sure the pore water pressures in function of time.

Moreover , remounting to causes of the phenomenon now called secondary time effect, it seems that material's nature should be of peculiar importance, and precisely its brittleness and the fact that possibly, on account of breaking of single grains, the volume of solid constituents of the soil may be no more considered as constant.

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CONSOLIDATION TESTS ON SOILS CONTAINING STONES

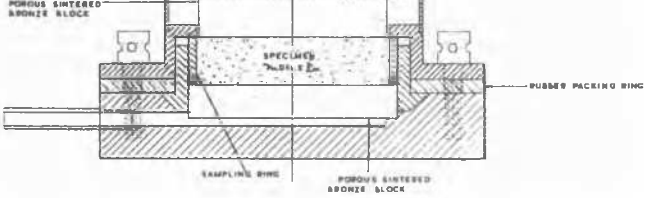
L.J. MURDOCK, Ph.D., M.Sc., A.M.I.C.E.

The test normally adopted in the determination of the consolidation characteristics of a soil, utilises a specimen of 3 inch diameter and approximately $\frac{3}{4}$ inch thick. This thin specimen is used and drainage allowed both from the top and the bottom, in order to shorten the time required for the test.

Soils containing stones, e.g., boulder clays, are common to many parts of the world and estimates of settlement under load can only be made from consolidation tests on thin specimens (1) by selection of a portion of the main sample free from stones, (2) by patching, if only an odd stone needs removal, or (3) by removal of the stones and remoulding. None of these methods can be considered satisfactory, especially when the high cost entailed in taking the sample, is considered. Investigations were, therefore, made to determine the effect of the presence of stones and the possibility of using larger test specimens, or other methods of testing.

Consolidation Apparatus.

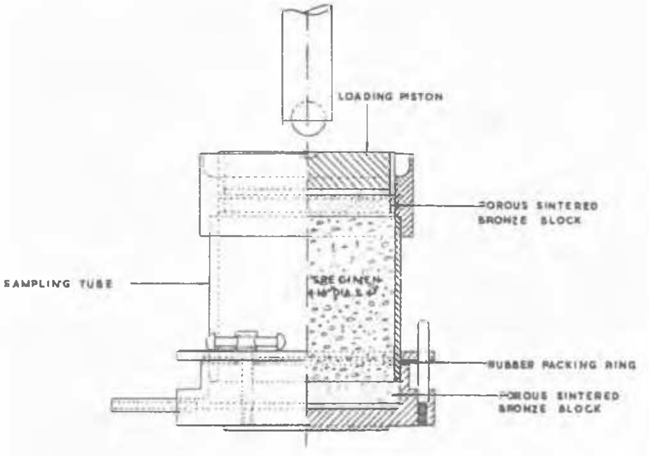
Specimens of 3 in., 4 in., and 6 in. diameter were tested by the conventional method, using a cylindrical mould with porous stones at each end, and the load applied vertically through a piston. The apparatus used for the 3 in. diameter specimens was not materially different from that used by other investigators, and is illustrated in Fig. 1. The method adopted to obtain the sample was to press the sampling ring into the soil being extruded from a 4 in. diameter sampling tube normally used for obtaining undisturbed samples in the field, and then to trim off the two ends. The finished thickness of the sample was $\frac{3}{4}$ inch.



Consolidation box 3in. Dia.

FIG.1

The apparatus for the 4 in. diameter specimen is illustrated in Fig. 2. In this case the sample was pushed direct into the 4 in. nominal bore sampling tube from another longer tube of the same diameter. The ends were then trimmed to give a finished thickness of approximately 4 in., and the load applied through porous blocks in the usual way.



Consolidation box 4in. Dia.

