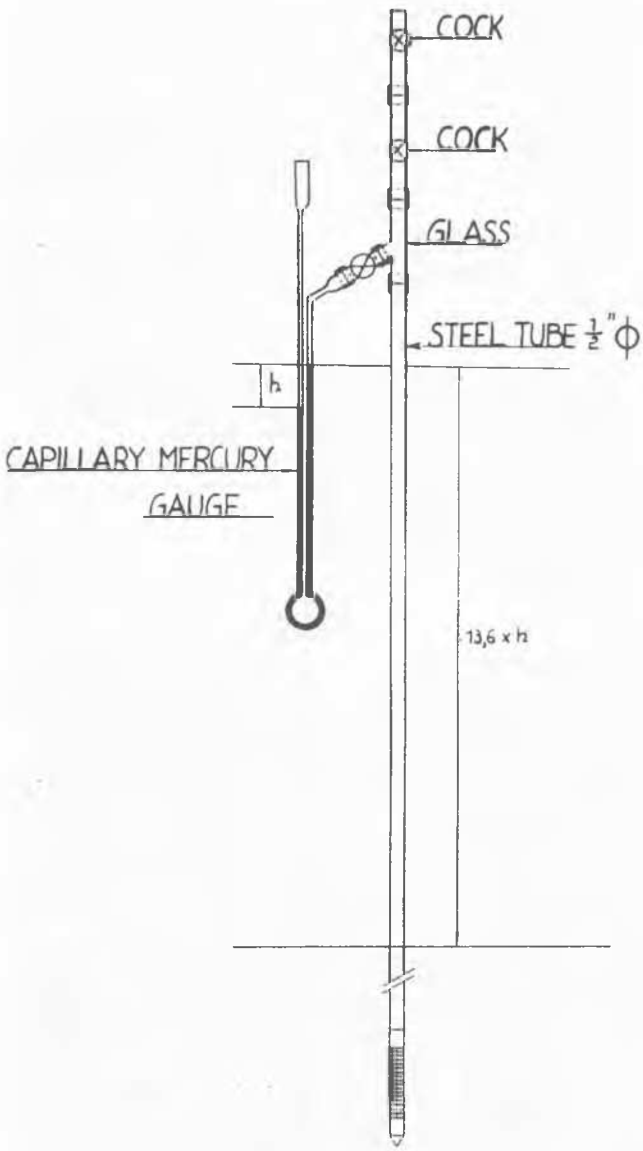

FIG.3

hydrostatic with respect to the water-level in the upper sandlayer R_{2b}.

Section 34 (Left Bank) fig. 2e.

Independent watertables are found in the layer Pl_1 of fluviatil origin and in the sandlayer R_{2b}.

In the claylayer R_{2c} was put the closed piezometer C. Although located very near the base of the claylayer, it doesn't show any influence of the waterlevel in the lower sandlayer, but the recorded pressure is nearly hydrostatic with respect to the waterlevel in the upper pervious layer Pl_1.

In the claylayer R_{1c} were put the closed piezometers A and B, which give nearly hydrostatic pressures with respect to the water-level in the upper sandlayer R_{2b}.

GENERAL REMARK.

All of the piezometers are not more distant than max 10 m from the transversal drains. Some are even located at only 4 à 5 m. from these drains. In spite of these very short distances, the piezometers located in the claylayers, and even those located in the clayey layers, are practically not influenced by the existence of these drains. At the contrary the waterpressures in all the sand- and gravellayers are very influenced by the same drains.

GENERAL CONCLUSION.

The waterpressure measurements performed at Godarville and Eigenbilzen indicate that in case of claylayers located between more pervious layers, in which exist two independent water-tables, the waterpressures in the claylayer are practically not influenced by the water-level in the lower pervious layer, but are generally nearly hydrostatic with respect to the water-level in the upper more pervious layer. These results are surprising, because the waterpressure in the clay should be expected to be a mean between the upper and the lower watertable. The much higher recorded waterpressures are a very peculiar fact for the stability of slopes in claylayers. When deep cuts are to be made in such layers it is thus suggested to control the waterpressures in the claylayers by means of closed piezometers.

