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REGULARITIES IN THE SHEAR BEHAVIOUR OF PLEISTOCENE AND PANNON SOIL COMPLEXES ALONG THE DANUBE RIVER

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Abstract

Aim of this study was to produce and analysis about the regularities of shearing properties of soils, identified by routine testing methods for the sequential soil masses of the Pleistocene and Pannon escarpment walls along the western bank of the Danube. The study was able to prove that significantly more information can be achieved to about the shearing properties of individual soil samples and sequential soil masses on the basis of known and presently introduced failure criteria.

These considerations renew the thesis that the shearing resistance and the relevant Φ, and c are picked out or guessed independently of each other.

Even a single test may produce results in close affinity to literary data.

Keywords: shearing properties, soils, failure criteria.

1. Introduction

The Geotechnics Department of the Technical University of Budapest completed several soil investigations in the past five to six years along the western escaping bank of the Danube (in the districts of Százhalombatta, Rácalmás and Dunaujváros) for various engineering jobs (like the foundation of a chimney, dredging of the river bed, stabilization of slopes, design of working pit supports, etc.). These jobs required but routine investigations and tests to be made.

The area is well described in engineering geological studies made in earlier times (SCHMIDT, 1964; PÉCSI, 1971; KARÁCSONYI – SCHEUER, 1976; SCHEUER, 1979). These studies were motivated mostly by the investigation of unwanted soil movements and contained hardly any information about soil mechanical properties (shear strength, for example). Numerical information about soil conditions can be extracted almost exclusively from the relevant reports of professor KÉZDI (1970).

The above mentioned preliminaries incited the desire to make use of the available routine testing results by assembling the most prominent and almost general regularities which describe the shearing conditions in the given soil types.

2. Conditions of Failure Tests

A triaxial C_U test of small diameter samples is carried out generally under the following in the Laboratory of the Geotechnical Department of TUB:

sample size: $d = 3.8$ cm, $h = 7.5$ cm;

consolidation in saturated state under isotropic stress conditions;

loading rate $v \leq 1$ mm/min;

load up to failure takes place in undrained condition, with or without pore pressure measurement;

all measured data will be registered, stored and evaluated automatically by the computer (system elaborated *by* Mr. A. Domokos, electrical engineer);

data processing may take place either in the function of total stresses (σ_1, σ_3), or effective stresses (σ_1, σ_3).

For plotting the results, both Mohr's (σ, τ) and LAMBE's (1967) system (p, q) are used. As it is known, ROSCOE's (1958) coordinate system (p, q) has not gained space in the Hungarian practice. The straight break lines, i.e. the Φ and c parameters, are generally evaluated under the following conditions:

T: $1/2(\sigma_1 - \sigma_3)_{\max}$;
H: $1/2(\sigma_1 - \sigma_3)_{\max}$;
$1/2(\sigma_1/\sigma_3)_{\max}$;
U: $\pm V = 0$, where $S_r < 1$
$U_{\max}$, where $S_r = 1$.

A customary representation and the physical principles of the failure condition are shown in *Fig. 1*.

Beside the first two failure conditions well known from the literature the physical principle of a third criterion has been introduced by the author, inasmuch the deformation has to be zero at the transition state, when the process changes from compression into expansion (see *Fig. 1b*).

When $S_r < 1$ condition exists, $\pm\Delta V = 0$; and when $S_r = 1$, U will attain its max. value at the same place. This physical condition has been confirmed by test results from ROWE (1962), and by several other authors: LAMBE, – WHITMAN, (1969); MITCHELL, (1976); CHEN, – BALLADI, (1985); LERONEL, – MAGNAN, – TAVENAS, (1990).

