On the Britteness of soft Clays and its Effect on Slope Stability

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General (Introduction)

Many of the extensive landslides which have occurred in Sweden cannot be readily understood by means of conventional analysis based on the concept of plastic failure. It is true that in many cases the slides have been triggered by man-made operations of unknown intensity (piling, fills etc.), but even in hindsight it has often turned out to be very unrewarding to attempt to correlate the actual extent of a slide by back-analysis in accordance with current procedures.

At Rollsbo [3] the safety factor (undrained analysis) was e.g. found to be 2.3. For the original slope at the Tuve slide the Swedish Geotechnical Institute (SGI-varia no 56 [2]) has evaluated safety factors with respect to slides of some length ranging from 2.2–2.6, thus pin-pointing the capriciousness of the conventional approach.

In references [4] and [6] Bernander & Olofsson have shown qualitatively and quantitatively how a minor disturbance acting in a slope of strain-softening material may trigger a progressive failure which eventually propagates into vast areas of - even horizontal ground.

The analysis according to [4], [5] and [6] involves studying the effect on the slope of an additional force (N) arising e.g. from increased active earth pressure \( A E_p \) on account of fills, piling activity, strain accumulation due to non-uniform creep [7] etc.

If the residual shear resistance of the soil is less than the shear stress corresponding to the slope inclination, there will always exist a characteristic mobilizable resistance \( N_{SR} \) (Fig. 1 and 2). If \( N_{SR} \) is exceeded (Fig. 1 and 11) a progressive failure is initiated which – at sustained additional load \( N \) – will terminate only when \( N_{SR} \) on account of changing conditions, again balances the algebraic sum of downslope forces and the total shear resistance.

The risk for progressive failure can thus be put as (see ref. [4]).

\[
F_c = \frac{N_{SR}}{N} = \frac{N_{SR}}{A E_p}
\]

In the very same way as \( E_p \) constitutes the passive Rankine earth resistance corresponding to the failure surface BC, \( N_{SR} \) stands for the passive resistance along the failure surface BD (Fig. 2) when the deformations due to compression in the potentially sliding soil are considered.

For ideally plastic soils \( N_{SR} \) will always be greater than \( E_p \). However, in a strain softening soil \( N_{SR} \) may - depending on slope inclination and brittleness etc. - be considerably less than \( E_p \) in which case failure will take place along zone BD instead of BC. Thus, if the compression of the soil is regarded, failures in brittle soils will tend to develop along the bedding planes (more or less parallel to the ground surface or firm bottom) rather than along cylindrical slipsurfaces such as line BC.

The Bernander & Olofsson approach has at least the undisputable merit of making it possible to define the theoretical limits of the validity of the plastic failure concept with respect to the deformation characteristics of the soil. Inversely if these characteristics are known for relevant strain rates the initiation and the extent of a slide may be predicted (Fig. 11).

Drained or undrained parameters?

Long term stability for a slope may be checked by drained analysis in which case the conventional approach with plastic failure ought to be applicable. However, when failure occurs in a natural slope the triggering agent is usually of temporary nature (increased pore water pressure due to rainfall, fills, piling etc.).

But even in the case when a local failure is of a drained character the subsequent rupture will take place under increasingly undrained conditions. Thus when analysing failure mechanisms, failure propagation and the ultimate extent (the degree of disaster) of potential slides it is necessary to use undrained parameters.

Fig. 1. Typical results and definitions in progressive failure analysis (uniform slope). See also Fig. 11.

\[ \tau_c = \text{prevailing shear stress corresponding to the down slope forces} = \frac{\varphi \cdot g \cdot H \cdot \tan \beta}{dE_p/dx}, \text{where } \beta = \text{slope angle} \]

Fig. 2. Mobilizable 'passive' resistance, \( (N_{SR}) \) in a slope if deformations in the soil are considered.
which are relevant to the strain rates at stake.
A slope may therefore remain stable under drainage conditions for thousands of years and yet be potentially unstable for any agent inducing undrained behaviour in the soil material if this is strain softening. Progressive failure may occur even when the conventional safety factor (plastic failure) is considerable. (Fig. 11a and b).

Brittleness of soft clays in direct shear
Potential instability due to a progressive failure mechanism is possible for
\[
F = \frac{c_u}{\tau_{\text{mean}}} \quad (\text{conventional safety factor})
\]
\[
c_u = \text{undrained (short term) shear strength}
\]
\[
c_u' = \text{undrained residual (large strain - high stress rate) shear strength}
\]

Thus if the ratio \(c_u'/c_u\) is 0.5 progressive failure can only be excluded when the conventional safety factor \(F\) exceeds 2.0 everywhere in the part of the potential failure zone, where the failure may be initiated.

