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STRENGTH AND DEFORMABILITY OF SANDS UNDER VARIOUS CONDITIONS
RESISTANCE ET DEFORMABILITE DES SABLES SOUS DIVERSES CONDITIONS
ПРОЧНОСТЬ И ДЕФОРМИРУЕМОСТЬ ПЕСКОВ В РАЗЛИЧНЫХ УСЛОВИЯХ

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SYNOPSIS, Two extensive series of triaxial tests were carried out, in order to study the influence of several parameters on the angle of friction and the critical strains of various sands. The effects of relative density and grain size, as well as that of lateral pressure and the size of the specimen have been investigated. Thus, critical strains seem to increase with the lateral pressures applied and the grain size, decreasing with larger specimens. Under some conditions, the angle of internal friction seems to decrease when determined by means of larger specimens. Some theoretical interpretations of the experimental findings are proposed.

1.- INTRODUCTION

This research was initiated in the purpose to examine some of the problems of reliability, related with stability analysis of structures affected by passive earth pressure. Design of a buried anchor-block, e.g., necessitates the following information

- . How the angle of internal friction of the surrounding sand is affected by the volume of the loaded earth mass.
- . What percentage of the total passive pressure can be mobilized at the limit state of maximum allowable displacement of the block.
- . How this mobilized pressure is influenced by the transversal earth pressures at rest, the rate of loading and other parameters.

This paper is intended to discuss some of these points, on the basis of an extensive experimental work in the laboratory and by means of some theoretical mechanisms as well. Some secondary findings are occasionally reported.

2.-EXPERIMENTAL WORK

- . The whole investigation comprised two series of triaxial tests carried out on several sands K_1, K_2, K_3 (Fig.1) (*), of constant uniformity coefficient $d_{60} : d_{10} = 1,5$.

(*) The 1st series comprised somewhat different

- Sands - $K_1 \cong$ silicious sand $0,15 \div 0,40$ mm
- $K_2 \cong$ Calcareous sand $0,40 \div 2,00$ mm
- $K_3 \cong$ calcareous sand $2,00 \div 4,50$ mm

tested in specimens of various heights ($H_1 = 7,5$ cm, $H_2 = 15,0$ cm, $H_3 = 30,0$ cm), their diameter being half as large as their height.

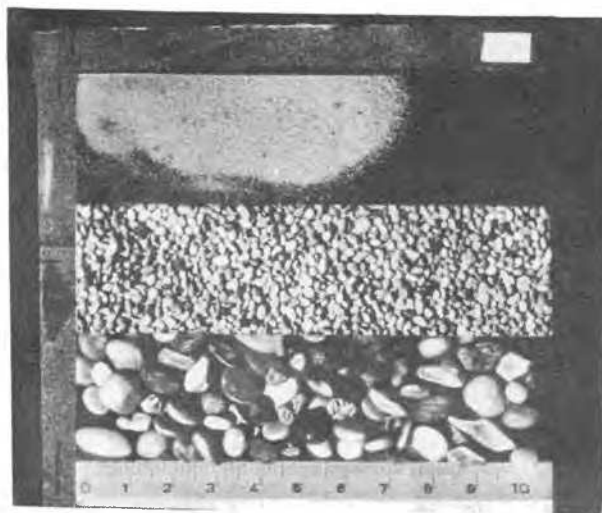


Fig.1.- Grains of the sands tested in 2nd series.

- $K_1 \cong$ silicious sand $0,15 \div 0,40$ mm
- $K_2 \cong$ calcareous sand $0,85 \div 2,00$ mm
- $K_3 \cong$ calcareous sand $6,00 \div 10,00$ mm

For each sand and each height of specimen, 3 relative densities have been examined ($D_{r1} = 0,25$, $D_{r2} = 0,50$, $D_{r3} = 0,95$). Consequently, the following experimental scheme has been investigated in each series

$K_i D_j H_l$, all combinations $i, j, l = 1$ to 3
Consequently, some 160 triaxial tests have been performed, for lateral stress values equal to 0,5, 1,5 and 2,5 kg/cm².

