NONLINEAR ROCKFILL PARAMETERS AND STABILITY OF DAMS

PARAMETRES NONLINEAIRES DES ENROCHEMENTS ET STABILITE DES TALUS

A. Marcu[1] *E. Luca*[2] *F. Craciun*[3]

[1]Professor, Civil Engineering Institute, Bucharest, Romania
[2]Professor, University of Agricultural Science, Bucharest, Romania
[3]Consulting Engineer, Design Institute for Land Reclamation, Bucharest, Romania

SYNOPSIS : On the basis of the results of about 700 direct shear tests and about 150 triaxial compression tests achieved on samples of large size, the variability of the shear strength parameters of some coarse-grained materials is rendered evident. Two laws of internal friction angle ϕ' variation with the normal stress σ' are studied, giving the values of the parameters of these equations for certain rockfill categories. The influence of angle ϕ' variability on the homogeneous dam slope stability is investigated.

INTRODUCTION

During the latest 20 years, more than 15 local material dams with heights H = 60 ... 175 m have been achieved in Romania. One of the features of such works is the important variability of the grain size of the rockfill, which, besides, sufers modifications due to transportation and compaction (Fig.1).

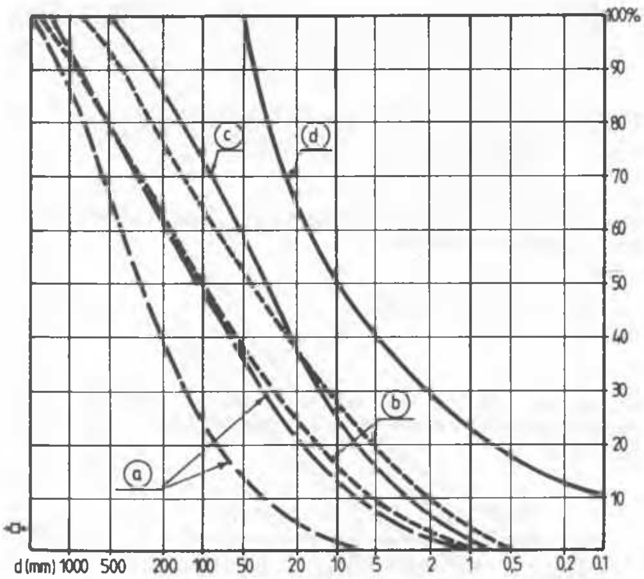

Fig.1. Gradings of rockfill in miccaceous schists : a - in the borrow area ; b - after compaction in the dam ; c - shear tests in large boxes ; d - tests in small boxes and triaxial compression

The great number of tests carried out on the same type of material permitted to render evident the main factors which influence the strength parameters, as well as to compare the results obtained using different types of devices. When these parameters were introduced in the stability calculations of the dam slopes, some differences were found out as to the results obtained by the methods of stability computation utilized up to now.

DEVICES AND COARSE GRAINED MATERIALS USED IN TESTS

Laboratory and Field Equipment

Considering the maximum size d_{max} of the grains used in the dam bodies, the tests were carried out in pilot-stations and in situ, in direct shear devices, whose features are shown in Figure 2 and Table 1 (positions no.1 ... 3).

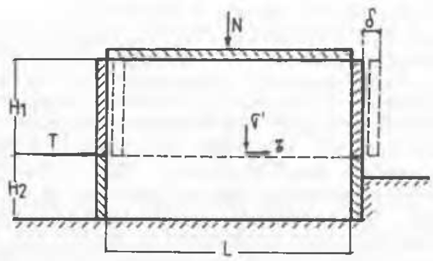

Fig.2. Direct shear box

The tests on "granulometric models" on the rockfill obtained by crushing and on coarse alluvium (sand and gravel) with $d_{max} \leq 50$ mm were also carried out in the laboratory in direct shear devices (no.4 and 5 - Table 1),and in the triaxial compression device on samples with 25 cm diameter and 50 cm height (Fig.3).