FIG. 2

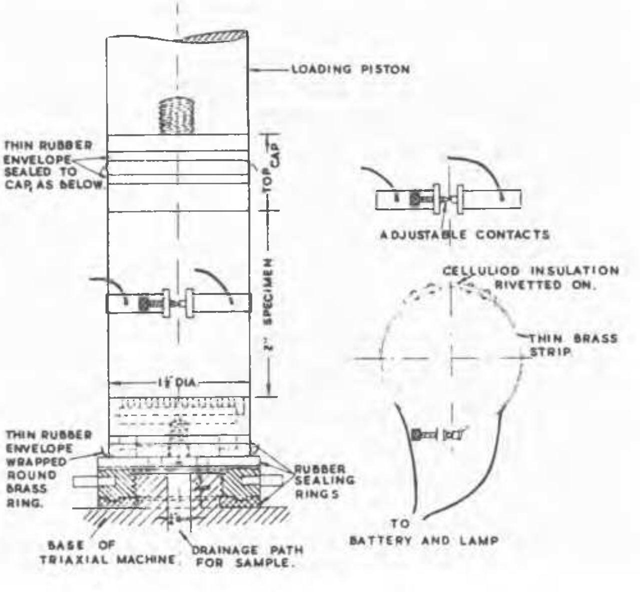
In view of the difficulties described later in connection with the use of the 4 in. apparatus, only one 6 in. diameter specimen was tried. The sample was contained in a mould normally used for C.B.R. tests. A piston was used for the vertical load in the usual way, and porous stones made from brick dust and cement provided drainage.

The effects of friction in the 4 in. diameter apparatus were found to be serious and the following methods of overcoming this were considered:

1) coating the inside of the tube with glue or other similar material to reduce friction.
2) pouring molten paraffin wax round the soil specimen placed inside a sampling tube of larger diameter, the object being to provide an incompressible container, but with comparatively low side friction. This method was not effective.
3) the use of the triaxial compression testing machine utilising the water pressure to maintain a specimen of constant diameter. The object of this arrangement was to eliminate friction entirely, while still obtaining one dimensional compression.
4) the use of the triaxial compression testing machine with fixed end pistons, and an ar-

rangement to measure the change in diameter of the sample when compressed by means of the water pressure.

Of these four methods, 3) was considered the most likely, and the arrangement shown in Fig. 3 was adopted. The soil specimen 2½ in. high x 1½ in. diameter, was encased in a rubber envelope sealed to the end caps shown, the lower one of which was porous to allow drainage under atmospheric pressure. The specimen was then placed inside the perspex cylinder of the triaxial compression testing machine and the thin metal band and contact device shown in Fig. 3(b), was attached. This contact was adjusted so that any lateral swelling of the specimen, broke the contact and put out the light. By using a small current, electrolysis and arcing at the points was avoided.



(A) GENERAL ARRANGEMENT

(B) DEVICE TO INDICATE TENDENCY TO SWELL LATERALLY.

Adaptation of triaxial testing machine for consolidation tests.

FIG. 3

During the test, pressure was applied through a 1½ in. diameter piston, and any tendency on the part of the specimen to bulge was corrected by increasing the lateral water pressure until the light just came on again. In this way the diameter of the specimen could be maintained constant within very narrow limits, and the vertical compression of the specimen measured from a dial gauge calibrated in 0.0001 in. divisions reading on a mounting at the top of the piston. It was found that the device was sufficiently sensitive to react to changes in the vertical pressure of less than 1 lb., and in the lateral pressure of 0.1 lb. per sq. inch.

The method is experimental and requires regular attention to the water pressure in the perspex cylinder. There is, however, no difficulty in fitting an automatic relay and pumping arrangement.

Details of Soil and Stones Used.

One type of soil was used throughout the investigations described; this was a clay with the following characteristics:

Liquid Limit 39
Plastic limit 19

Mechanical analysis:		per cent
Coarse sand	(2mm - 0.6 mm)	13
Medium sand	(0.6 mm - 0.2 mm)	11
Fine sand	(0.2 mm - 0.06 mm)	16
Silt	(0.06 mm - 0.002 mm)	24
Clay	(Smaller than 0.002 mm)	36

Specific gravity of clay particles 2.73
Dry density as used 105 lb. per cub. ft.
Saturated moisture content 23.0 per cent.
Initial voids ratio = e_0 = 0.628

The stones were all sieved from a flint river gravel of irregular 1) shape. The sizes adopted were 3/4 - 3/8 in. for use in the 4 in. diameter specimens only, and 3/8 - 3/16 in. The specific gravities and moisture contents of the stones are as follows:

	3/4-3/8 in.	3/8-3/16 in.
Apparent specific gravity	2.48	2.45
Moisture content, saturated surface wet, per cent.	3.6	6.2

In all the experiments the soil mixture was compacted to a predetermined volume, in order to obtain the specified dry density in the clay, i.e., the initial voids ratio of the clay was kept constant.