Additional tests were carried out, concerning higher values of lateral pressure (5,0 kg/cm²) and rates of loading others than the usual value of $4 \cdot 10^{-4} \text{ min}^{-1}$, i.e. $20 \cdot 10^{-4}$ and $0,6 \cdot 10^{-4} \text{ min}^{-1}$.

Some of the materials examined during the first series of tests, have been reused several times. Consequently, the main conclusions of this paper are based on results of the second series, in which no reuse of materials has taken place.

3.- ROUTINE RESULTS

Isostrain curves for various $\sigma_1/\sigma_2 = \sigma_3$ combinations have been drawn, useful for stress path calculations of settlements. A typical example is given in Fig.2.

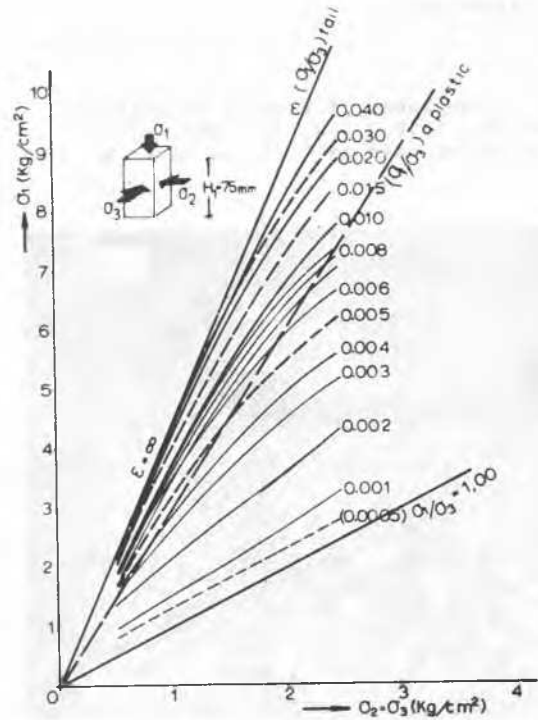


Fig.2.- Typical isostrain curves, under triaxial loading (silicious sand $d_m = 1,4 \text{ mm}$, $D_r = 95 \%$).

The well known relations between the angle of friction and the relative density of sand has been reconfirmed (see e.g. Kirkpartick, 1965). An example is given in Fig. 3.

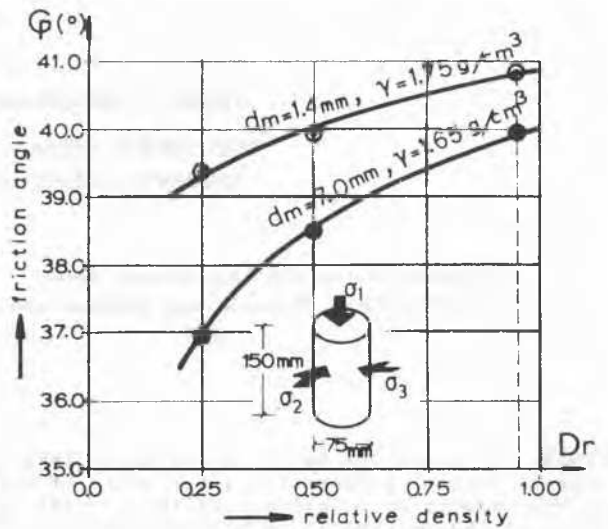


Fig. 3.- Frictional resistance of calcareous sand as a function of its relative density (second series of tests).

4.- STRESS INSTABILITY

When measured very frequently during the test, the major stress σ_1 shows a characteristic "instability" in cases of sands with large maximum grain d_{max} , tested in small diameter (d_s) specimens. Fig.4 seems to suggest that for $d_s : d_{max}$ values greater than 20, the phenomenon of this instability becomes negligible.