The conclusion of the aforesaid is obviously that the relationship between the residual shear resistance \(c_u'\) and the shear strength \(c_u\) must be an important target for future research. At the SKANSGA geotechnical laboratory (Gothenburg) an investigation has been carried out in order to give a background experience of how to perform new types of routine shear tests designed to reflect the behaviour of soft clays in planar landslides more accurately than the current standard test procedures.

The shear tests have been carried out at different strain-rates and for various values of OCR (overconsolidation ratio) as these two factors were expected to have a decisive effect on the ratio \(c_u'/c_u\). (Bernander, [1], [2]). Thus the residual shear strength \(c_u\) does not have a unique value which is characteristic for the soil material in question – a complication which, however, also applies to the undrained shear strength \(c_u\).

Stress-strain relationship in direct shear
The failure mechanism in a potential failure zone of a planar slide is taken to be as follows:

Stage I
Initially the material in the potential failure zone is sheared more or less uniformly as shown in Fig. 4b. The total displacement in the failure zone is then
\[
\delta_z = \int_{0}^{h} \tau_{\text{eff}} \cdot dz = \tau_{\text{m}} \cdot h
\]

where
\(\tau_{\text{eff}}\) is mainly a function of the shear stress, OCR and the strain rate. (Other notations are defined in Fig. 1 and 4).

Stage 2
At some point a slip surface is formed – an event usually accompanied by a gradual loss of shear strength. The displacement now has two components – one due to deviatoric strain in the material and one due to the slip in the slip surface itself \(\Delta \delta_s\). (Fig. 4c).

\[
\delta_s = \int_{0}^{h} \tau_{\text{eff}} \cdot dz + \Delta \delta_s
\]

\[
\approx \frac{\tau_{\text{m}}}{G_o} \cdot h + \Delta \delta_s = \gamma_{\text{eff}} \cdot h
\]

where
\(G_o\) = the elastic modulus in shear
\(\gamma_{\text{eff}}\) = apparent or effective angular deformation
\(\tau_{\text{m}}\) = mean shear stress in the failure zone.

In a strainsoftening material the stress level will drop to lower values at which a more or less linear relationship applies between stress and deviatoric strain. \((G_o)\). The residual shear stress in the slip surface, however, will tend to be highly dependent on strain rate owing to the interaction between pore pressure build-up in the slip surfaces (hydraulic lubrication) and the rate of dissipation of these excess pore water pressures. It should be noted that the magnitude of these high local pore water pressures cannot be studied in conventional apparatus for undrained testing as with such gear only the mean pore pressure rise in the sample can be registered. (See Fig. 5c).

Synopsis
The stress-deformation relationship in direct shear is thus composed of two ranges which differ in principle – in the first range the deformation is composed of deviatoric strain and in the second it is dominated by the slip in the slip surface.
itself. Hence the stress deformation properties of the failure zone can be expressed in terms of the displacement (εδ) or the 'apparent' deviatoric deformation (γeff = δh). The relationship constitutes basic data for stability analysis according to the concept of progressive failure as described in references [2] through [6].

Description of tests
Most of the tests carried out in this investigation are shear box tests (Fig. 5a). This has been found appropriate as our interest in this case has been mainly focused on the residual shear strength properties in the slip surface proper at high strain rates.

In some tests the sample was contained in an extremely flexible spiral as in Fig. 5b. The clays have been tested for various values of OCR and at different strain rates. Four types of clay have been tested, the properties of which are seen in Table 1.

The investigation shows that the residual shear strength is highly dependent on strain rate and on the overconsolidation ratio (OCR). At lower strain rates δδ/dt, high OCR values (i.e. low effective stresses) tend to give low residual shear strengths and low peak values. (Fig. 6). (Cf. Ladd & Foot [16], Lefebvre, La Rochelle [17]). At high strain rates (slip rates) on the other hand the residual shear strength becomes rather independent of the prevailing effective stress (as expressed by OCR)—at least the mentioned tendency at low strain rates is hard to detect at higher strain rates. (Fig. 8).

Nevertheless, the general trend for the residual shear strength (σrs) to decrease with growing strain rate is very pronounced for the investigated clays and seems to be a more crucial parameter than the inherent clay characteristics themselves—including sensitivity (Fig. 7).