Table 1. Data for Direct Shear Tests

No.	L (cm)	B (cm)	H_1 (cm)	H_2 (cm)	σ_{max} (kPa)	d_{max} (mm)
1	80	80	50	30	1,050	200
2	110	110	55	55	1,500	300
3	210	110	55	55	1,500	300
4	40	40	10	10	400	50
5	30.4	30.4	7.5	7.5	500	50

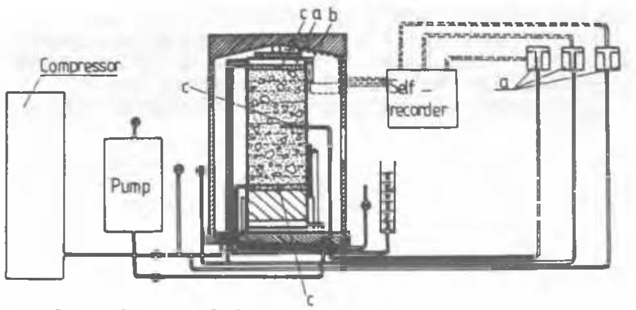

Fig.3. Scheme of triaxial compression device : a - pressure traducer ; b - strain traducer ; c - pore-pressure measuring

The measurements showed that, due to the upper box wall friction, at the direct shear tests, some corrections should be operated on the normal stress σ', of the same order of magnitude like the ones recommended by Bourdeau et al. (1989). The direct shear tests were carried out by increasing the shear force T in steps or at a constant shear velocity v_h = = 1 mm/min. The results obtained by the two methods are close to one another. The "peak" value of shear strength was considered in diagrams (τ , δ).

The triaxial compression tests were made in drained system, by increase in steps of the main stress σ_1'.

Particularities of Tested Materials

The tests in large boxes (B$\geq$ 80 cm) used a grain size distribution curve (Fig.1) located at the lower limit of the range of placed rockfill. In the triaxial compression device and in small boxes (B = 30 ... 40 cm) the "granulometric model" used was characterized by a curve parallel to the real one and having the same uniformity coefficient. The tests on granulometric models led to values of angle of internal friction $\emptyset'$ slightly inferior to those obtained on the material with the real grain size distribution, in the case of direct shear tests.

It should be mentioned that, during the compression and shearing, the rockfill with a low content of fine grains sufer a more pronounced process of crushing.

SYNTHESIS OF EXPERIMENTAL RESULTS

The more than 700 tests carried out in the direct shear devices and about 150 triaxial compression tests showed that for all the coarse-grained materials (crushed rockfill, boulders and gravels with sand) a non linear failure envelope is obtained.

Model of Hyperbolic Variation of Angle $\emptyset'$

The direct shear tests, which define the angle of internal friction $\emptyset'$ on the secant to the failure envelope (Fig.4) lead to a good agreement of the experimental data with the hyperbolic variation proposed by Maksimović(1989) :

$$\emptyset' = \emptyset'_B + \frac{\Delta\emptyset'}{1 - \sigma'/p_N} \qquad (1)$$

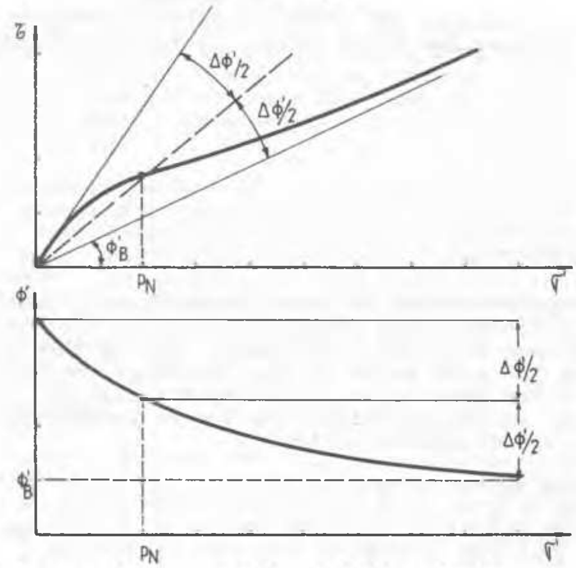

Fig.4. Failure envelope and hyperbolic variation for the angle of internal friction

The triaxial compression tests are interpreted in a similar manner :

$$\emptyset' = \emptyset'_B + \frac{\Delta\emptyset'}{1 + \sigma_3'/p_T} \qquad (2)$$

in which σ_3' is the minimum main stress. The relationship between the parameters p_N and p_T is :

$$p_N = p_T \frac{\cos^2(\emptyset'_B + \Delta\emptyset'/2)}{1 - \sin(\emptyset'_B + \Delta\emptyset'/2)} \qquad (3)$$

The dispersion of the values obtained on sandstone rockfill in the box of side B = 80 cm and on the granulometric model in the box of side B = 30.4 cm, as well as the medium curves given the relation (1) are shown on Figure 5.

