$\sigma_s = \sigma_e - \sigma_c$$

Where:

σ_s, is the "stored stress"

σ_c, is the applied normal stress, assuming - - that the clay sample was first loaded to a maximum pressure σ_p, and then unloaded - - under drained conditions to σ_c. s_o σ_c is a applied stress in a overconsolidated sta te.

σ_e, is a stress corresponding to a point in -- the virgin branch of the compression curve, having the same void ratio than point A in the swelling branch. Physically, σ_e means the stress that the soil structure "feels" when σ_c is applied.

The "potential shear strength", before any - - shear stress is applied, may then be expressed as:

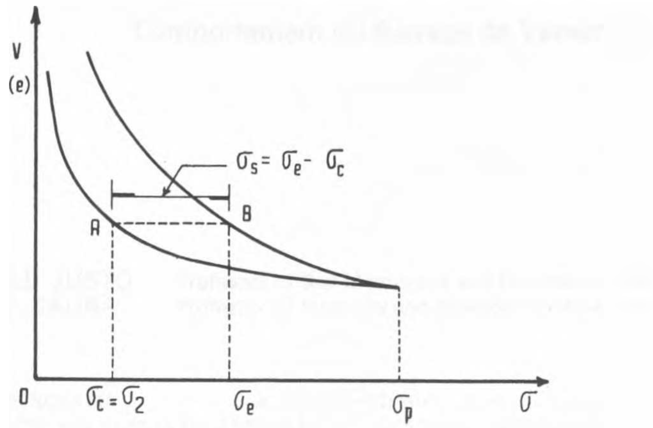
$$s_2 = (\sigma_e - \sigma_c) \tan \phi + \sigma_2 \tan \phi \qquad (6)$$

Because no shear stress has been applied until now, no - pore pressure has been developed in the clay.

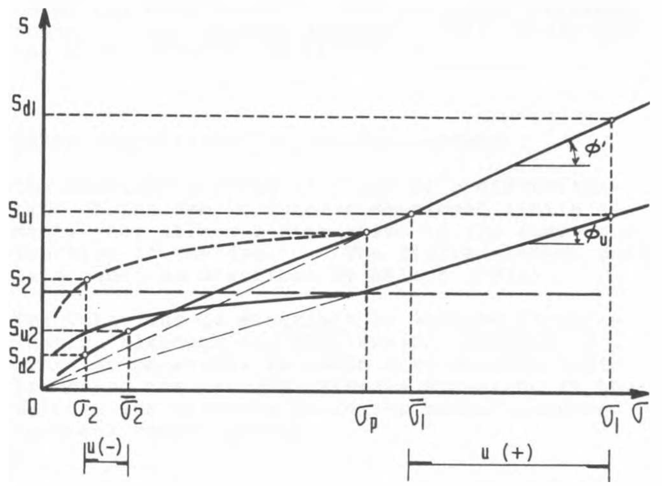
In drained test, the application of shear - -- stresses disturbs the soil structure and the "stored - - energy" produces a negative pore pressure, diminishing - as the sample expands; so the "potential strength" s_2 -- decreases to a final value:

$$s_{d_2} = r\,(\sigma_e - \sigma_c) \tan \phi + \sigma_2 \tan \phi \qquad (7)$$

Where r is a number less than 1, because the - stored energy is not completely vanished at failure (in the failure plane).



b. typical virgen compresion and expansion curves



a. typical shear strength envelopes

4. Drained and undrained strength of normally and preconsolidated clays.

In undrained tests, the swelling tendency is - restrained by the pore water tension (negative pore pressure). The stored energy liberated by structure - - - breakdown in the failure plane is now absorbed by the water tension, resulting in a reduction of the initial potential strength to the value:

$$s_{u_2} = r\ (\sigma_e - \sigma_c)\ \tan\phi + \sigma_2\ \tan\phi \qquad (8)$$

beeing $\bar{\sigma}_2$ greater than σ_2 because the negative pore pressure. So the shear strength component due to - the really applied effective stress has increased, but - the shear strength component due to the stored energy — has decreased, beeing initially due to $\sigma_e - \sigma_c$ and at - - failure only to a small fraction of that value (according to the r value).

Measurements made in drained and undrained - - triaxial tests on Weald clay (Juárez-Badillo, 1969) - - - showed a r value of 0.04 for compression tests and 0.06 for extension tests.

It can be seen as in drained and undrained - - tests made on highly overconsolidated clays, the potencial shear strength always decreases, with a greater decreasing in the first type of test. But the greater - - undrained strength there is not due directly to the development of a negative pore pressure. In both cases, - drained and undrained strength diminished from the potencial initial value s_2 (Fig. 4.a).

This fact could be of practical importance - - according to the authors thought. Many colleagues consider that negative pore pressures will built up in highly overconsolidated clays, with the corresponding increase in the available soil strength. So, the adoption of a - low safety factor in a specific earth work seems to be - justified. Of course, this is not real considering the future of the earth work, because the available potential strength will always decrease and also the safety - factor. The eventual failure will be confuse if it is - not explained in terms of a decreasing of the available shear strength.

The consideration of stored energy effects in highly overconsolidated soils, for explainning shear - - strength generation and evolution is also useful when -- the differences between the real lifetime of a natural - slope and that obtained from laboratory tests are analyzed. The loss of stored energy in fissures (a normal — condition in overconsolidated soils forming slopes) will produce a decreasing in the potential strength that - -- explains a safety factor evolution less favorable than - the one obtained from laboratory tests analysis.

REFERENCE

Juárez-Badillo. (1969). Failure Theory for Clays .- 7th Int.Conf.of Soil Mechanics and Foundation Engineering.- México