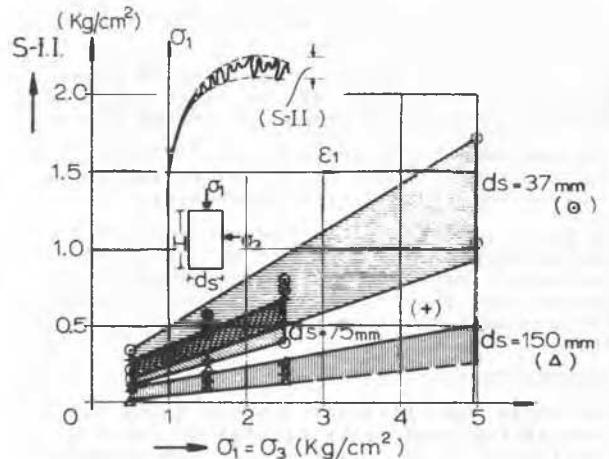


Fig.4.- "Stress-instability interval" (S.-I.I.) at peak-point, for calcareous sand $d_{max} = 10 \text{ mm}$, as a function of the size of the specimen.

Lateral stresses tend to increase the above variations of σ_1 - values during strain controlled triaxial tests.

If grain interlocking is supposed to be responsible for these phenomena, Fig. 5 shows a rather oversimplified model for a limit case of direct influence of grain rotations on the bearing plates. This case corresponds to a somewhat similar ($d_s : d_{max}$) value. As a matter of fact,

$$H = 2d_s \approx 1,72d_s + 6,91d_{max} \rightarrow \frac{d_s}{d_{max}} \approx 25$$

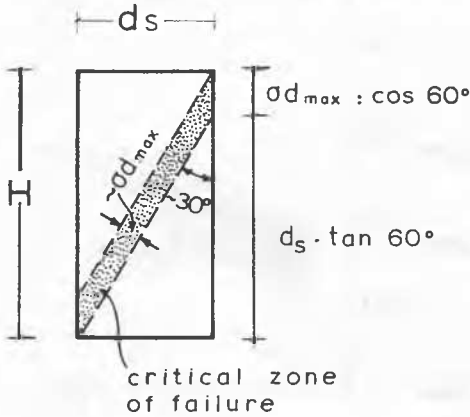


Fig. 5.- Limit case of critical zone of failure, affecting directly the platens of the loading machine (oversimplification).

5.- MOBILIZED EARTH PRESSURE

Fig. 6 summarizes the widely scattered relation between axial strain ϵ_1 and the respective percentage of mobilized pressure σ_1 reported to the critical value $\sigma_{1,cr}$ in failure.

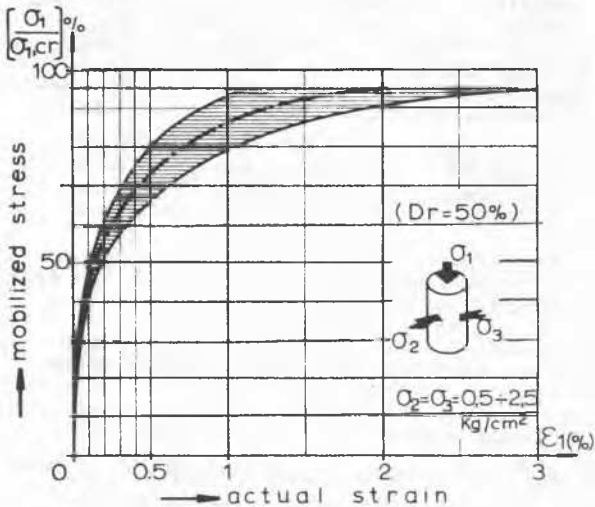


Fig. 6.- Mobilized "earth pressure" as a function of the axial strain applied, for several sands ($d_m = 0,2$ to $7,0$ mm), tested in specimens 75×150 mm and 150×300 mm.

The results concern a large variety of sands tested under various conditions of lateral confinement and volumes of samples investigated, for an average relative density equal 50%. No significant distinction of particular curves has been possible. Nevertheless, the discussion of the next chapter allow for some kind of distinction of this character.

Previous investigations had proved that one seventh of the critical strain is sufficient to mobilize 50% of the maximum passive pressure.

The results of Fig. 6. seem to reconfirm this rule, leading to a somewhat higher mean value of mobilized pressure ($\approx 60\%$).