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III c 3

LOADING TESTS ON CLAY

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SYNOPSIS.

This report deals with the strength of clay. In certain borderline cases, where ground rupture has occurred at a known load, the strength according to computation by tentative sliding surfaces is compared with the strength obtained in the laboratory by nonconfined compression tests and by Swedish cone tests on undisturbed soil samples taken from the same ground. A hypothesis is stated for the relation between the strength of the soil sample and that of the native soil, indicating in principle how strength of soil may be obtained.

INVESTIGATIONS.

In the laboratory, nonconfined compression tests are made on "undisturbed" soil samples according to fig. 1. The vertical normal stress is increased from zero until either real rupture occures, usually in an inclined sliding surface, or the height of

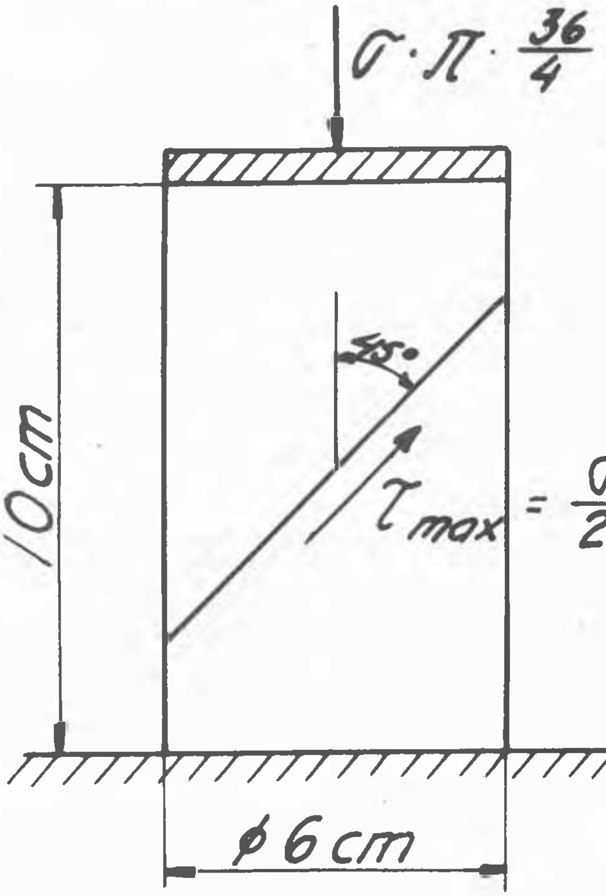
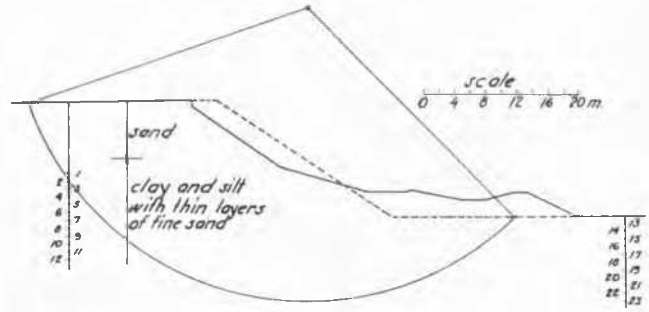


FIG.1



	γ	τ	c
1	1,98	0,602	1,13
2	2,02	>1,000	1,07
3	1,98	0,517	1,04
4	1,81	0,186	0,395
5	2,10	0,177	0,440
6	1,91	0,821	0,405
7	1,97	0,872	0,470
8			
9	1,96		0,145
10	1,90		0,185
11	1,91	0,185	0,859
12	2,00	0,814	0,580

