the shearing test was begun.

Displacements were comparatively large, but curves were very smooth and no jumps occurred (Fig. 7). In the light of the theory previously mentioned, in this case two processes come about simultaneously: the equalization of normal and tangential stresses of internal friction.

The active part of both stresses took place with time, and therefore the actual relative shearing stress (the ratio between active tangential and normal stresses) was greater than in the corresponding stage of the common test (Fig. 4). Hence the displacements must be greater (Fig. 7).

On the other hand by the same reason the excessive stresses are smaller compared with the common test, and no jump occurs.

Experiments with Quartz Powder. Fig. 8 shows results of an experiment made with dry quartz powder. The process is accompanied by a successive series of jumps. The first jump occurs at the relative shearing stress equal to 0.250, the next ones with smaller intervals. This fact is a result of the conservativeness of the sand structure, owing to which the discharge of excessive stresses must be accomplished chiefly in jumps, but not in smooth displacements.

Fig. 9 shows the result of a range of experiments with fine silty sand. Rather consistent stress-strain curves were obtained, proving the above statement.

Finally, Fig. 10 illustrates results of an experiment with dry quartz powder, the shear beginning immediately after the vertical load was put on. In spite of jumps, which are also presented here, the curve has smoother appearance. As before, deformations are greater, compared to the case, when consolidation was finished before the shear began.

Experiments with Clay. Finally a series of experiments with clays was made. The material represents typical varved clay (Bänderton), a sort of fluvio-glacial soil. In undisturbed state it consists of thin sandy and clayey layers. Atterberg limits of the soil are: lower liquid limit 30.7, and the plasticity index 11.0.

Fig. 2 represents a result of the test with remoulded varved clay. The character of stress-strain curve is easily understood in the light of the aforementioned. A jumplike re-organisation of structure occurred in the interval of stress $k = 0.325 - 0.350$. The remaining parts of the diagram are very smooth. Note, that the self-orientation did not take place, because of the presence of sandy particles, which prevented the free movement of the scale-like clay particles in the remoulded soil mass.

This test was performed with unloading after the shearing strain reached $k = 0.55$ (Fig. 2). Both curves of unloading and repeated loading are quite symmetrical. The dotted line (Fig. 14) shows the unloading curve turned upside down.

The coincidence is extremely good ($1 - 2\ \mu$).

The time-displacement curves for successive increments of the shearing stresses are shown in Fig. 2 (c) to (f). Note, that the shape of these curves for the process of the jump (increment $k = 0.325 = 0.350$, section op) is quite different compared with the other time-displacement curves of the same series. This attests the difference of the nature of the two processes.

No. D-9 THE INFLUENCE OF THE SPEED OF LOADING INCREMENT ON THE PRESSURE VOID RATIO DIAGRAM OF UNDISTURBED SOIL SAMPLES

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Clays and slimes have a very large importance in foundation engineering, being frequently the cause of settlements of buildings, due to their large compressibility. The final settlement of buildings founded on these soils is not reached before a long delay, because of their feeble permeability.

Following the up till present settlement observations on already executed buildings, there was a very good agreement between the calculated and observed settlements, for the buildings founded on medium soft or soft clay and slime.

But it was often shown, that the calculated settlements were too large for the foundations made on mighty deposits of stiff fat clay, while for the ones made on weak beds, or beds separated by sand lenses the differences were less important.

The reason of this observation was experimentally found to be due in a large part to the way the loading was increased in the compression apparatus.

Long experiments were made on a stiff, little permeable, clay and some on medium stiff clay and slime, in such a way as to bring the charge very slowly on the different samples.

The differences in the pressure void ratio diagrams between the standard and slow pressure increment, were very small for the soft clay and slime, for the slime there was nearly no difference at all. But opposite to these there was rather large difference for the compression curves of the stiff Paris region clay.

All the compression tests were executed in the compression apparatus designed by Prof. Terzaghi, which is based on the following principle: the sample is placed between two porous stones in a cylindrical tool, and lateral extension is not allowed. Depending on the pressure the sample can give up or absorb water. We will not go further into the theory of the compression test, but we give the bibliography of the question at the end.