Very low residual shear strengths have been found for soft clays at the tested strain rates, which—although they are considerably higher than those commonly used in direct shear tests—are still very far from the strain rates actually occurring during the evolution and in the final phase of a landslide. (Fig. 8). The Fig. 9 and 10 refer to results from measurements made in the field when a pipe is being jacked through clay. The diagrams clearly show that the mobilizable shear stress in the soil is strongly dependent of the dimensions of the structure—in this case the length of the pipe. These phenomena underline the importance of considering deformations also in soil mechanics.

Consequences of brittleness in clays with regard to landslides
The stress deformation relationship for clay in direct shear can be expressed in terms of the displacement (εδ) or the apparent angular deformation Yst (= θ/h), and constitutes the basic data for stability analysis according to the concept of progressive failure as described in ref. [2] through [6]. In these reports the brittleness ratio c/s/σc has been found to have a most decisive influence on the initiation, propagation and ultimate extent of a slide. Fig. 11a and b.

As can be seen from Fig. 11a, the question as to whether the c/s/σc ratio is 0.55 or 0.40 constitutes the crucial issue with regard to the degree of disaster of a potential failure in that particular slope. For a value of c/s/σc = 0.55 a failure in the upper part of the slope will result in a minor slip (causing a moderate crack), whereas for c/s/σc = 0.40 a major disaster will take place heaving the ground in the passive zone over some 250 m. The stress deformation characteristics derived from the tests in this investigation indicate that the brittleness ratio c/s/σc may attain values as low as 0.3 for soft clays at strain rates, which are relevant to landslides. This emphasizes the necessity of taking the time factor into account when evaluating the shear strength properties of soft clays. Brittleness ratios of this order readily explain the large planar landslides in Sweden. The risk for a local failure and in particular the final configuration and extension (the 'scope of disaster') of a potential landslide can thus be predicted by means of progressive failure analysis based on stress deformation data, which are relevant to the actual strain rate during the slide. Nst is the decisive parameter for...
Fig. 8. Britteness ratio \( c'/c_u \) as a function of strain rate. The value of \( c' \) is related to \( OCR=1 \).

Fig. 10. Jacking of two sections of a concrete pipe in clay—separately and both together.

Fig. 11. Progressive failure analysis of two slopes having the same conventional factor of safety vis-à-vis failure (along AB).

Predicting the risk for initiation of a slide. As the strain rate during initiation of a slide is not well defined it is a fortunate circumstance that the value of \( N_{SR} \) (Fig. 2) depends more on the slope inclination and on the modulus of elasticity of the soil than on the brittleness ratio. Thus \( N_{SR} \) may be computed with reasonable accuracy even with a rough estimate of the brittleness ratio.

Progressive failure analysis—along with conventional methods—is now a routine approach in the assessment of slope stability at SKANSKA design office in Gothenburg. Fig. 11 shows a comparison between the results from such nonlinear failure analysis for two different slopes having the same conventional safety factor (1.90) with respect to extensive sliding. The two slopes have the same mean stiffness and differ only with regard to the shape of the contour defining firm strata. Hence brittleness in slopes may even be more a question of slope geometry than of brittleness in the soil itself.

If the increase in active earth pressure by some agent is \( AE_p = 50 \text{kN/m} \) slope C will fail \( AE_p = 33 \text{kN/m} \) whereas slope A will remain stable \( \text{F}_{DC} = N_{SR} \).

Thus \( \text{F}_{DC} = \frac{167}{50} = 3.3 \)

As can be seen in Fig. 11b (slope C) the downslope forces will exceed passive earth pressure creating a passive Rankine zone over a length of some 240 m with due disastrous consequences.
If, however, the disturbance force—for some reason—should exceed 167 kN/m in slope A ($c_1^*/c_m^*=0.5$) the downslope force $N$ will hardly exceed passive Rankine pressure, in which case failure will only result in a moderate slip causing only cracking in the active zone.

Resumé

The stress–deformation characteristics derived from the tests in this investigation indicate that the brittleness ratio $c_1^*/c_m^*$ may attain values as low as 0.3 for soft clays at strain rates, which are relevant to landslides. Brittleness ratios of this order readily explain the large planar landslides in Sweden. The risk for a local failure and in particular the final configuration and extension (the ‘scope of disaster’) of a potential landslide can thus be predicted by means of progressive failure analysis based on stress–deformation data, which are relevant to the actual strain rate during the slide.

The progressive failure analysis reveals inherent properties in a slope, the nature of which cannot be evaluated by the conventional approach. This is of course a serious matter as e.g. one of the studied slopes in Fig. 11 is quite stable, while the other treacherously conceals a potential disaster. Yet, both have the same conventional safety factor of 1.90. Thus the importance of considering the time factor in analysis of slope stability cannot be overestimated.

References