	γ	τ	c
13	2,07	0,134	0,805
14	2,24	0,168	0,575
15	1,88	0,144	0,840
16	1,86	0,146	0,335
17	2,06	0,032	0,080
18	1,93	0,153	0,850
19	2,05	0,179	0,440
20	1,57	0,064	0,865
21	1,91	0,252	0,485
22	2,03	0,506	0,785
23	1,93	0,150	0,555

volume weight γ t/m³
shear strength in kg/cm²
according to
nonconfined compression test τ
cone test c

FIG.2

the sample has been decreased 10%, this strain being considered equivalent to a real rupture. During the test the soil sample is placed in paraffin oil, preventing it from drying. The rate of the load increase is 0,05 kg/cm²h. For such fine-grained clays, as are now in question, this rate is great enough to render the consolidation of the sample negligible. The strength according to the test is $\tau_{max} = \sigma/2$ at rupture.

The cone test is another method of obtaining the strength of clay, which to some extent is used in geotechnical laboratories in Sweden and also in the other Scandinavian countries. In this test one observes how deep a small metal cone with sharp point sinks in the soil sample, when released from the position where the point touches the surface of the sample. By means of stability computations of known borderline cases, where the soil is considered to be near rupture, one has obtained a relation between the impress depth of the cone and the strength of the soil. This relation is different for different kinds of clay.

Usually the strength, according to nonconfined compression tests in the laboratory. is lower than the real strength of the soil according to computation with sliding surfaces. As an example from practice a stability computation for a slope of a clay pit in a brickyard at Falkenberg (fig. 2) may serve. The slope is 14-15 m high and very long. After a slide, which raised the bottom of the pit 2 or 3 m, a geotechnical survey was made, the results of which are shown in fig. 2. The natural ground, in which the pit is dug, has a top layer of sand, 7 m thick, and under it down to a great depth clay and silt with thin layers of "mo" (grain size 0,2-0,02 mm). A stability computation was made for the tentative sliding surface shown on fig. 2, assuming that the slope before the slide was as shown by a dashed line on the figure. Giving different importance to the different soil samples according to the length along the sliding surface which they represent, the average value for the cone tests is 0,52 kg/cm² and that of the nonconfined compression tests 0,28 kg/cm². The stability computation shows that the shear stress along the tentative sliding surface in the part through the clay is 0,51 kg/cm², and thus in accordance with the cone test but greater than the nonconfined compression test. These results are representative for many such cases.

In order to investigate further the relation between strength according to stability computations and strength according to laboratory experiments the author has made loading tests on clay in the field. These tests were conducted in pits 1,1 m deep and 2,0 m square, which were cut through the dry crust to the underlying soft clay. In the test place the ground has never been consolidated by any overburden. The soft clay was homogenous in the thin layer, the strength of which was tested. The test slab was made up of steel beams and steel plates and was rigid compared with the clay. Its area was 2,0 x 0,4 m square, and it was placed on the levelled bottom of the pit along one of the sides. The load was applied on the centre of the slab from a steel bar 1 m in height, which had a bearing at each end and which before the test was made exactly vertical with water-level. The load was produced in a special loading test device by pouring water into a barrel, the total weight applied to the steel bar being enlarged 10 times by a lever arrangement. The device was arranged in such a way, that only the load from the test slab acted on the tested earth volume. The load was increased at such a rate, that the rate of the shear stress increase in the presumed sliding surface was the same as in the nonconfined compression test. The sinking of the slab was measured by micrometer dials at its 4 conters, and the raising of the pit bottom beside the slab was measured at 8 points along a line at right angles to the length of the slab drawn from its centre. The load was increased until ground rupture occurred. The rupture was very distinct as seen in fig. 3, which for one of the tests shows the sinking of the corners of the slab and the corresponding load. The loading tests were made in 9 pits.