The removing of the undisturbed sample was made with the tool used by the Paris Soil and Foundation Laboratory. It consists of two cylinders: one out and one inside. The inside cylinder is cut in two

following two generants in two half-cylinders. Inside the second cylinder is a thin-walled tin can, also cut in two along its generatrices, which takes up the sample.

The sampling apparatus is fixed on the drill rod and pressed by means of a hydraulic press in the drill hole ground. As soon as the sample is in the apparatus it is sheared off by turning the drill and the apparatus around 180°. The inside cylinder is brought out; the tin containing the sample has its two ends covered and is transported to the laboratory.

The stiff clay used for the experiments, belongs to the formation of the "argile plastique de la region Parisienne", and was removed from a 14 m deep drill hole. It is a stiff little permeable clay, lead gray to gray-blue, remarkable for its homogeneity. It can be considered as isotropic as it gives the same physico-mechanical properties in two directions normal to each other.

Its consolidation pressure must have been of about 25 kg/cm^2; concluding from the geological conditions the higher pressure must be due to its drying out.

The most important soil physical numbers are:

Original water content	30%	
Specific gravity	2.67	
Liquid limit	92%	
Plastic limit	24%	
Compressive strength	4 kg/cm^2	(1)
Angle of internal friction	16°30'	(2)
Cohesion	1.4 kg/cm^2	

(1) The break always appears as a shearing fracture, with well expressed sliding planes. A good agreement was always observed between the angle of internal friction and the angle on the gliding plane. See photograph.

(2) Determined in the shearing apparatus of Casagrande. The increment in the shearing force was every 2 minutes of one hundredth of the vertical charge.

One more specific property of the "argile plastique", as well as of all the clays of the Paris region, is its extraordinary swelling property, as soon as it is free of its external pressure, and if there is any possibility of taking water out of its immediate neighbourhood. To determine the swelling property of the clay, an undisturbed sample was brought into the apparatus, while the weight of the piston was balanced out by a special disposition, so that there was no external pressure on the sample. (See Fig. 2). The void ratio of the undisturbed clay was 0.846, at the end of the swelling it had grown to 1.280, which is an increase of 51%.

The swelling curve of the "argile plastique" is given on Fig. 3, expressed in per cent of the whole swelling. Abscissa represent minutes. Three French clays characteristics are shown on the figure: curves I and II correspond to samples of the Sparnatian clay formation, one removed from Paris immediate surroundings (curve I), the other from St. Brice near Provins (curve II). Curve III corresponds to a recent clay formation, from the Loire estuary, removed near St. Nazaire.

Typical is the turnpoint of the swelling curve of the Paris region clays, which has been noted every time.

Fig. 5 shows the same undisturbed samples of "argile plastique", submitted to four times repeated compression test, without any loss by the material of its original elastic properties. At the start of every run and at the end of it, the clay sample was entcharged as shown on Fig. 2. The corresponding swelling points are pointed out on the void ratio diagram and numbered from 0 to 4. The tests are carried up to 6.5 kg/cm^2, and at the end of every test a little diminution of the swelling property was observed. It is interesting to note that the endpoints of the compression diagrams fall on the same point.

It was now interesting to inquire if the speed of loading increment, would have any influence on the pressure void ratio diagram, as well as on the direct determination of the permeability of the clay.

As we have already pointed out, this clay having an extraordinary swelling property, to avoid premature swelling and changes in structure, the tests were carried out to overlook the sample in the apparatus, and to load in such a way as to prevent swelling tendency, without compressing the sample.

Four compression tests were made. During every test the loading increment speed was different. Test I was made as the standard test, that means the charge was loaded on stepwise, and so as to double every loading to the preceding.

During test II the loading was increased 200 gr/cm^2 daily
" " III " " " " 70 " "
" " IV " " " " 40 " "

Fig. 6 shows the speed of loadings for the four tests, abscissas representing time in weeks, the ordinates giving the loadings in kg/cm^2 and the void ratio.

Fig. 7 gives the pressure void ratio diagram, obtained and in particular curve I the standard test, curves II, III, IV, the tests where loadings were delayed.

The differences are notable between the different compression curves, and the curves get flatter and flatter while the loading was slower on the sample.