A displacement equal to 2.10^{-3} of the length of critical volume, behind an anchor block is necessary in order to mobilize half the peak-passive pressure. For a depth of such a block equal to 2,50 m. and a material having $\phi = 38^\circ$, the corresponding absolute displacement should be equal to 1 cm., approximately. A factor of safety equal to 1.5 is suggested for such a displacement-controlled design, due to the scattering encountered.

6.- CRITICAL STRAINS

Coming back to the highly scattered values of strains corresponding to a mobilization of 95% of the strength, Fig. 7 is an attempt to separate the parameters affecting the above critical strain.

Apparently, large grain sizes and large lateral stresses result to an increase of that critical strain.

Fig. 8 represents a special case of silicious sand, with similar results.

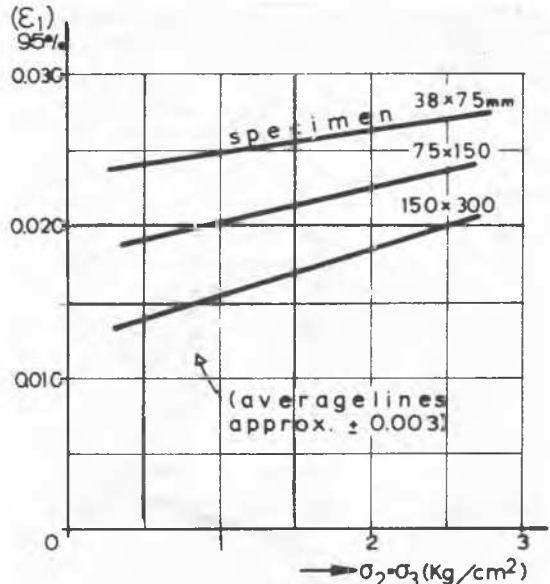


Fig. 8.- Critical strain (ϵ_1) 95% at σ_1 stress-level equal 95% of the strength. (silicious sand, $d_m = 0,25$ mm, with several relative densities).

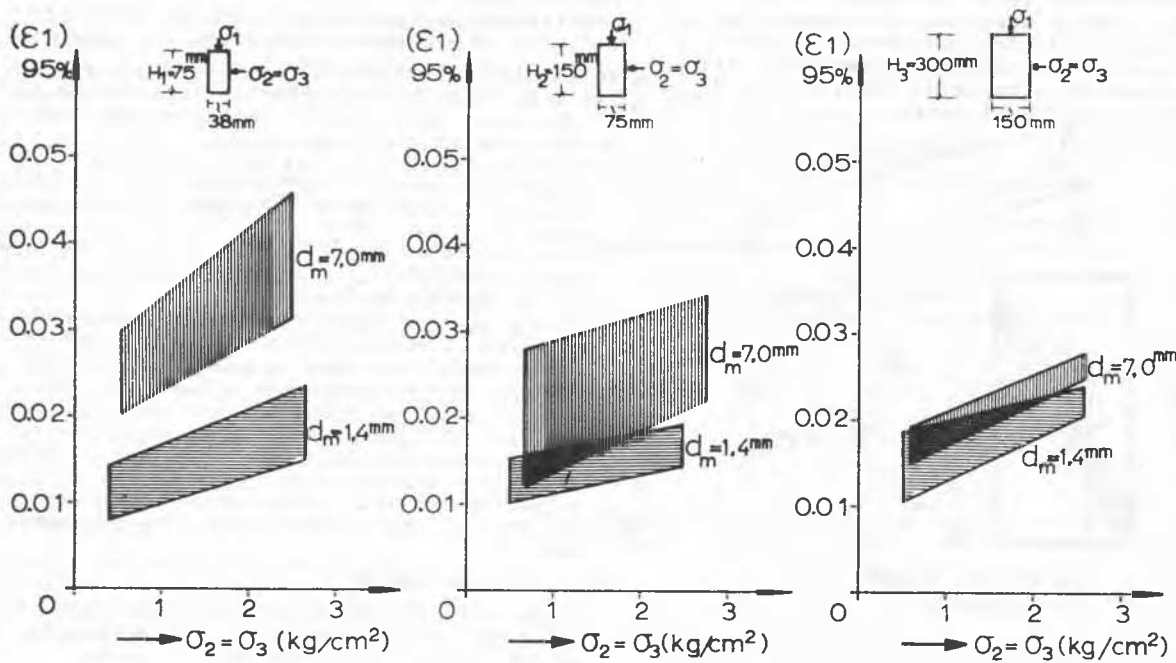


Fig.7 Critical strain $(\epsilon_1)_{95\%}$ at σ_1 -stress - level equal 95% of the strength. (Calcareous sand for rel.densities ranging from 0,25 to 0,95).

