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On the long-term strength of undisturbed cohesive soils

Sur la résistance durable des sols cohérents non-remaniés

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SYNOPSIS A new, simplified method is proposed for the determination of long-term strength. The method makes use of the results from triaxial tests. A number of analyses were carried out with the aim to justify the validity of the solution suggested. The data obtained substantiate the importance of soil deformations in mobilizing the shear resistance, which is essential in the case of inhomogeneous soils.

INTRODUCTION

The description of soil strength in terms of the Coulomb and Terzaghi criterion requires that the soil strength characteristics be determined separately for each case of loading history and time. This means that soil strength is described by various parameter values, depending on the stage of construction or the performance of the object. The parameter values of interest should be measured in test procedures simulating actual conditions as accurately as possible. The problem raises particular difficulties when the determination includes long-term strength parameters. In engineering design, a simplified approach is practiced, in which long-term strength is identified with residual shear resistance. But this approach cannot be considered acceptable either from the theoretical or from the economic point of view. It cannot be recommended, either, because residual shear resistance, which is associated with a large post-failure displacement, depends on other structural processes than does long-term strength. In fact, long-term strength describes the maximum stress, at which no failure occurs irrespective of the time elapsed.

COMMENTS ON THE METHODS OF TESTING LONG-TERM STRENGTH

The available methods make use of the results from creep tests. In this methods, the loading at which the stress-strain rate relationship loses its linearity is identified with the state of long-term strength (τ_T). The weak points of the methods in question refer to the adopted interpretation and to their possible uses in engineering practice. The accepted criterion for the determination of the long-term strength confines the possible soil loadings to the range at which soil behaves as a linear body. And this means practically that the soil loadings mentioned are not large, because the predominant feature of the soil is the non-linearity of the stress-strain relation. It is only

at less accurate approximations that the linear part may be expanded to include stresses corresponding to about 1/3 or 1/2 of the peak strength. However, no evidence is available that exceeding the linearity range means unavoidable failure of soil. Both literature reports (e.g., Bishop and Lovenbury 1969) and the authors own results indicate that even at a stress which approaches the peak strength (80-90%) no failure is found to occur in many undisturbed soils. Another shortcoming of the available methods of determining τ_T is their troublesome application to laboratory testing and the problems they raise in the interpretation of results. The highly heterogeneous structure of undisturbed soils makes impossible any objective interpretation. For illustration purpose, a typical example is shown in Fig. 1a. This same holds for the simplified Murayama-Shibata method (1966) of determining the upper yield value which is identified with the long-term strength.

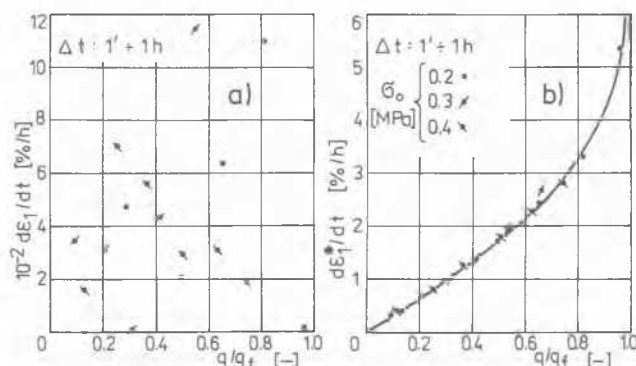
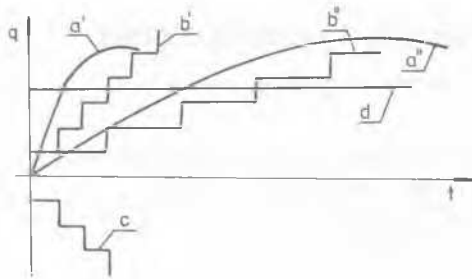


Fig. 1 Viscosity diagrams for (a) creep rate values, (b) average strain rate

Having these all in mind, the writers decided to develop such a method of determining the long-term strength value that would be of greater utility in practice. The method will aim at determining at least the lower limit of the range for the probable values of the long-term strength.



| Scheme | Testing method | Rates of strain or stress increments |
|--------|---|--------------------------------------|
| a | σ_3 constant σ_1 increased | 0.15-3.68 mm/h |
| b | σ_3 constant σ_1 increased | 2-176 kPa/h |
| c | σ_3 decreased σ_1 constant | 80-160 kPa/h |
| d | σ_3 σ_1 constant | creep test |

σ_1 and σ_3 are the major and minor principal stresses.