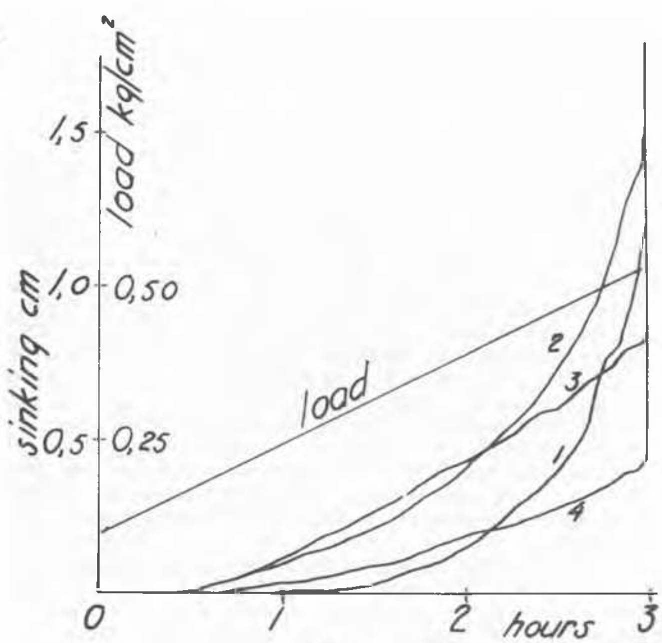


FIG.3

The load at rupture being q, the shear stress τ_B in the circular arc sliding surface is computed to $\tau_B = 0{,}173$ q the stabilizing moment of the shear stresses in the end faces of the moving soil mass being taken into account. In each test pit 4 soil samples were takên, on which nonconfined compression test and cone test were made in the laboratory. The results from the loading tests and the laboratory tests are collated in fig. 4. The results of the nonconfined compression tests appear to correspond well to the real strength of the soil, while the values of the cone tests are somewhat too high.

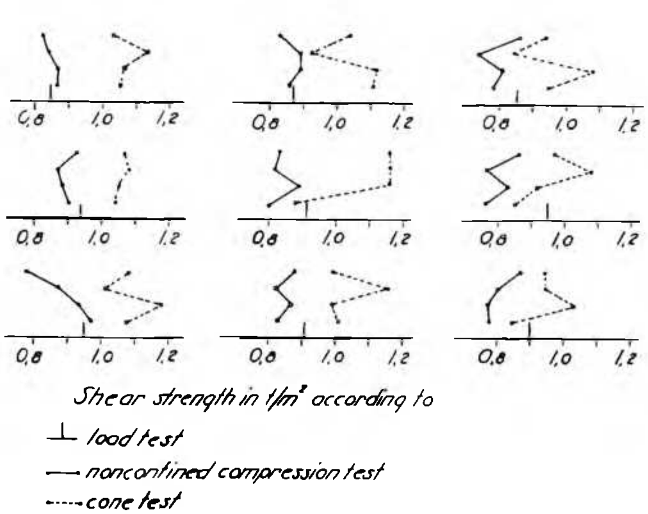


FIG.4

THEORETICAL EXPLANATION.

As mentioned above the nonconfined compression test usually gives too low strength values. The author will give a tentative explanation of this circumstance. This explanation also shows why in the loading tests described above the relation between the strength according to laboratory tests and the real strength of the soil (i.e. according to stability computation at ground rupture) diverged from its usual value. For this purpose the stress conditions of the soil before and after the sampling in the laboratory test are considered.

The clay is considered as a 2-phase system consisting of a solid phase, the grain skeleton, and a fluid phase, water. To the solid phase may then be assigned that part of the water which, owing to molecular forces, more completely adheres to the grains and possibly surrounds them as a more or less solid envelope. The fluid phase consists of ordinary water. The strength of the clay,i.e. its capacity for transmitting shear stress is then identical with the strength of the grain skeleton. The grain skeleton is considered as a somewhat yielding but yet solid body possessing cohesion, owing to internal molecular forces, and internal friction. Slow shear tests, which have been published, show that the angle of internal friction of clay, i.e. of the grain skeleton, usually is arc tg 0,2 to arc tg 0,4. The rupture condition of the grain skeleton is shown in fig. 5 where Mohr's representation is used and the angle of internal friction is presumed as arc tg 0,3; the principal stresses p_1 and p_2 then cause rupture if the revelant Mohr's circle touches the inclined straight line that is