It is worth to note the significant reduction in critical strains in specimens of larger sizes. Nevertheless, it seems that this reduction will not be effective in specimens larger than 20×40 cm.

A less distinct relation seems to exist between $(\epsilon_1)_{50\%}$ and lateral pressure, according Fig. 9.

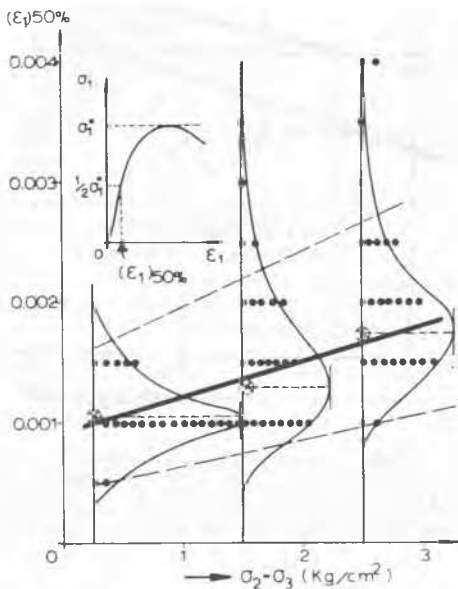


Fig.9.- Critical strain $(\epsilon_1)_{50\%}$ at σ_1 stress - level equal 50% of the strength, for various sands, rel.densities and sizes of samples.

Among earlier experimental evidence confirming the findings of this chapter, the authors would like to refer themselves to the fundamental work carried out on artificial sphere packings in Karlsruhe (Institute fur Bodenmechanik und Felsmechanik, Prof.H. Leussink).

In fact, according to Waseloh, 1967, biaxial compression of steel spheres resulted to a very clear reduction of ultimate strain (un to 50%), when the height of the packing was doubled.

A less spectacular change of ultimate strain has been reported by Brauns, 1967, (in the case of triaxial loading of quartz spheres), as a function of the lateral pressure σ_3 . On the basis of the findings of Brauns, the following empirical expression could be formulated:

$$\epsilon_{u,1}^{(\sigma_3)} \approx \epsilon_{u,1}^{(1)} [0,2(\sigma_3 - 1) + 1], \sigma_3 < 20 \text{ kg/cm}^2$$

where $\epsilon_{u,1}^{(\sigma_3)}$ designates the ultimate axial strain ϵ_1 for the peak value of σ_1 , under lateral stresses $\sigma_2 = \sigma_3 < \sigma_1$

The experimental results of the present investigation seem to reconfirm quantitatively the above formula in the case of the larger samples examined (150×300 mm).

A similar result of a more pronounced plasticity, in case of increased lateral pressures, is noted as well by Marsal (1965,a) and by Schultze (1965).

- A tentative interpretation of the relation between critical strain and the volume of the sample, could be based on the theory of stochastic processes of mechanical phenomena at intergranular contacts. It has been shown (Marsal, 1965,b) that larger thicknesses of soil layers manifest smaller strains, within a constant period of loading, as compared with strains of layers of small thicknesses, provided that the long term strain is a constant characteristic.
- Finally, for the well known relations between critical strain and confining pressure, it might be speculated that lateral compression minimizes the effect of local leakages, otherwise they would contribute to a more rapid generalization of yielding (brittle behaviour).
- It is worth to note that after several times of repetitive loadings of a sand to failure and reuse of it, its ultimate strain becomes larger, in as much as the confining pressure is higher (Fig.10).