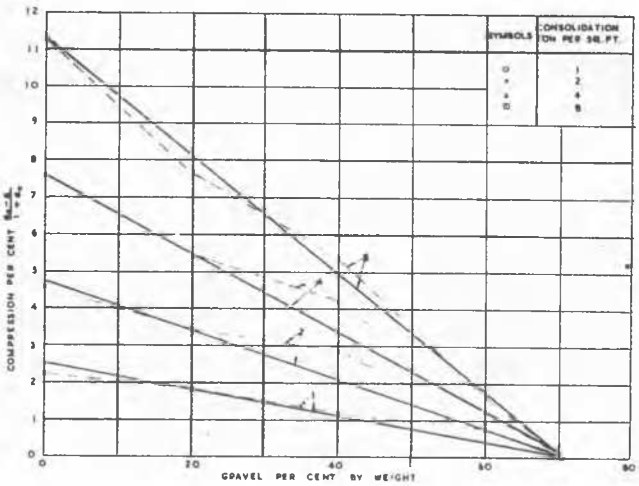
Percentage Compression of 3 in. Samples.

Consolidation tests were made on 3 in. samples ¾ in. thick with drainage both top and bottom, using clay and 0, 20 40 and 60 per cent of gravel by weight of total mix. In order to obtain a reasonably constant voids ratio on the clay for the series of tests, dry clay was mixed with the necessary quantity of water and was then mixed with the saturated surface wet gravel. The sample was next compressed in a cylinder to a predetermined volume.

In order to present the data obtained in a form as closely related as possible to a final estimate of settlement, the results of the tests are given as the percentage compression, that is $\frac{e_0 - e}{1 + e_0} \cdot 100$

where e_0 = initial voids ratio
e = voids ratio at end of consolidation due to applied load.

In Fig. 4 the percentage compression of

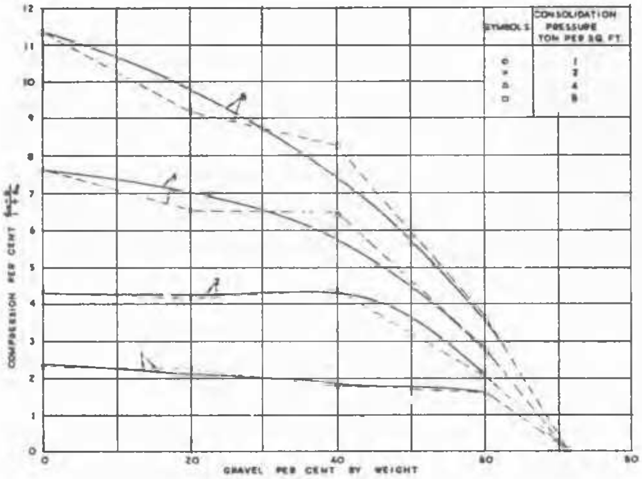


Effect of increasing proportions of gravel ⅜ in. - $\frac{3}{16}$in. on percentage compression of 3in. specimens, ¾in thick.

FIG. 4

the soil as a whole is plotted against the gravel content for four increments of consolidation pressure, 1, 2, 4, and 8 tons per sq. ft. As may be seen, the percentage compression varies inversely with the percentage of gravel by weight, and the compression becomes zero at a gravel content of about 71 per cent. This is the point at which the gravel particles make contact, and the load is therefore carried entirely by the gravel. There is a tendency for the percentage compression to fall less between 20 and 40 per cent than is expected from a straight line relationship. This tendency is even more marked in the 4 in. samples (see Fig. 6), and requires further investigation.

Curves have been plotted in Fig. 5 showing the percentage compression in the clay alone. At the lower loads of 1 and 2 tons per sq. ft., it appears that percentages of gravel up to 40 per cent have little or no effect on the consolidation in the clay. At 4 and 8 tons per sq. ft., however, and more especially with the higher load, the percentage compression decreases even with small proportions of gravel. It is, therefore, evident that errors are incurred if it is assumed that settlement in a soil can be computed from consolidation tests on samples from which stones have been removed, or on samples sèlected from a portion of the main sample free from stones, and that any such estimates are likely to be in excess of the settlements which actually occur.

