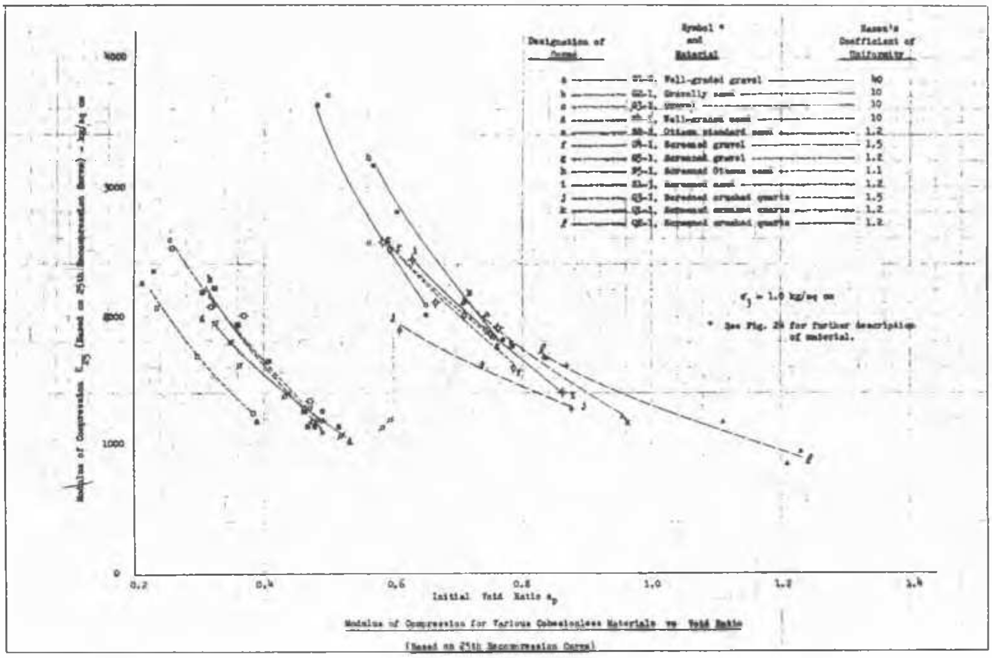

FIG. 26

CONCLUSIONS

The investigations presented above lead to the following conclusions:

1) The stress-strain and strength characteristics of cohesionless soils can be accurately determined by means of a simple vacuum type triaxial compression apparatus for the dry state and for confining pressures less than one atmosphere.

2) By means of improved equipment for placing and compacting cohesionless soils, great uniformity in the density of test specimens was achieved. It was also possible to duplicate accurately a given density and test results.

3) For the tests performed, the major portion of the stress-strain curves can be approximated by straight lines on logarithmic plots.

4) The angle of internal friction of cohesionless soils increases with increasing angularity of the grain and with increasing Hazen's coefficient of uniformity.

5) The compressibility of cohesionless soils increases with the angularity of grain, but is not appreciably affected by Hazen's coefficient of uniformity.

6) The lateral strain of test specimens of cohesionless soils increases at a faster rate than the axial strain. A cohesionless soil subjected to a constant lateral pressure does not possess a constant Poisson's ratio.

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THE CAUSES OF THE JERKY PROCESS OF DEFORMATION IN THE CASE OF SATURATED AND DAMP SANDS

II d 12

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By subjecting sand-cylinders to pressure tests we find that in the case of dry sand, the process of deformation is steady, and parallel tests carried out by means of precisely working apparatus show practically no dispersion. On the other hand, tests with damp sands as well as with sands saturated with water, even if carried out with the greatest care, have shown that only the initial and final stages of the tests, i.e. the first stages of deformation and break load are free from dispersion. As soon as the process of deformation ceases to take a linear course and begins to develop progressively, deformation takes place by jerks and is accompanied by considerable dispersion which cannot be avoided even though tests be carried out with the greatest possible care and precision. The cause of this phenomenon is as follows: As long as the shearing forces from grain to grain are nowhere strong enough to surpass friction, deformation takes place practically without changing the density of the bedding and in linear manner. However, with an increased main tension ratio, a progressive displacement of grains takes place until the entire mass of sand is set in motion at break point. Displacement is accompanied at the beginning by a slight compression, which in its final stages develops into a loosening of the structure as shown by Figure No 1.