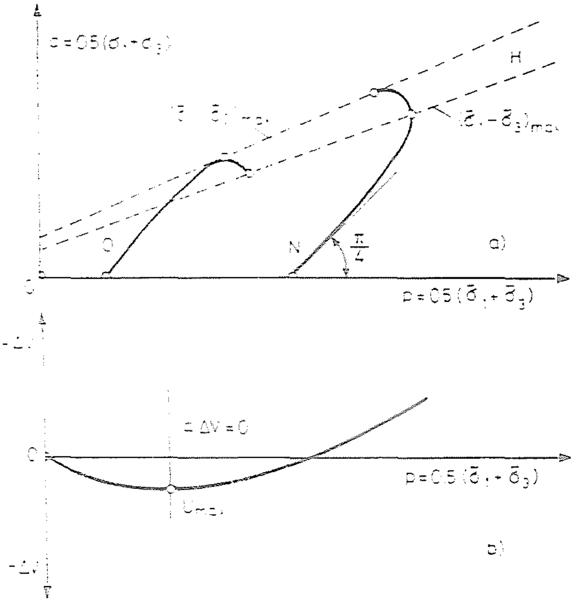

Fig. 1. Failure criteria applied in triaxial test:
a – two standard conditions,
(N) normally consolidated sample,
(0) overconsolidated sample,
b – the introduced third condition ($\pm\Delta V = 0$ or $U_{\max}$)

3. Test Results

For treating the jobs mentioned in the first paragraph, conventional strength tests were carried out (BANCSIK – JUHÁSZ – KABAI – NAGY, 1992; KABAI, 1986, 1991). Main identification characteristics of the samples used, their consistency parameters and grainsize distribution curves are summarized in *Figs. 2* and *3.*

Test results from the Rácalmás samples (1, 2, 3) and their shear parameters are given in the Φ, and c coordinate system in *Figs. 4* to *6*, showing the following results:

- average values and standard deviations of physical parameters ($\overline{w}$, $\overline{e}$, $\overline{\varrho_n}$, $\overline{S_r}$, c_v);
- average Φ and c values determined in accordance with the failure condition T, H and U;
- the regression line (a) determined on the basis of all available data;
- the regression line (t_r) adjusted to the expected values pertaining to the T, H and U conditions;

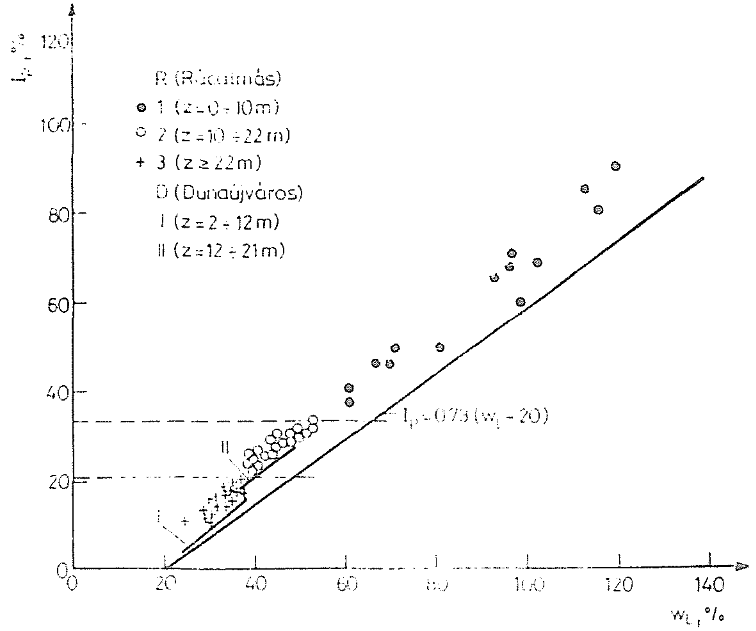

Fig. 2. Consistency parameters of the samples used in the tests in the plasticity chart: R – Rácalmás, D – Dunaujváros