The Table below gives an overlook over the compressibility factors, deduced from the different curves: a =

Test N°	Loading = 2 kg/cm^2	Loading = 3 kg/cm^2	Loading = 5 kg/cm^2	Loading = 6 kg/cm^2
I	$3.12 \cdot 10^{-5}$	$2.58 \cdot 10^{-5}$	$2.20 \cdot 10^{-5}$	$2.12 \cdot 10^{-5}$
II	$3.00 \cdot 10^{-5}$	$2.44 \cdot 10^{-5}$	$1.96 \cdot 10^{-5}$	$1.74 \cdot 10^{-5}$
III	$2.58 \cdot 10^{-5}$	$1.96 \cdot 10^{-5}$	$1.58 \cdot 10^{-5}$	$1.42 \cdot 10^{-5}$
IV	$2.54 \cdot 10^{-5}$	$1.50 \cdot 10^{-5}$	$1.50 \cdot 10^{-5}$	$1.26 \cdot 10^{-5}$

The same table carries the directly obtained permeability observations, belonging to the pressure void ratio diagrams. As well as the compression curves, the permeability shows with equal pressure but different loading speeds, notable differences.

The matter of these differences has probably two reasons. During the standard test and stepwise loading, shocks while loading are unavoidable, even if great care is taken; opposite to it, there certainly are no shocks in loading with small weights at a time, with the continual loading method.

The second reason could be due to the clay itself. Following to a very general consideration, clay is composed of a lamellar structure in which separate particles are tied together by a colloidal mass. All the pores are filled with water. The solid as well as the liquid part being but slightly compressible, the compression of clay can only be due to the squeezing out of water. If the loading increases slowly and without shocks, the colloidal mass can support changes in form as it can support a traction force, without loosing contact between the separate particles and without pushing one over the others. If now the pressure suddenly increases, the whole pressure will be supported by the interporous capillary water, which can not escape immediately because of the feeble permeability, and internal ruptures occurs in the colloidal mass, as the overpressure is at the moment higher than the traction resistance.

A statement of the rupture of the colloidal structure is found on the examination of the different swelling curves, of the tests made. The swelling of the clay after the standard test diagram I, is much smaller than the swelling of the samples which were slowly loaded. The ends of the tests I--IV are specially pointed out, the discharge was brought down to the own weight of the piston, down to about 50 gr/cm^2.

<u>2. Clay from St. Nazaire.</u> This clay is a medium stiff material, gray-green. Its grain size distribution is represented on Fig. 8; the amount of colloidal particles is very low. The sample was removed from a 6 m strong bed, and about 11 m depth.
The most important soil physical numbers are:

Original water content..................45%
Specific gravity......................... 2.69
Liquid limit.............................67%
Plastic limit............................28%
Angle of internal friction...............19°
Cohesion................................. 0.200 kg/cm^2

Two compression tests were made: test I as standard test, test II with continuous pressure increment, about 60 gr/cm^2 daily.

The obtained pressure void ratio curves show but little difference. The difference between the direct permeability under equal pressure is nearly negligible.

This agreement must be due to the fact that little traction resistance of the colloids and rupture by overpressure is provoked in any case, as well with rapid as with slow loading increment.

<u>3. Slime from Rouen.</u> As a third example an undisturbed sample of slime, was examined, removed from the Seine valley near Rouen. The sample was removed from a drill hole about 10 m deep.

The original water content was of 121%. This slime is a very recent sediment and has a very high compressibility. Its grain size distribution is represented on Fig. 8. Compression curve I represents the standard test, curve II was obtained from a slow going test with pressure augmentation of about

60 gr/cm^2 daily. Fig. 10 shows the pressure void ratio diagrams obtained, the differences are nearly negligible.

It has been proved that the traction resistance in the colloidal mass of a slime is so little, that rupture occurs in any case of pressure increment.

On these three different soils, which are totally unlike in structure and consistency, the difference is shown, which the various manners of pressure increment, can have on the pressure void ratio diagram, of undisturbed soil samples. It seems to prove that the influence is larger as stiffer and less permeable the soil sample is. The difference in the compression diagrams of soft soils is much smaller and is not anymore notable for the slime samples.

So it would be required to find the law, for the stiff undisturbed samples, which exists between the loading increment speed and the differences in the void ratio diagrams, or to put a new method of research for this kind of soils.