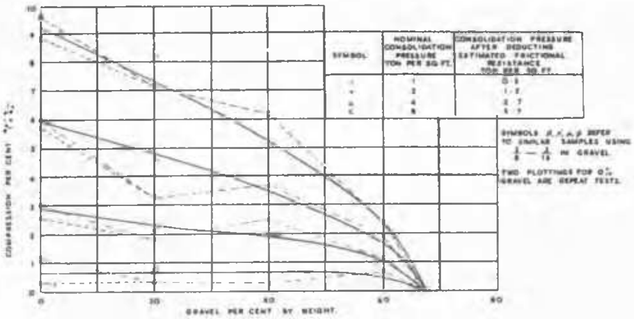


Effect of increasing proportions of gravel $\frac{3}{8}$ in - $\frac{3}{16}$in on percentage compression of soil fines only, 3in Dia. specimens, $\frac{3}{4}$in thick.

FIG.5

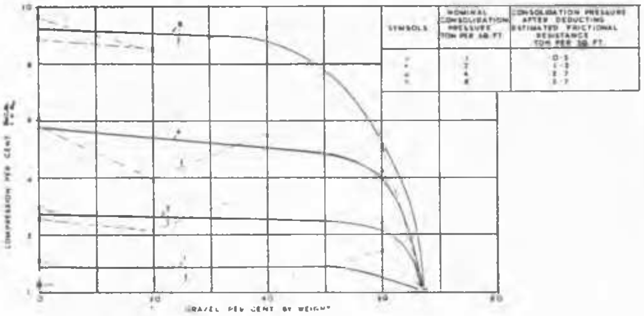
<u>Percentage Compression of 4 in. Specimens.</u>

The results of tests on 4 in. specimens are given in Figs. 6, 7, and 8. Specimens 3 in. diameter and $\frac{3}{4}$ in. thick are normally fully consolidated after 36 hours. Terzaghi 2) has shown that the time factor is dependent on the square of the thickness, and it follows that for specimens 4 in. thick, the comparative consolidation period is about 1,000 hours. In order to save time during testing, this period was not strictly adhered to, when it was found that little or no movement was occurring, and the compression was extrapolated by plotting on semi-logarithmic paper. It is felt that any error incurred is likely to be of minor importance.



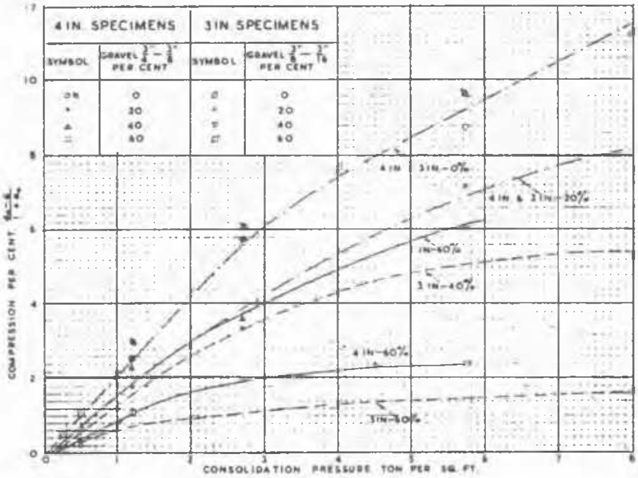
Effect of increasing proportions of gravel, $\frac{3}{4}$in. - $\frac{3}{8}$in. on percentage compression of 4in. Dia. specimens 4in. thick.

FIG.6



Effect of increasing proportions of gravel, $\frac{3}{4}$in. - $\frac{3}{8}$ in. on percentage compression of soil fines only 4in. Dia. specimens, 4in thick.

FIG.7



Comparison of percentage compressions of 3in. & 4in. specimens with various gravel contents.

FIG.8

The form of the curves obtained in Figs. 6 and 7 are somewhat similar to those given in Figs. 4 and 5 for 3 in. specimens. It is evident, however, that the percentage compression in each increment was lower than would be expected from the nominal loading, and that side friction had become much more serious than it was in the 3 in. specimens.