Fig.2 Scheme of test programmes

SCOPE OF INVESTIGATIONS,
DISCUSSION AND INTERPRETATION OF RESULTS

The investigations included three different undisturbed cohesive Tertiary soils. Taking into account the methods of testing and duration of tests, the programme of the experiments was highly differentiated (see Fig.2). The soils were examined by unconsolidated undrained triaxial compression tests (with measurement of pore pressure) on 38mm diameter and 76mm height samples. Cell pressure varied from 0 to 0.6 MPa. In the tests itemized as (b), (c) and (d) applied were 6 to 8 steps of loading, each of them amounting to about 15% of the peak value. Creep tests were carried out for 2 to 3 weeks. In addition, the unconsolidated 60x60mm cross-section specimens were subject to multiple shear box tests. Major physical parameters of the soils are given in Table I.

TABLE I
Index properties and other data

| Item to be measured | | Clay | | |
|---------------------|-------------------|------|------|------|
| | | S | L | P |
| Density of soil | kg/m ³ | 2080 | 2130 | 2090 |
| Water content | % | 22 | 20 | 22 |
| Liquid limit | % | 61 | 68 | 64 |
| Plastic limit | % | 20 | 27 | 22 |
| Clay fraction | % | 28 | 25 | 33 |

The sets of specimens differed in sampling method. Clay S samples were cut out of a soil

"block" exposed on a slope. The remaining samples were obtained from boreholes. The stresses are described by the following parameters:

$$q = 1/2(\sigma_1 - \sigma_3), p = 1/2(\sigma_1 + \sigma_3), p' = 1/2(\sigma'_1 + \sigma'_3) \text{ or } \tau \text{ and } \sigma. \text{ Axial strain is denoted as } \epsilon_1. \text{ Peak strength values are given the symbols } q_f \text{ or } \tau_f.$$

Irrespective of the type of soil and sampling method, the test results showed considerable scatter, which should be attributed to the inhomogeneity of the samples. As shown by Fig. 1a, the measured creep rate values made the interpretation of results impossible. In this situation, viscosity diagrams were expressed by average strain rates instead of transitory creep rates. The average strain rate for $t_i - t_{i-1}$ was determined as follows: $\dot{\epsilon}_i = (\epsilon_{i1} + \epsilon_{i-1}) / (t_i + t_{i-1})$. Although such a substitution poses certain limits to the interpretation of results, it enables, on the other hand, further analyses. In this way, a smooth distribution of results was obtained for all the samples of interest (Fig. 1b).

Being aware of the fact that investigations of this type would not give an explicit answer to the question of what was the long-term strength value, emphasis was placed on determining the lower limit of the probable range. Thus, the following hypothesis was put forward: the soil will not undergo failure in future if its strain rate is not greater than the one the soil would have if it behaved as a linear body up to the moment of failure. This means that the stress corresponding to point A in Fig. 3 has been adopted as the long-term strength. Point B (Fig. 3) represents the stress which has been identified so far with the long-term strength. To make a distinction between these two strength values, the q_f symbols were given subscripts A and B, respectively.

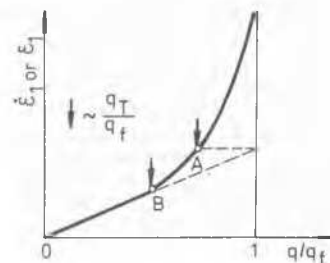


Fig.3 Graphical representation of stresses identified with long-term strength

Since viscosity diagrams, expressed in terms of the average strain rate ($\dot{\epsilon}_i$) are similar in geometry to the strain-stress diagrams, we can define the long-term strength on the $q-\dot{\epsilon}_i$ curves as well. Attempts were made to prove the validity of the hypothesis both by theoretical interpretation of results and by intuitive premisses. The latter are referred to the stress and strain values at which critical states were observed and to the residual shear resistance values for the investigated soils. All the averages are gathered in Table II. These data substantiate the pessimistic evaluation of the long-term strength resulting from the so-far adopted method. The values obtained via the 'old' procedure are almost equal to, or even less than, the residual shear resistance. The strain values of the critical state are also small and range

TABLE II
Stress and strain at critical states
(% of failure values)

| Clay | Point A | | Point B | | Residual resistance |
|------|-----------------|-----------------|-----------------|-----------------|---------------------|
| | q _{TA} | ε _{1A} | q _{TB} | ε _{1B} | |
| S | 72 | 40 | 48 | 18 | 55 |
| L | 60 | 25 | 42 | 14 | 50 |
| P | 70 | 37 | 39 | 16 | 42 |

from 0.03 to 0.41 of the strain value at failure (ε_f). When long-term strength is determined by the novel method, the strain values vary from about 0.08 to 0.60 ε_f. This means that the strain value is quite far from the failure value.