It will be seen that while the density of the bedding remains unchanged, two main tension

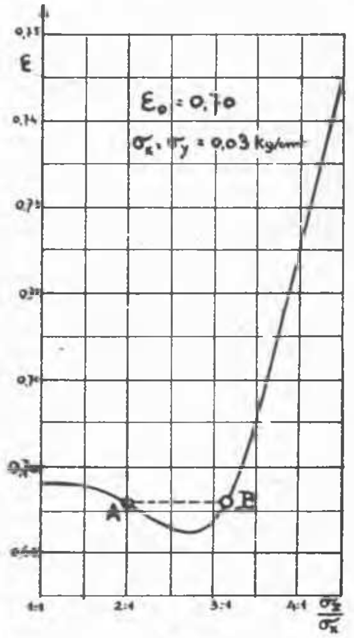

FIG. 1

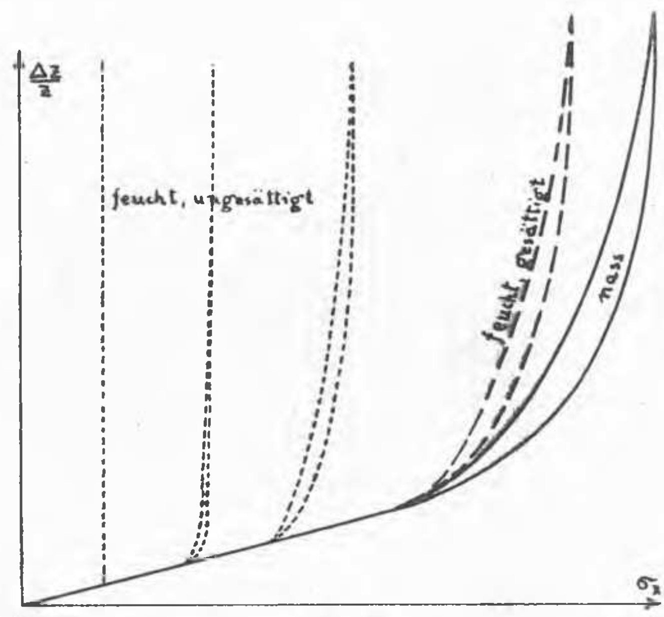

FIG. 3

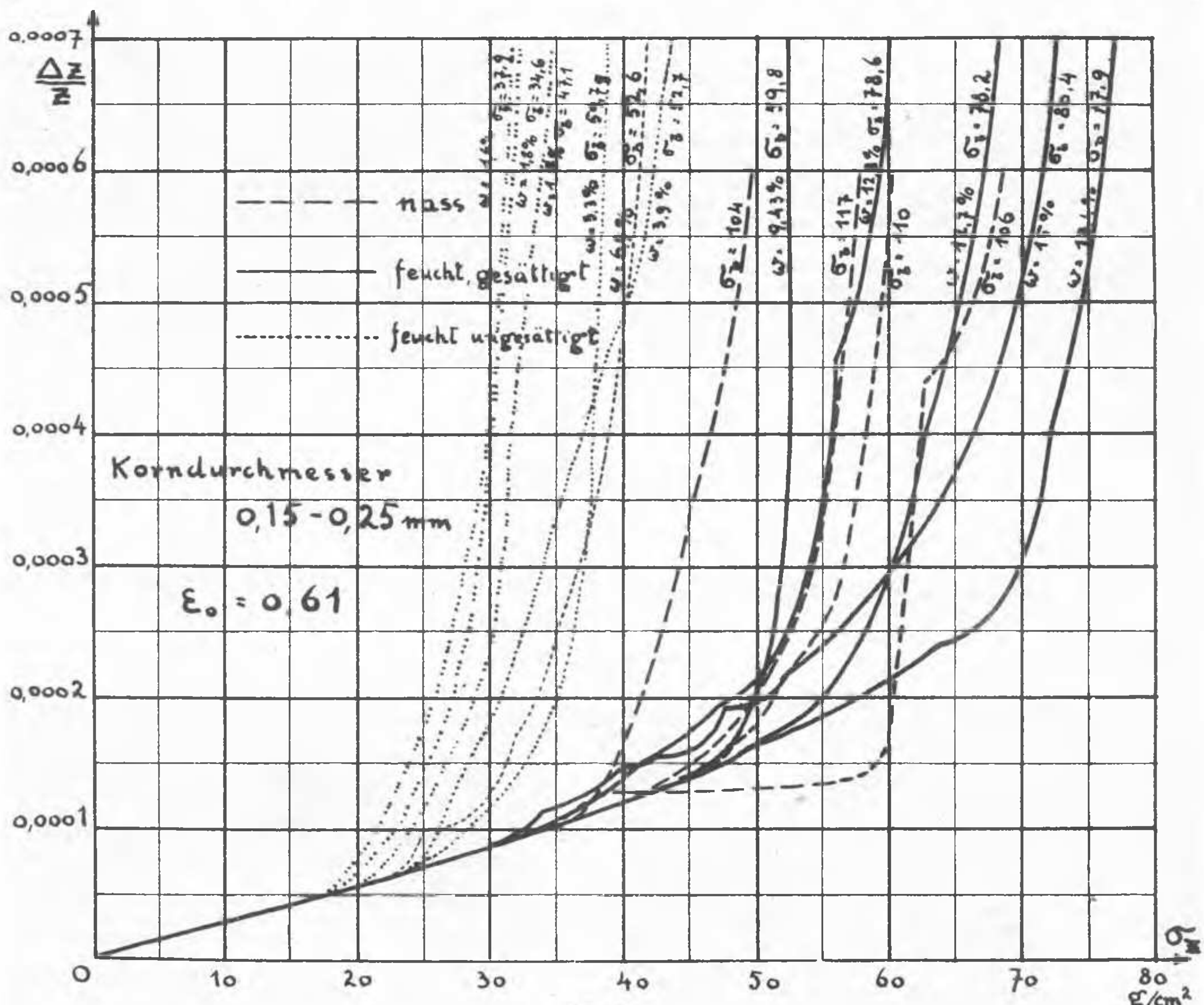

FIG. 2

ratios are possible corresponding to the points A and B in Fig.No.1, and in accordance with the displacement of the individual grains, one or the other process takes place within separate zones in the interior of the sand, so that tension becomes inhomogeneous, which is the cause of the progressive nature of the deformations. This change-over of the grains from one tension ratio to another takes place without retardation in the case of dry sand, as no resistance is offered by air. If, however, the pores contain water, the process is retarded, which leads to a break-down of the granular structure within a considerable sphere thus causing the jerky process of deformation.
Figure No. 2 allows a comparison of the results produced by prism-tests carried out by the author with water-saturated and damp sands of various humidity (containing different percentages of water) and a granulation of from 0.15 - 0.25 mm grain-diameter, and an initial void ratio of ε_o = 0,61. The interrupted lines represent sand saturated by water, fully drawn-out lines represent humid sand containing a maximum quantity of water, whilst dotted lines show sand with low percentage of water. Break tensions σ_b are given for each curve, Figure No. 3 gives a diagrammatic description of the process.
As follows from what has been said above, deformation by jerks in the case of tests carried out with sand cylinders is caused by pore water and it has proved impossible in this case in spite of the most carefully selected test-apparatus to avoid considerable dispersion within the sphere in which deformation increases in a progressive manner, - which is in direct opposition to results obtained by tests carried out with dry sand.

REFERENCE

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II d 13 SHEARING STRENGTH DETERMINATIONS BY UNDRAINED CYLINDRICAL COMPRESSION TESTS WITH PORE PRESSURE MEASUREMENTS

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GENERAL

Data on magnitudes of shearing strengths are needed in the solutions of many types of practical problems relative to the stability of earth embankments and foundations. When the soil in question is a thick deposit of highly impervious, completely saturated clay, there often is opportunity for practically no drainage during shear, and laboratory tests for the determination of shearing strengths for such cases must be tests in which no drainage is allowed. If a limited amount of drainage occurs, the best approach to the problem usually is by the determination of shearing strengths for both the completely undrained and the completely drained cases, followed by an interpolation that must be based largely on judgment. Tests with complete drainage often require much time, but they offer no particular difficulties of interpretation. Tests in which no drainage occurs are more difficult of interpretation and are the subject of this paper.