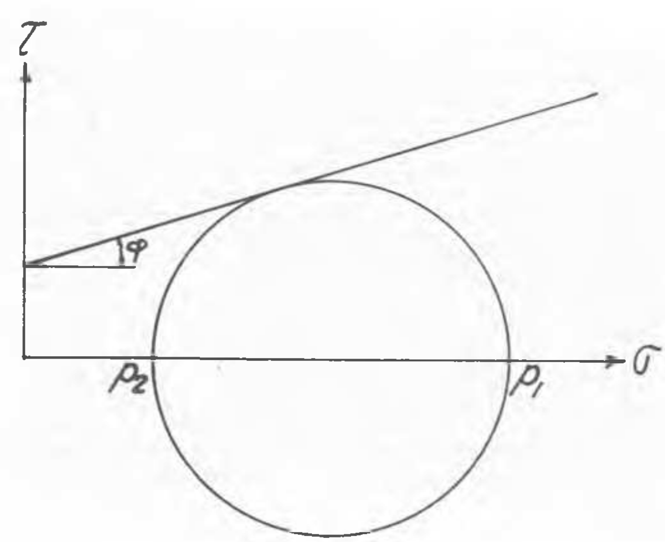


FIG.5

characteristic for the substance. In order to illustrate the stress conditions somewhat further we may assume that the grain pressure in the soil after sampling is p_a (fig. 6). This grain pressure is hydrostatic, i.e. equal in all directions. Furthermore we assume that the grain skeleton behaves like a homogeneous, isotropic elastic body following Hooke's law. The soil sample being subjected to nonconfined compression of the amount σ, the normal stress in the grain skeleton increases by $\Delta p_1 = 2/3\sigma$ in axial direction and decreases by $\Delta p_2 = 1/3\ \sigma$ perpendicularly to the axis, and the pore water pressure increases by $1/3\ \sigma$. The total

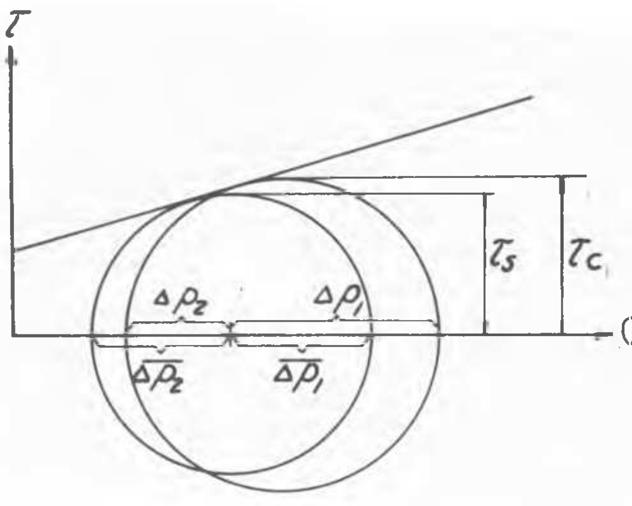


FIG.6

change of pore water pressure and axial grain pressure then equals the load; further the equation $\Delta p_1 - 2\Delta p_2 = 0$ is valid, i.e. the volume of the grain skeleton is unchanged, the pore water preventing volume change. The shear strength of the soil sample is found to be τ_c. If, instead of this, the sample is subjected to direct shear test at constant water content, the equation $\overline{\Delta p_1} = \overline{\Delta p_2}$ is valid, and the shear strength τ_s is obtained.