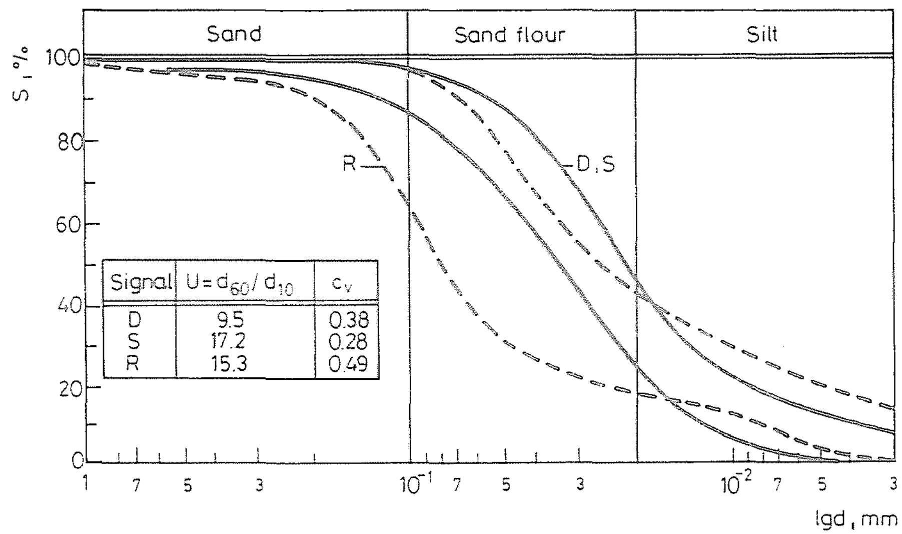

Fig. 3. Grading curves of the tested soils: R – Rácalmás, D – Dunaujváros, S – Százhalombatta

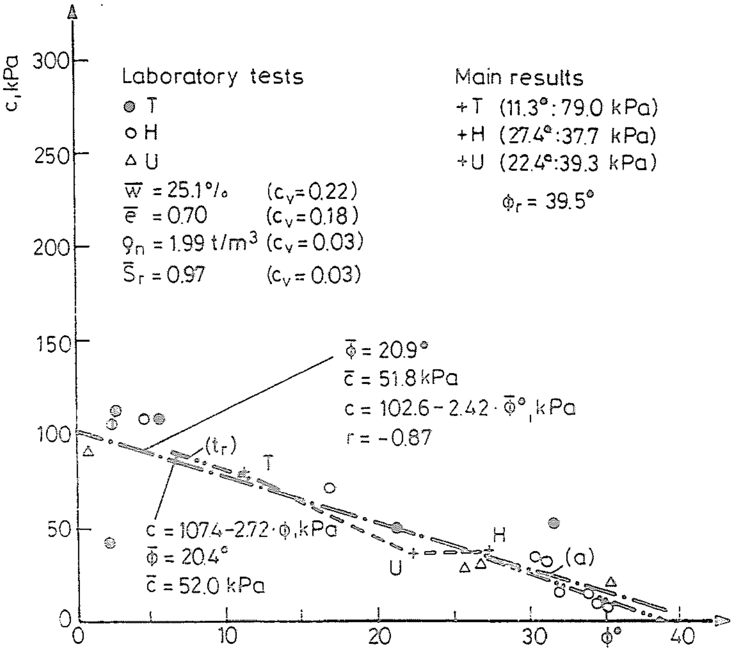

Fig. 4. Shear parameters of the samples in the 1. category:
T – total stress condition,
H – effective stress condition (acc. *Fig. 1a*),
U – effective stress condition (acc. *Fig. 1b*).

- lower boundary limit line (m) pertaining to the least values of the tests;
- the value Φ_r determined on the basis of the regression line t_r, whereby $c = 0$.

Presented evaluation technical indicates for even these extremely heterogeneous data that the limit values of the shearing process can be described in an appropriate manner ($\Phi = 0$, $c = c_{\max}$ and $c = 0$, $\Phi = \Phi_r$). This means that even a single test may supply valuable information about the relationship between the Φ, and c parameters within the boundary values of ideal shear behaviour.

Fig. 7 summarizes the test results of sample types D and S. Corresponding values of pertinent soil parameters and their standard deviations are compiled in *Table 1*.

For material represented by D (Pleistocene mass) also the variation of Φ and c parameters are presented to demonstrate the development of the shearing process.