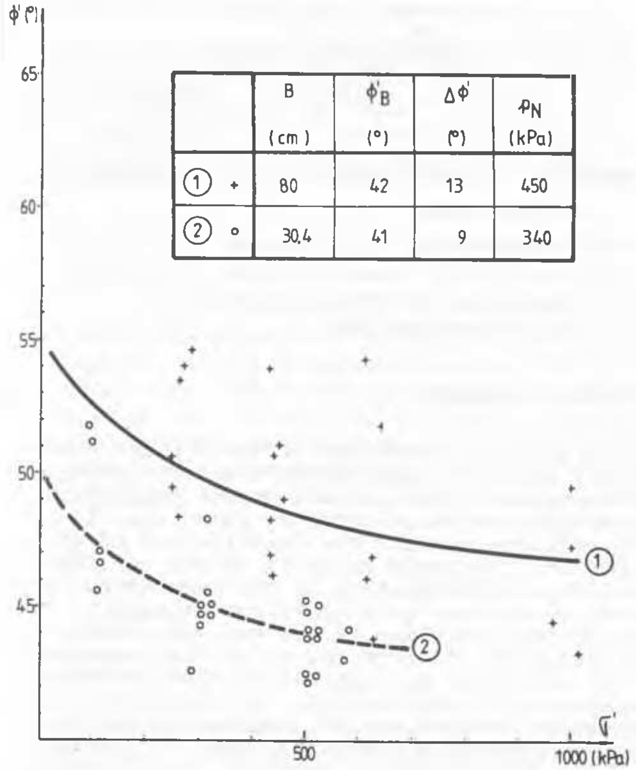

Fig.5. Direct shear tests on sandstone rockfill (average density ρ_d = 1900 kg/m^3)

Figure 6 contains the medium curves obtained experimentally by triaxial compression on minimum 8 samples for each material and the hyperboles defined by the relation (2).

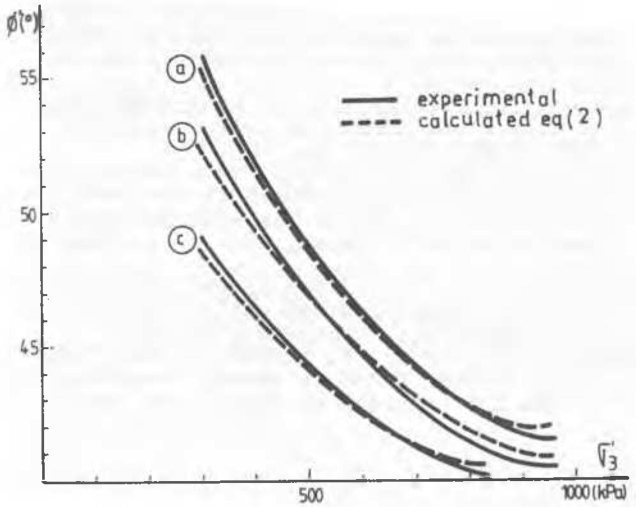

Fig.6. Triaxial compression tests on rockfill: a-micaceous schist ; b-sandstones ; c-limestone

The processing of the direct shear and triaxial compression tests results allowed to determine the medium values of the parameters of equation (1) for certain categories of coarse-grained materials (Table 2). The values ϕ'_B are specific for each type of material and do not depend practically on the relative density. The parameters $\Delta\phi'$ and p_N which introduce the contribution of dilatancy, combined with that of grain crushing, depend on the rock nature, grain size distribution and the relative density. Since the modification of grain dimensions and shape during shearing is higher in the case of rockfill, the variation of angle ϕ' with the normal stress σ' is higher than the one determined in the case of sands (Bolton,1986).

Table 2. Mean Values of Equation (1) Parameters

Granular material	ϕ'_B (°)	$\Delta\phi'$ (°)	p_N (kPa)
Gravel with sand, loose	36	4	250
Gravel with sand, compacted	37	7	350
Boulders with gravel and sand, compacted	37	19	400
Chrystalline schist rockfill, loose	32	10	200
Chrystalline schist rockfill, compacted	33	25	250
Micaceous schist rockfill, compacted	32	35	650
Limestone rockfill, compacted	36	27	300
Sandstone rockfill, compacted	42	13	450

Model of Power Type Relationship for Angle ϕ'

The triaxial compression tests on granulometric models (with maximum grain size d_{max} = 50 mm) led to values of the secant angle ϕ' close to the ones determined by direct shearing of the materials with d_{max} = 200 ... 300 mm (Fig. 6). Nevertheless, a better grouping of values is noticed in the stress range σ'_3 = 100 ...1,000 kPa, if an exponential relationship is adopted in the forme presented below :

$$tg\ \phi' = \frac{1}{R(\sigma'_3/p_o)^r} \qquad (4)$$

in which p_o = 100 kPa.