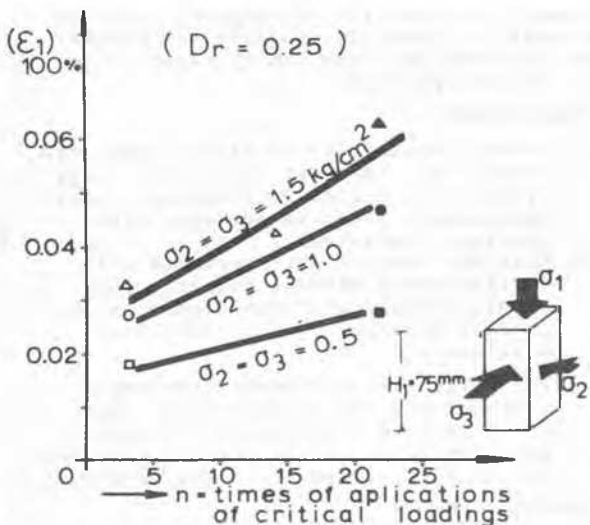


Fig.10.- Ultimate strain (ϵ_1) at peak-point after repetitive critical loadings and reuse of a calcareous sand, $d_{max} = 3,4 \text{ mm}$.

In this case, softening of the interfaces results in greater steps performed by the grains, contributing to higher values of coefficient of diffusion of the solid phase (see Marsal 1965,b/page 305). The stochastic theory already mentioned foresees an explicit increase of

(*A somewhat similar interpretation could be ventured for the role of larger grain sizes in the increased critical strains observed. By virtue of similitude, a given layer of large grains, corresponds to a thinner layer of fine grains, which in turn manifests greater strains.

the actual strain when the coefficient of diffusion is higher.

7.- VARIATION OF FRICTIONAL RESISTANCE AS A FUNCTION OF THE SPECIMEN SIZE.

Wide scattering of ϕ -values in both series of this investigation is attributed to the usual causes, plus the poorly controlled reuse of materials in the first series. Additional scattering is particularly related to the smallest specimen's size; its results should be considered as less consistent. However, for a ratio $d_s : d_{max}$ (of specimen's diameter d_s to the maximum grain size d_{max}) higher than 10, Fig.11 offers a statistical evidence of decrease of the angle of internal friction as the volume of the specimen increases.

A remarkable exception of this rule is apparent in the results of the 2nd series concerning the smallest specimen size, where $d_s : d_{max} = 37,5 : 10,0 \approx 4$. It is interesting to note that this exception did not appeared in the 1st series, where, for the smallest specimen and the larger grain size, it was $d_s : d_{max} = 37,5 : 4,8 \approx 8$.

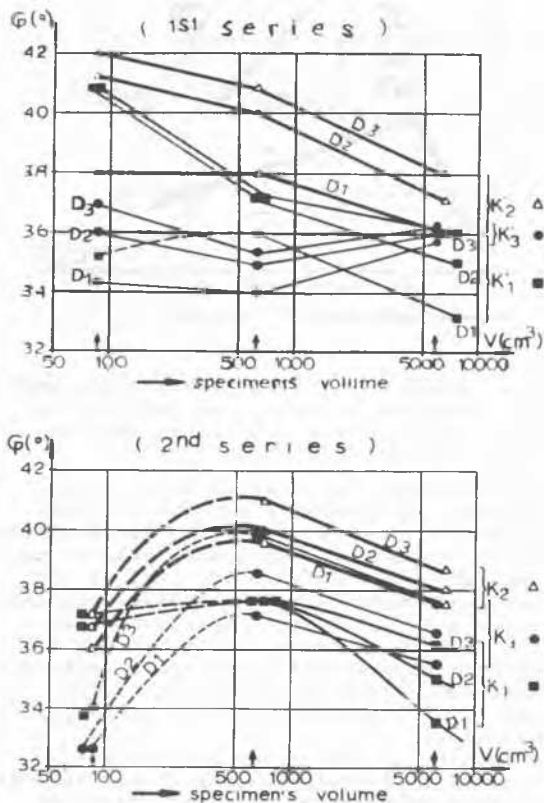


Fig. 11.- Angle of internal friction versus the volume of the specimen, investigated in triaxial loading ($\sigma_3 = 0,5$ to $2,5 \text{ kg/cm}^2$). For K_i and D_i notations, see para.2.