Also in *Fig. 7* the single test result from KÉZDI (1970) is inserted, having been made in 1964 in connection with the investigation of the land-

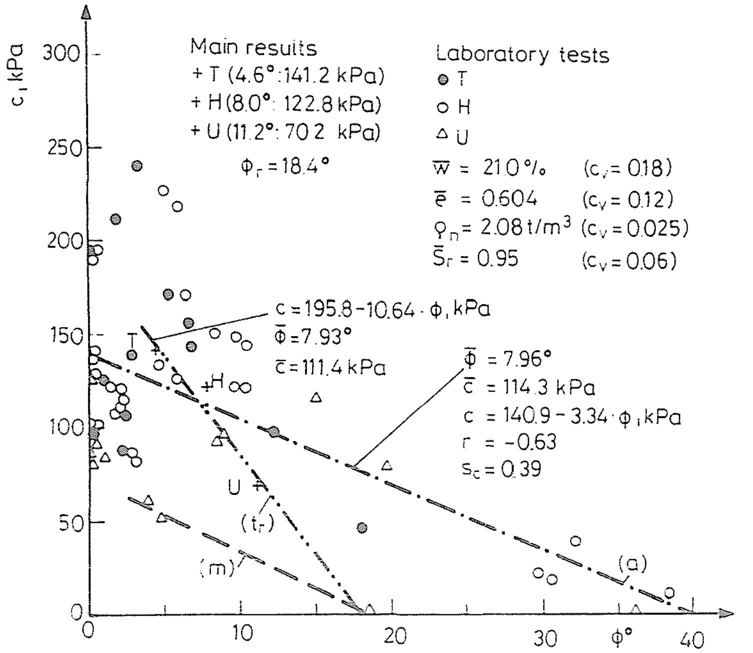

Fig. 5. Shear parameters of the samples in the 2. category.

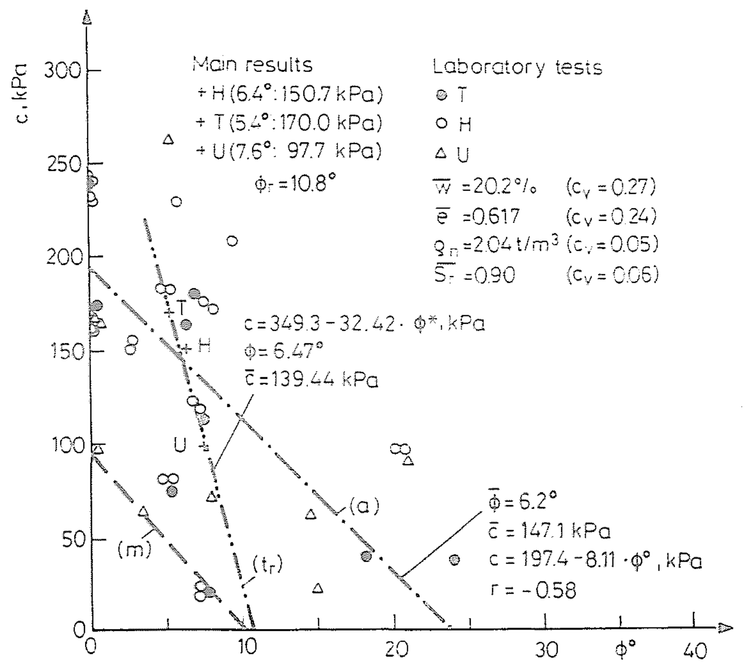

Fig. 6. Shear parameters of the samples in the 3. category.