The parameters R and r are easily deduced by logarithmaring the equation (4) which becomes linear (Fig.7).

Table 3 gives the values R and r determined for different rockfill types as well as those of the parameters A and a of the failure envelopes which result in the form :

$$\tau = A.p_o\ (\sigma'/p_o)^a \qquad (5)$$

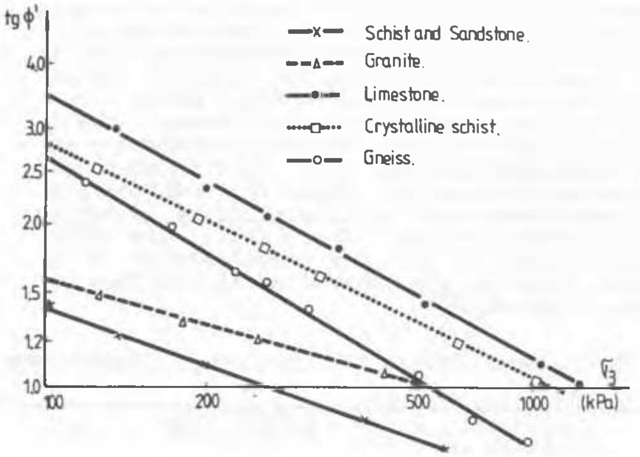

Fig.7. Determination of the parameters of equation (4) from triaxial compression tests

It can be seen that the values A and a are close to the ones determined by other researchers (Marsal,1976).

Table 3. Values of the Parameters of Equations (4) and (5)

Rockfill type	R	r	A	a
Schists+sandstones	0.731	0.33	1.364	0.76
Granites	0.678	0.22	1.475	0.78
Granitic gneiss	0.542	0.37	1.845	0.63
Chrystalline schists	0.512	0.32	1.953	0.67
Limestones	0.458	0.34	2.180	0.65

INFLUENCE OF VARIABLE ANGLE OF FRICTION ON DAM SLOPE STABILITY

The model given by equation (1) can be also used in the case of "classical" computing methods along continuous slide surfaces.

In the case of rockfill dams with upstream watertightening screen, the classical methods for the stability coefficient calculation have a limited application, since they do not consider the modification of the stresses caused by the reservoir. For this dam type,more realistic results were obtained by the use of model (4) for ϕ' dependence on the stress σ' and by the calculating the stresses in the dam body by a more accurate method. Figure 8 shows the isolines of the failure ratio $R_f = (\sigma'_1 - \sigma'_3)/(\sigma'_1 - \sigma'_3)f$, in the body of a gneiss and schist homogeneous rockfill dam, before and after the reservoir impounding. The stresses were determined by the finite element method, considering a non-linear stress - strain relationship determined by triaxial compression (Duncan et al.,1970). The computation program (Luca et al.,1987) uses, for the angle ϕ', the relation given by equation (4) and the parameters shown in Table 3.

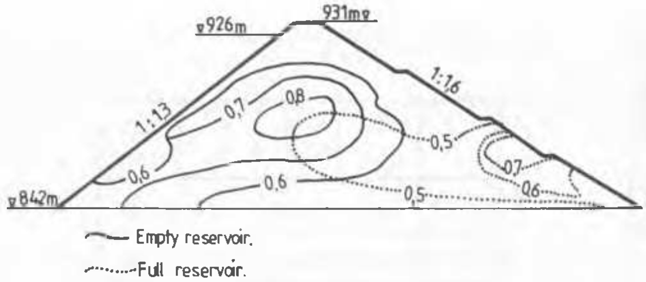

Fig.8. Variation of failure ratio R_f in Sebes-Oaşa dam

CONCLUDING REMARKS

The non-linear character of the failure curve in the case of coarse-grained materials leads to variable values of the secant angle ϕ'.The hyperbolic relation given by equations (1) or (2) reflects clearly the contribution of dilatancy and the grain crushing effect but, for a practical determination of the parameters of these equations, more tests are necessary. The exponential relation (4) covers satisfactorily the range of moderate values of the pressure σ'_3 and allows an easy calculation of the parameters by linearization. The stability calculations carried out by considering the non-linear dependence between stress and strain and the variability of angle ϕ' lead to a more accurate delimiting of the shear strength maximum mobilization zones and to the more correct determination of the stable slopes.

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