Among earlier investigations on this subject we are referring to Schultze (1965) and Marsal (1965, a), their results on several granular materials being retaken on Fig. 12.

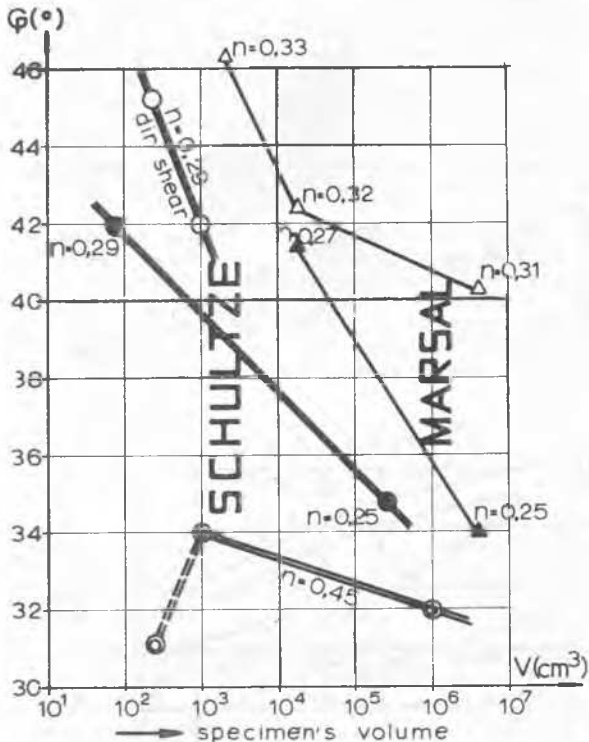


Fig. 12.- Angle of friction versus specimen's volume, according to earlier investigations (Schultze, 1965 and Marsal, 1965, a).

Small differences in porosity reported on this figure, are probably not fully responsible for the observed variations in ϕ -values. On the other hand, a discrepancy related to the smaller sample with $n=0,45$ reminds the similar discrepancies of the present investigation (Fig.11), for small specimen sizes. To this context, Fukuoka (1965) attributes the smaller strength of small-scale tests to the magnitude of the maximum size of the soil grain, as compared with the dimension of the sample.

The authors of this paper are aware of the arguments of Prof. Leussink (1965) against the postulated decrease of shear strength as measured in larger specimens. However, they wish to observe that the tests mentioned by Dr. Leussink were not carried out on the same material. In fact, for the sake of common $d : d_{max}$ ratio, larger specimens contained larger grains ($d_{max} = 10\text{cm}$ for $d = 100\text{cm}$, instead of $d_{max} = 0,5\text{ cm}$. for $d = 5\text{cm}$). To this regard, it could be argued that natural or artificial fragmentation process of one and

the same material, leaves larger grains with relatively sharper edges and higher modulus of deformability, as compared with smaller grains. Therefore, in this case uniform materials containing larger grains could have higher angle of internal friction. This is probably a counteracting cause to the decrease of ϕ -values expected in larger samples.

As far as the present investigation is concerned, a quite high value of $d : d_{max}$ ratio was secured ($d : d_{max} = 10$ to 350), in the hope that this parameter will not affect the strength of the samples tested. Furtheron, comparisons are made between results taken on exactly the same granular material. Unfortunately, wide scattering and relatively poor control of these tests, do not allow for final conclusions on the subject exposed.

Nevertheless, as it is the case for other materials too, several theoretical approaches are offered to interpret such an inverse relation between strength and volume of a sample; Statistical theory of distribution of defective points, mechanism of end restrains in small specimens etc, or a combination of them.

Further, very carefully designed, research is needed in order to separate the parameters involved and make a more clear statement on this problem.

8.- CONCLUSIONS

- Stress instability in strain controlled tests, near the peak-point (for large grain sizes tested in relatively small specimens), tends to decrease with specimen dimensions.
- Critical strains are increased with smaller loaded masses, larger grain sizes, higher confining pressures and gradual breakage due to repetitive loadings.
- For large ratios between specimen's diameter and the maximum grain size, there is some experimental evidence for a decrease in strength of granular soils, when tested in larger specimens.

9.- ACKNOWLEDGEMENTS

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