Fundamentally, the shearing strength in all instances depends on the intergranular pressures that prevail when failure is taking place. However, in tests in which no drainage occurs, there is a gradual build up of pore-water stress that can only be measured by special testing apparatus; thus the pore-water stress and the intergranular pressures at failure are of unknown magnitude when ordinary testing methods are used. However, the intergranular pressures that are attained at failure in tests that do not drain bear definite relationships to the intergranular pressures prevailing at the start of the tests, and in some cases these initial intergranular pressures are known. Therefore, the logical relationship for use in the case without drainage is one in which the shearing strength is expressed as a function of the pressure to which the sample was consolidated previous to shear.

If the initial consolidation pressures on a given soil in nature are the same on all orientations of planes within the sample, relationships between shearing strength and consolidation pressure may be predicted from the data of a standard type of shear test. x) In general, however, the original intergranular pressures in nature have a principal stress ratio that is of unknown magnitude, but that is known to be greater than unity, and the general form of the relationships between shearing strength and consolidation pressures, applying for any given principal stress ratio, can be obtained only by the use of special tests in which pore pressures are measured.

This paper presents examples of tests on saturated, or nearly saturated clays, in which pore pressures are measured. The main object of the paper is to present methods for the interpretation of these test data. The methods presented apply in fullest degree to clays that have not been precompressed; that is, to clays which have never been consolidated to pressures greater than those existing at the start of the shear test. Therefore, the method as it applies to clays that have not been precompressed will be presented first, and the precompressed case will be considered later.

Before proceeding with the main subject, special and simplified conditions of testing, that impose limitations on the applicability of the results, will be mentioned. The tests that are reported in this paper are cylindrical compression tests (often called triaxial tests) of more-or-less standard form, with equal minor and intermediate principal stresses acting throughout. They should, without exception, be run on specimens that are in good

x) These tests were proposed by A. Casagrande and are known as "quick-consolidated" or "consolidated-undrained tests" 1), 2).