Taylor 3) in discussing the effects of side friction, showed that inaccuracies in the pressure in Boston Blue Clay could amount to 6 to 11 for remoulded soil and 5 to 7 per cent for the undisturbed state, when using samples only $1\frac{1}{4}$ in. thick. Other investigators 4) 5) have designed special apparatus in connection with investigations of side friction, and indications have been obtained that the friction is of larger magnitude than is generally believed. The result was, therefore, anticipated and the following approximate method of making an allowance was adopted.

After testing, the pressure required to push the sample out of the tube was determined, and shear box tests were also made to find the cohesion and angle of friction of the material. Estimates were then made of the value of the constant K in the equation:

$$A_c p = A_s (c + Kp \tan \phi)$$

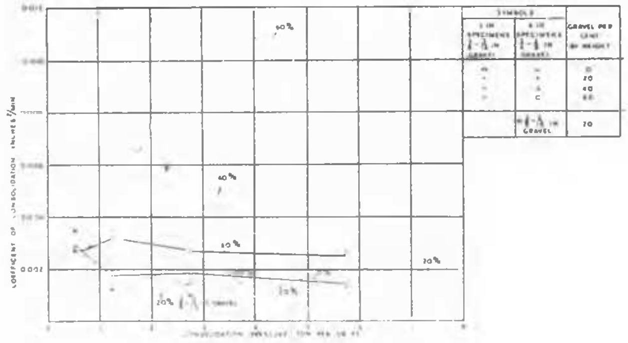
where A_c is the cross-sectional area of the sampling tube
A_s is surface area of tube in contact with the specimen.
p is the applied pressure
c is the cohesion
ϕ is the angle of friction

For all practical purposes with the material used in the tests, it was found that K could be taken as equal to 0.5, c equal to 2.87 lb. per sq. in. and ϕ equal to 19°. From this data, the actual load transferred to the bottom of the specimen for any load applied at the top can be calculated. Taylor 3) has shown that the consolidation is then equivalent to that due to the average of the pressures at the top and the bottom. Values for nominal loadings of 1, 2, 4, and 8 tons per sq. ft. are given in Figs. 6 and 7.

With these corrections the percentage compression is plotted against consolidation pressure in Fig. 8, and it is shown that for 0 and 20 per cent gravel, the agreement between the 4 in. and 3 in. samples is good. When, however, the percentage of gravel is increased to 40 and 60 per cent, a greater compression is obtained in 4 in. samples than in 3 inch. This appears reasonable, since with specimens ¾ in. thick, more arching and resistance to movement are to be expected when large quantities of gravel are present.

Rate of Consolidation.

Coefficients of consolidation were calculated using the time fitting method 3) 6) plotting degree of consolidation against $\sqrt{t}$ The coefficients are shown plotted against the consolidation pressure in Fig. 9.



Comparison of coefficient of consolidation of 3in. & 4in. specimens with varying gravel contents.

FIG.9

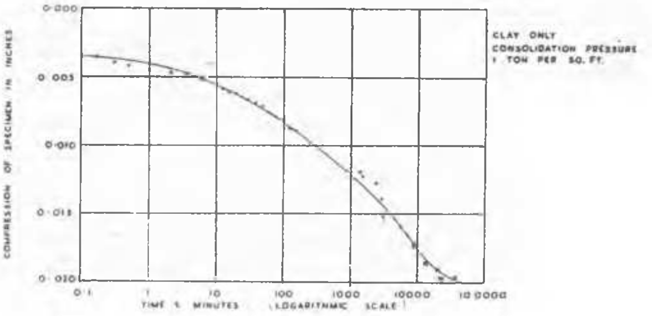
The most important point indicated by the plottings is the difference in the time rate of consolidation of 3 in. diameter specimens ¾ in. thick, as the percentage of gravel increases. In comparison there is little change for 4 in. diameter specimens, 4 in. thick, until the proportion of gravel reaches 60 per cent.

The large increases in the coefficient of consolidation for 3 in. diameter specimens when gravel is present, are attributed to arching and uneven consolidation, which results in a lowered resistance to the flow of water. Greater freedom of movement of the gravel is possible in 4 in. thick specimens, and any uneven consolidation in the thin layer adjacent to the porous stones top and bottom, is of less importance. Serious errors in time-settlement curves are therefore likely to result from the use of thin specimens of stony soil.

Use of Triaxial Compression Testing Machine.