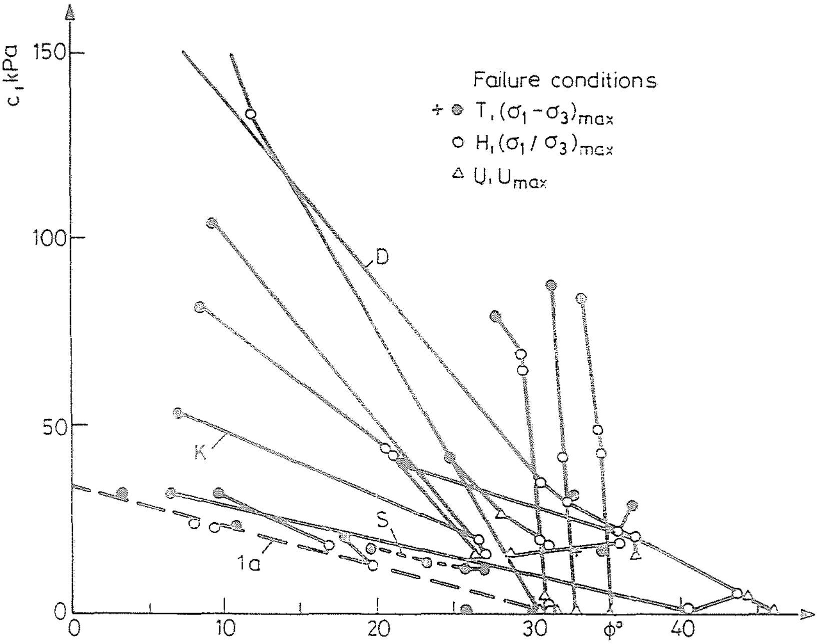

Fig. 7. Φ and c parameters for samples of the D an S categories, independently of breaking conditions:
1a– lower boundary limit,
K – Kézdi's test in 1970,
S – Samples from Százhalombatta in total stress condition for $s = 5$ to 25 mm displacements

slide of the river bank at Dunaujváros. Based on the test results of D-type samples also the critical boundary line (1a) could have been plotted.

If there were no possibility for measuring pore pressures but test results had to be plotted in the function of total stresses, Φ and c values can be determined in the function of constant vertical deformation. Such a result is shown by the line S which developed between the axial deformations of 5 to 25 mm. (Here only the end result is given without a detailed analysis of the procedure.)

The regression lines (t's) from *Figs. 4* to *6.* (see samples 1 to 3 from Rácalmás) are summed up in *Fig. 8*. For reasons of comparison here also the lines K and 1a from *Fig. 7* are reproduced. By extrapolating the t_r regression lines in *Fig. 8*, Φ_r and c_u values can be established together with the correlations between them, i.e. the shearing process is visualized. Based on these Φ and c values, an imaginary line (F) can be drawn, which infers the coherent values of c_u-s and Φ_r-s. Taking any arbitrary point (F_i) on this line, the relevant shearing line (i) can be determined. Such a

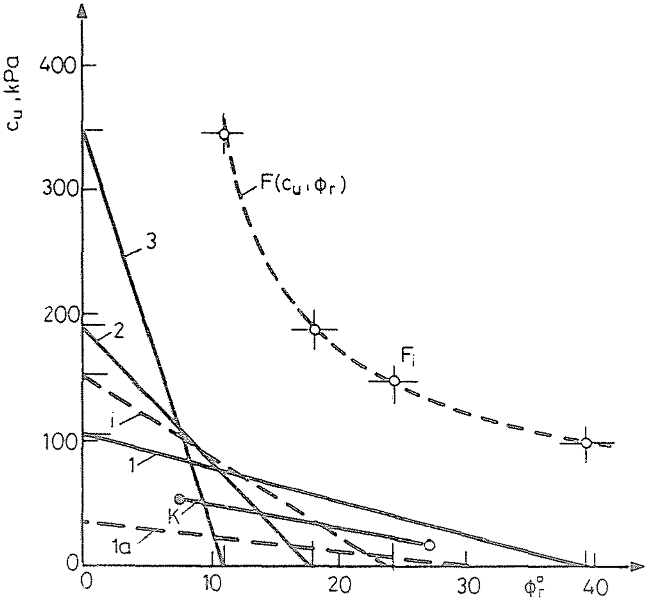

Fig. 8. Probabilistic shear lines for samples in the 1. to 3. categories, in the Φ_r and C_U coordinate system