More convincing arguments for the hypothesis were sought by a theoretical interpretation of the test results. The interpretation includes such test results alone that were typical of the given test scheme and the problem considered. Making use of the data obtained in the stress-controlled test scheme (b), it is possible to plot the appropriate viscosity diagrams for different transitory strain rates. The inhomogeneity effect was partly eliminated by performing several test cycles on the same sample. An example is given in Fig. 4a. For comparison, Fig. 4b contains the plots of stress-strain relation with indicated critical points for these same samples.

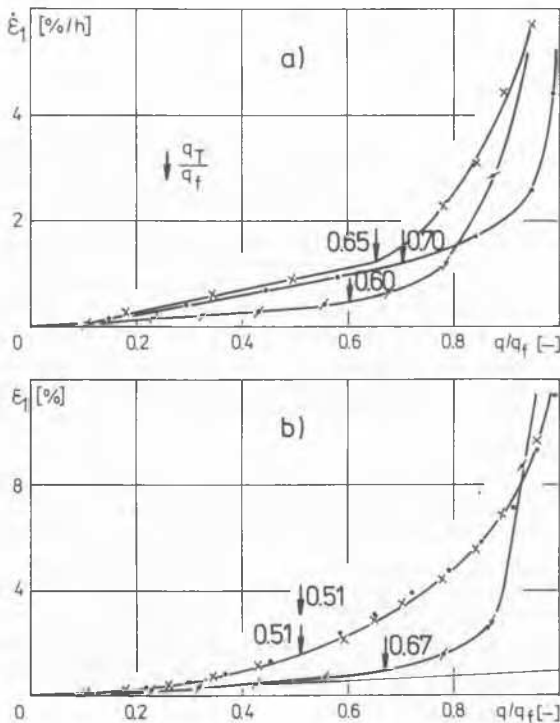


Fig. 4 Curves representing (a) $\frac{q}{q_f} - \dot{\epsilon}_1$, and (b) $\frac{q}{q_f} - \epsilon_1$ obtained in stress-controlled test (scheme (b)).

The viscosity diagrams show the structural changes of the material which occur in the course of the shearing process. As shown by these curves, the nonlinear increment of the strain rate occurs at a stress of about q_T. In the case of clay L, the average value amounted to 0.6q_f. Assuming that the increase of the strain rate represents the increase of the internal response of the material, we have to anticipate that there will be no failure until such an increase takes place. And this seems to be an indication that the proposed method of determining q_T is at least admissible.

The results obtained from this same stress-controlled test were used for estimating the long-term strength according to Murayama-Shibata's suggestion (1966). The estimation makes use of the analysis of the ln q - ln ε₁ relation. However, this estimation method was found to be insufficient (Fig. 5), because for a number of samples the first bend point of the stress-strain curve appeared at very low stresses (0.3 q_f and 0.35q_f for clay L and clay S, respectively).

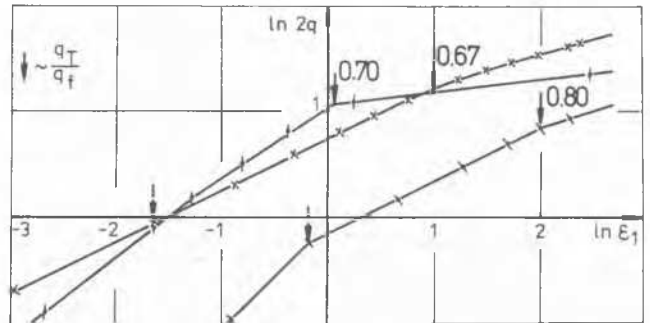


Fig. 5 Curves of stress-strain relation obtained at different rates of stress increment (scheme (b))

Thus, it would be unreasonable to identify them with long-term strength. On the other hand, these same results confirm the non-linear behaviour of the soils tested.

The results of strain-controlled tests (scheme (a)) were interpreted in terms of the notions and definitions suggested by Dmitruk, Lysik, Suchniok (1973) for the description of mechanical processes occurring in soils. To characterize the loading applied, a new dimensional quantity, i.e. the stress density, defined as $g = \int_0^t \frac{\partial \sigma(\xi)}{\partial \xi} \frac{H(t-\xi)}{t-\xi} d\xi$, has been introduced. The notation is as follows: σ(ξ) = load-realization function determined in the interval 0 < ξ < t, t = time of observation, H(t) = Heaviside function, ε = sufficiently small positive number. The integral of the stress density function in relation to time, G = ∫ g dt, has the same dimension as does the stress, and represents the history of loading. Adopting ε₁ as the indicator of the internal response of the material, the G - ε₁ relation may be regarded as the mathematical model of soil. The loading values at which a nonlinear increase of the strain (associated with the acceleration of internal response) is observed, are critical values. Once they are exceeded, we