The method evolved for this test has already been described under "Consolidation Apparatus". Preliminary experiments were made with a sample of clay 1½ in. diameter and 2½ in. high. The piston through which the load was applied, was made the same diameter as the specimen in order to avoid vertical compression when the lateral water pressure was increased. The use of the contact device at the centre of the specimen is based on the assumption that any increase in diameter is likely to be greatest at the centre, as is found in the normal change of shape of specimens in triaxial compression tests.

An example of the consolidation time curve obtained is given in Fig. 10. Owing to the fact that the experimental apparatus only had one drainage path, however, and the time required for full compression is estimated to be about 50 days for each increment, insufficient evidence is as yet available concerning the comparison with the normal consolidation apparatus. The results, are, however, encouraging and modifications are being made to provide drainage at the top and bottom of the specimen, and the contact device has been improved. Apparatus has also been designed to deal with larger specimens in the same way. In this case further modifications have been made to the loading piston arrangement, and a relay system is incorporated for automatic correction of any lateral swelling of the specimen.



Example of consolidation / time curve obtained when using triaxial compression machine.

FIG.10

Discussion of Results.

The investigations described show that the consolidation characteristics of a soil are affected by the presence of stones, and that computations of settlements cannot be made accurately by testing samples of the soil fines, and assuming that the compression is reduced in proportion to the volume of stones present. Errors are likely to be particularly serious in estimates of the time rate of settlement. It is, therefore, evident that for clays containing gravel or boulders, e.g., boulder clays, use of the standard apparatus with comparatively small samples is unsatisfactory.

When larger specimens are used, difficulties are encountered due to side friction in the consolidation box, but it appears that for a rough evaluation, corrections can be made from estimates of the reduction in pressure due to friction, and the assumption that the consolidation is due to the average of the pressure at the top of the specimen and the resultant pressure at the bottom. It is impossible to eliminate friction in the normal type of consolidation box, and the use of the triaxial compression type of machine with a device such as that used by the author appears likely to provide a solution of the difficulty.

The investigations described were regarded as exploratory and were made on one type of remoulded soil and one shape of gravel. Further investigations are necessary to determine the effect of the shape of the solid particles, effect of remoulding, and other properties before any definite conclusions on the general effect of the presence of stones and other coarse particles on the consolidation can be reached. This further research is dependent, however, on the final development of a suitable form of test for the large samples necessary.

ACKNOWLEDGMENT.

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SUB-SECTION II d

TRIAXIAL TESTS

II d 1

CORRELATION BETWEEN THE RESULTS OF CELL-TESTS AND COMPRESSION TESTS

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INTRODUCTION.

In the cell-tests performed at Delft and Ghent the sample is subjected to a certain number of vertical loads, and for each of these loads the minimum lateral pressure, necessary to still maintain the sample in equilibrium, is determined. When quick cell-tests are performed on clays, the obtained envelope of Mohr consists of two straight lines intersecting in the so called "singular point" of the diagram of Mohr. When the sample is given opportunity to consolidate under the last applied load, the necessary minimum pressure is decreasing with time. The circle obtained after consolidation is complete, is tangent to the first straight line of the envelope. The first line gives the cohesion c by its ordinate at the origin, and the angle of internal friction φ by its inclination. The second straight line gives the so called apparent cohesion c' and apparant angle of friction φ'. The ordinate W_c of the singular point should be the shearing-resistance of the material in its natural state in the ground, while the circle tangent to the two straight lines should give the natural vertical effective pressure σ_t.

For several clays the quick-test may not be performed too quickly. Indeed, because of phenomena at the surface of the very small clay-particles, the shearing-resistance of these clays under an instantaneously applied load can be larger than that existing under a remaining load. The velocity of performing the cell-test must be so, that the maximum value of the minimum lateral pressure, necessary to maintain the equilibrium under a given vertical load, can be measured, after what a new increase of the vertical load shall be applied as quickly as possible. In this way the normal cell-diagrams dealt with in this report have been obtained.

In the U.S.A. and in several other countries the shearing-resistance is computed from compression-tests. If q is the crushing strength of a sample, it is admitted that the shearing-resistance of the sample in its natural state in the ground W_o is given by

$$W_o = \frac{q \cos.\varphi}{2} \qquad (1)$$

In the same countries the shearing-resistance is also computed from triaxial compression tests. In the quick tests the sample enclosed in a rubberbag, is subjected to an overall