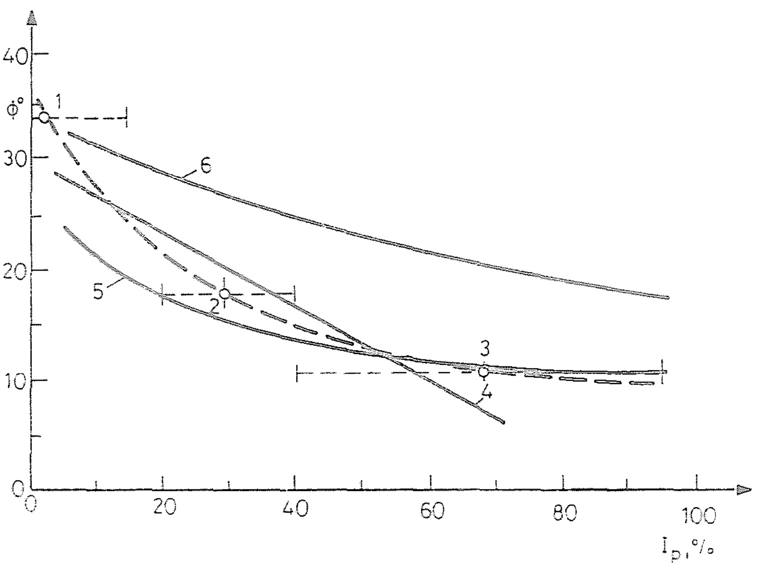

Fig. 9. Correlation between internal friction angle and plasticity index

Table 1
Expected values and variables of soil parameters, z = depths in m.

	$\overline{W}$, %	$\overline{\epsilon}$	$\overline{\rho}$, kN/m^3	$\overline{S_r}$
		c_v		
D	24.8	0.71	1.94	0.90
	0.14	0.08	0.04	0.0
S	20.7	0.70	1.91	0.83
	0.21	0.13	0.02	0.08

D – Dunaújváros ($z = 2 \div 21$ m),
S – Százhalombatta ($z = 2 \div 15$ m),

shearing configuration can be established theoretically in relation to any soil or sequential soil mass.

It has to be mentioned at this point, that this type of shearing configuration has also been attained in relation to the Lower and Upper Pannon sequential soil masses on the east bank of the Danube.

Similar shearing configuration can be attained for even a quasi-homogeneous soil, the Φ and c values of which were determined by various testing methods (in the shear box, by torsional shearing test, in the triaxial apparatus, etc.). Namely, not exactly the same process develops between the ideal shearing process and the ideal plastic boundary limits in the various testing equipment. These testing methods will not be dealt with, however, at this place.

Literature sets forth several achievements whereby the authors presented internal friction values in the function of the plasticity index. The following analysis made use of these correlations, summarized in *Fig. 9*.

1, 2, 3: average Φ_r values of the tested Pleistocene and Pannon soil masses, and relevant $\Phi = f(I_P)$ correlations;

4: SZILVÁGYI's (1955) simple function $\Phi = 30 - I_p/3$; on the basis of tested various clays;

5: DEERE's (1974) probabilistic average values: Φ_r: $f(Ip)$; (MITCHELL, (1976))

6: regression curve derived from available data in the literature;
$\Phi = 33.96 \times 0.993^{I}p$; I_p : per cent;
$r = -0.715$; $N = 130$ results.
$s_f = 0.195$;

It can be assumed that the presented elaboration technic is in good agreement with the data from the literature.

4. Summary

Aim of this study was to produce and analysis about the regularities of shearing properties of soils, identified by routine testing methods for the sequential soil masses of the Pleistocene and Pannon escarpment walls along the western bank of the Danube. The study was able to prove that significantly more information can be achieved to about the shearing properties of individual soil samples and sequential soil masses on the basis of known and presently introduced failure criteria. Even a single test max produce results in close affinity to literary data.

These considerations renew the thesis that the shearing resistance (and the relevant Φ and c parameters) cannot be taken as soil constants and, that spare esteem can be offered to statistical analyses where the parameters Φ and c are picked out or guessed independently of each other.

Completed investigations and the study confirm the assumption that the regional characteristic sequential soil masses would require more attention and would be a merit to carry out more detailed basic research studies thereupon, that the problem is treated presently in the technical literature.