If an entirely undisturbed soil sample could be taken, the original vertical and horizontal grain pressures should be converted during the sampling to a hydrostatic average pressure p_a^1 of such a magnitude that the volume would be unchanged. The shear strength τ_B^1 should then be obtained in the laboratory (fig. 7). In reality also an "undisturbed" soil sample becomes somewhat disturbed during the sampling, as is apparent inter alia from confined compression tests. The grain skeleton may now be compared to a kind of framework having the grains as bars and exposed in the ground to the internal average stress p_a^1. During the sampling the adhesion between the grains break down to some extent and thus the grains find a chance to avoid the pressure; they turn and slide one on another and the average pressure of the grain skeleton or the framework sinks to a lower value p_a. In the laboratory then a shear strength τ_B is obtained which is lower than the shear strength τ_B^1 of the native soil. Evidently the decrease of the grain pressure mentioned can take

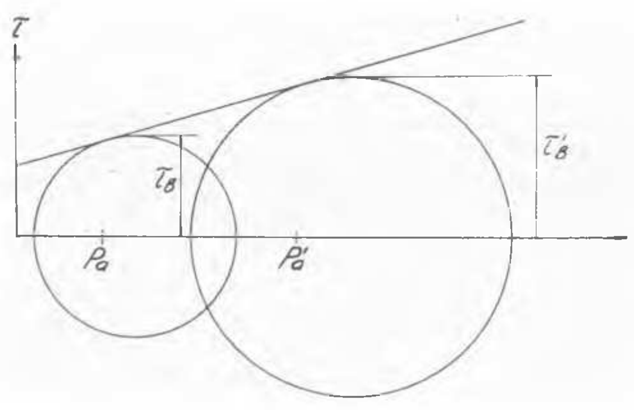


FIG.7

place without volume change precisely as the decrease of strength when clay is remoulded at maintained water content.

It is to be expected that in entirely consolidated clay layers the grain pressure and thereby also the strength usually should increase downwards. Such is the position at those tests with a rotating auger which are published by Lyman Carlson in a report to this conference. In accordance therewith the absolute difference between the vertical and horizontal grain pressures of the soil probably increases downwards, at least in ordinary cases. Therefore if an entirely undisturbed soil sample could be taken, the change of the grain pressure during the transition to hydrostatic pressure also should increase with the depth, from which the sample was extracted. The injury to the soil sample, i.e. the reduction of the hydrostatic grain pressure from p_a^1 to the lower value p_a, therefore seems to increase with the depth from which the sample was taken, provided that it is caused by the change of the grain pressure mentioned above. The injury can also be caused by the sampler acting on the earth mass out of which the sample is stamped; also in this case the injury seems to increase with the depth from which the sample was taken, as the greater average grain pressure makes possible a greater pressure reduction in the grain skeleton.

The hypothesis that the injury is greater at greater sampling depth explains why in the laboratory the nonconfined compression test and the rapid direct shear test give lower strength than corresponds to the real load in those cases, (e.g. Falkenberg) where the bearing capacity of the earth to a great extent depends on the strength of rather deep layers. The circumstance, that the cone test often gives a better result, is dure to the fact that it is calibrated from such cases. Furthermore the hypothesis explains why on the other hand the load tests in the field and the nonconfined compression tests in the laboratory belonging thereto correspond fairly well; owing to the small sampling depth in this case no pressure reduction in the grain skeleton occurred at the sampling. The reason why the cone test gave somewhat too high values may be explained by the circumstance that it is calibrated from cases where the sampling involves strength reduction of the sample.

Thus, according to the stated hypothesis the following should be observed. Prior to executing the nonconfined compression test or the rapid direct shear test in the laboratory, the sample should consolidate in such a way, that its grain pressure is restored to the same value as in the native soil. Otherwise the strength should be determined in some way directly in the ground. (The reader may compare with the conference report by Lyman Carlson concerning a rotating auger).

SUMMARY.

Experience shows that nonconfined compression tests and rapid direct shear tests at maintained water content give lower strength than corresponds to those normal loading cases, where the bearing capacity of the earth at least partly depends on the strength of rather deep clay layers. The inferior strength of the soil sample is explained by the average grain pressure being reduced at the sampling. The grain skeleton is thereby

considered as a kind of framework under pressure in the earth, at the sampling the soil sample is injured in the manner that the "bars" of the framework, the grains, to some extent turn and slide one on another, whereby the average grain pressure sinks from p^1_a to p_a and, according to fig. 7, the shear strength from τ^1_B to τ_B. This statement conforms also with loading- tests which have been carried out.