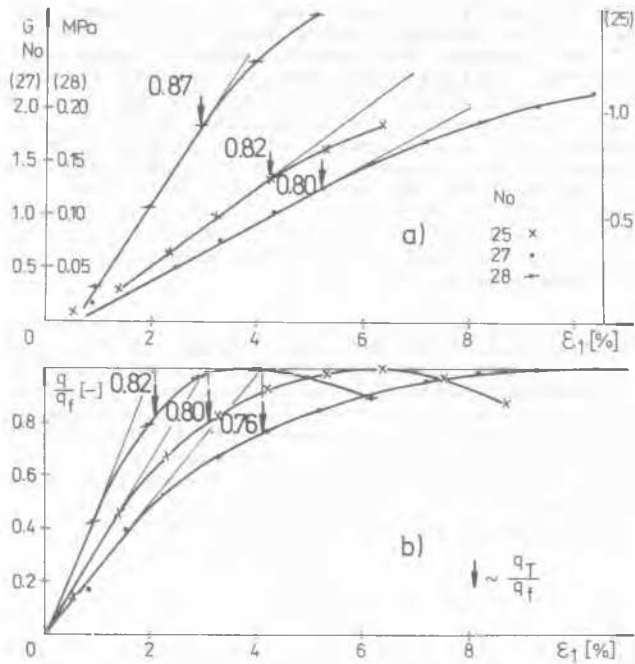


Fig.6 Curves representing (a) $G-\epsilon_1$, and (b) $q/q_f-\epsilon_1$ obtained in strain-controlled test (scheme (a))

may expect failure in future. Figure 6a gives the typical plots of the $G-\epsilon_1$ relation for clay S.As shown by these curves, the stresses at which these relations are no longer linear in nature, correspond to the load values that are somewhat higher than those suggested for q_{TA} (generally at $0.78q_f$, long-term strength, q_{TA} , being $0.75q_f$).

Considering the results presented here, as well as a number of other results which have not been discussed in this report, we can assume that the changes observed in the experimental plots (which are associated with the structural changes of soils) occur at loadings equal to, or somewhat greater than, the proposed long-term strength values. This is an indication that our new method yields safer results. More details are given in an unpublished report for internal use, or are under preparation.

THE STRENGTH OF INHOMOGENEOUS SOILS

Random inhomogeneity of undisturbed soils is indicated by a differentiated plot of the stress strain curves representing the same ground. An example is given in Fig.7. From these curves it becomes obvious that the estimation of strength parameters involving peak strength values is too optimistic. As soil exhibits various degrees of deformability, local yield may appear even at average shear stress lower than is the long-term strength. This yield is accompanied by a loss in shear resistance. In such case, the long-term stability of the structure need not be determined by the long-term strength, but may be the consequence of a progressive failure. To examine such failure hazard, it is advisable to find such a $q-\epsilon_1$ relation that will be representative of the soil considered. Thus, we should distinguish in the data set a number of subset represent-

ing similar behaviour of the $q-\epsilon_1$ relation. The ordinates of the ideal stress-strain curve can be calculated as follows: $\bar{q}(\epsilon_{1i}) = \sum \alpha_i q_i(\epsilon_{1i})$, ($p=c$), where q_i denotes the shear stress value in the subset for strain ϵ_{1i} , and α_i indicates the percentage of soil behaviour, i , in the ground. The curve obtained via this route should be made use of primarily for the improvement of the peak strength values involved in the analysis of short-term stability of structures. The curve may also be of utility for the estimation of the long-term strength by the method proposed (Fig.7). In this case, as well as in the case of brittle soils (soil type (a) in Fig.7), it is necessary to consider whether the long-term stability of the object is influenced by the long-term strength or by the progressive failure. Such an analysis, along with an appropriate choice of the safety factor, is an effective aid in performing the idealization suggested.

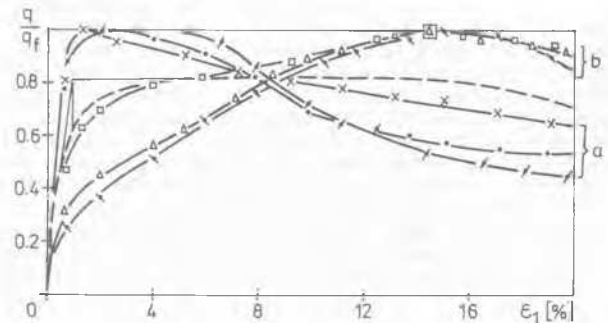


Fig.7 Curves of stress-strain relation of inhomogeneous soil

CONCLUSION

- (i) The tests show that inhomogeneity is an important feature of undisturbed soils. The considerable scatter of data makes their interpretation very difficult.
- (ii) The proposed simplified method of determining the long-term strength in standard triaxial tests is sufficiently safe to be employed in engineering practice until more accurate solutions are at hand.
- (iii) Emphasis should be placed on the importance of the state of deformation in mobilizing the shear resistance.

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