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III c 4

MEASUREMENT OF PORE WATER PRESSURE

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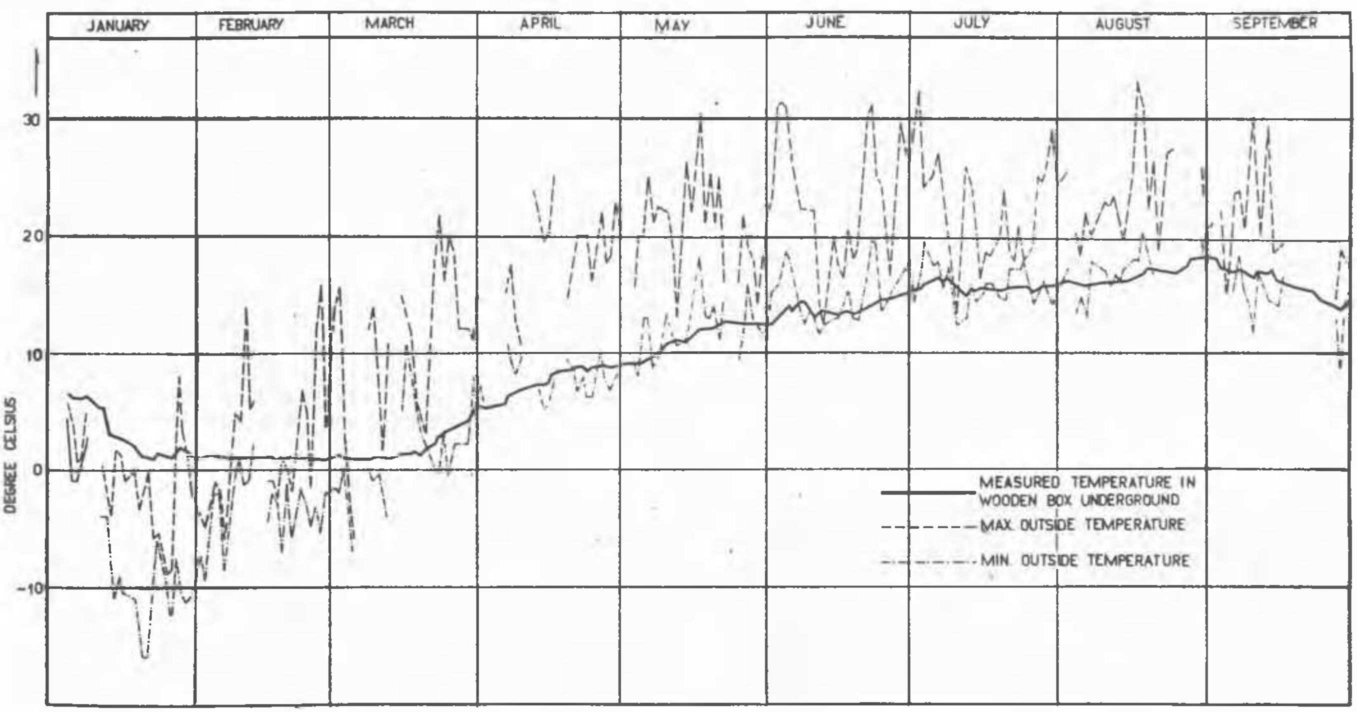
INTRODUCTION.

In the Proceedings of the First Conference on Soil Mechanics Ringeling and Biemond reported on the measurement of pore water pressure in peat and clay layers (F 8, F9). The importance of such measurements in view of the preparation, the design or the construction of civil engineering works needs hardly be stressed nowadays. Since then many measurements have been carried out in the Netherlands, in most cases under the guidance or under the supervision of the Laboratory. In the course of these measurements we encountered various difficulties which had to be overcome, resulting in the gradual improvement of the apparatus.

OBJECT.

The measurements may serve several objects, e.g. to collect the necessary data for the estimation of the probability of earth failures in excavation works, both for a con-



Measured temperatures in 1942

FIG.1