It would be very desirable to have perfect settlement observations for the buildings founded on stiff clays, and to get in this way a correlation between test, theory and practice.

Further laboratory investigations, will be necessary concerning the influence of wall friction in the compression apparatus, on the void ratio diagrams, by tests with different sample diameter and sample thickness.

BIBLIOGRAPHY

Terzaghi — Erdbaumechanik, F. Deuticke, Vienna, 1925.
A. Casagrande — Structure of Clay in Foundation Engineering Boston Society of Civil Engineers. April 1932.

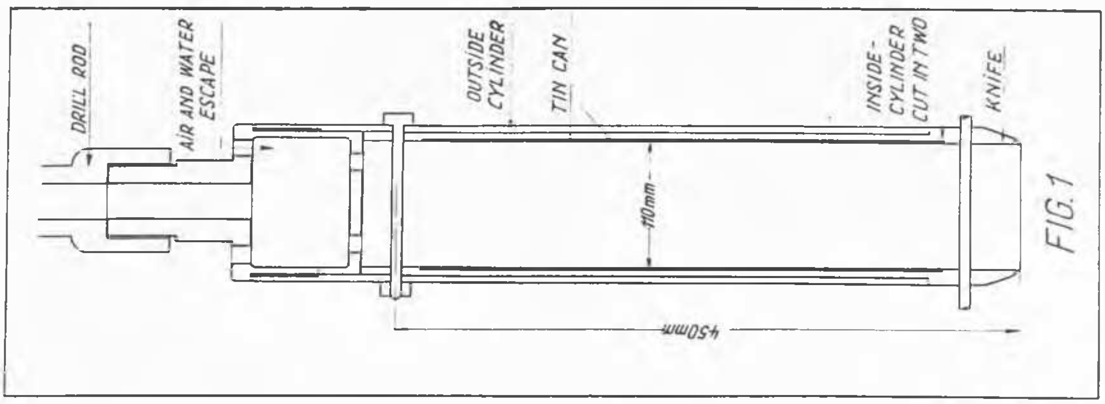


FIG. 1

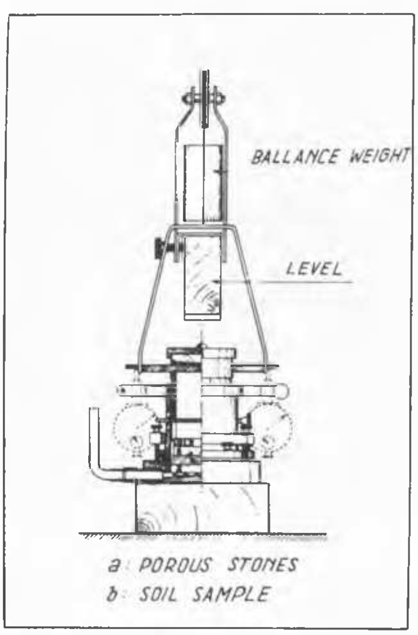


FIG. 2

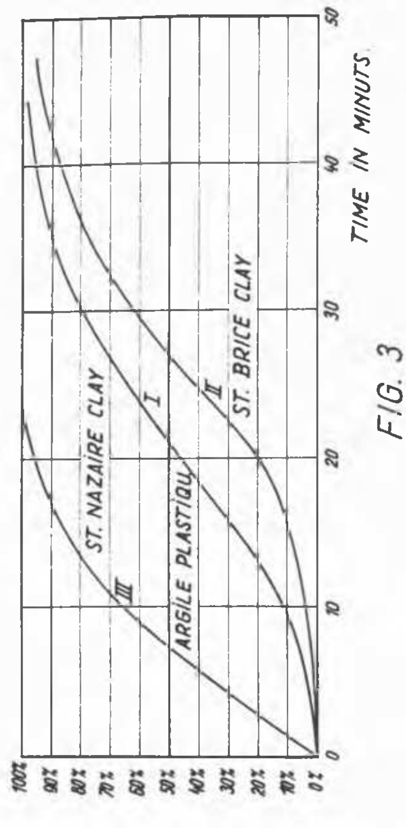


FIG. 3

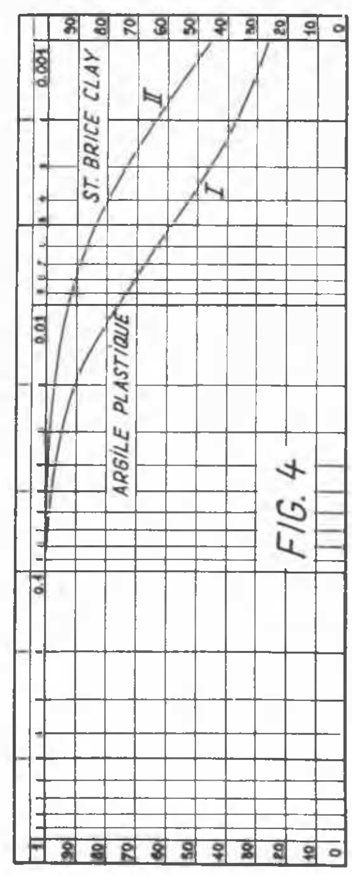


FIG. 4

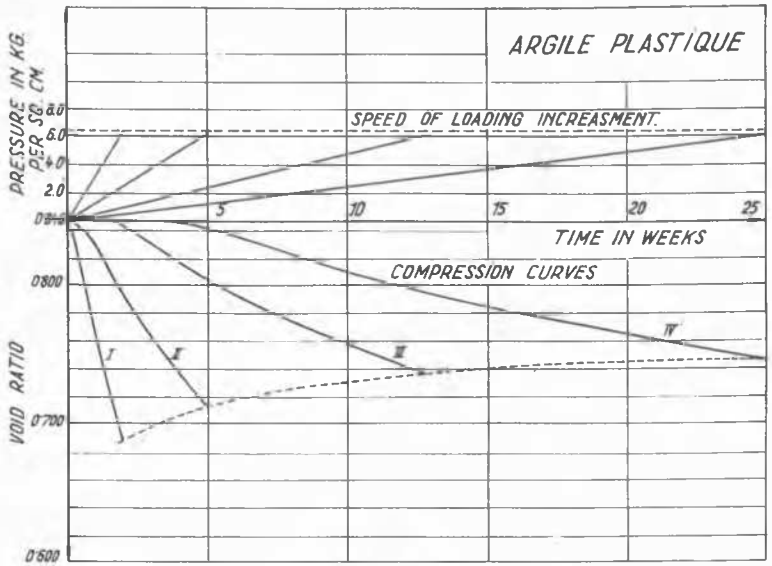


FIG. 6

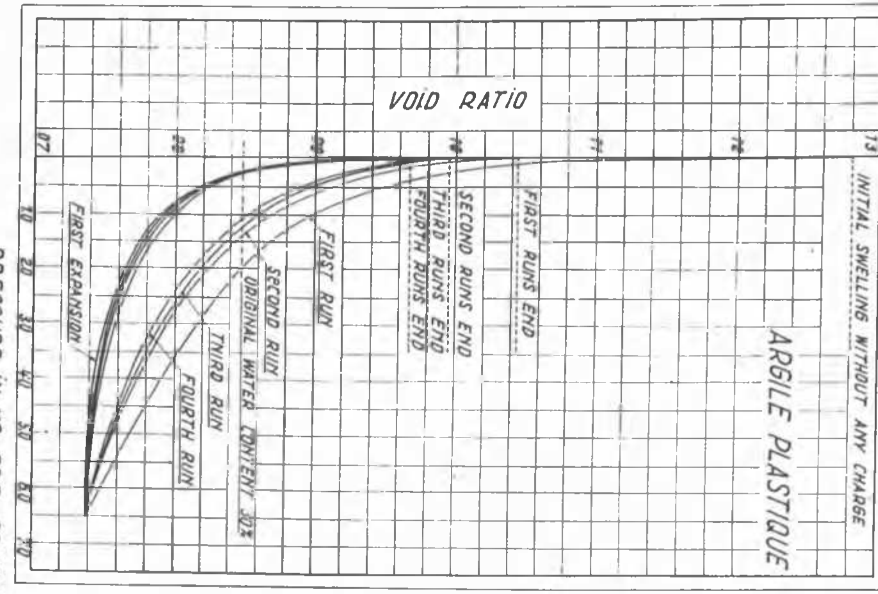
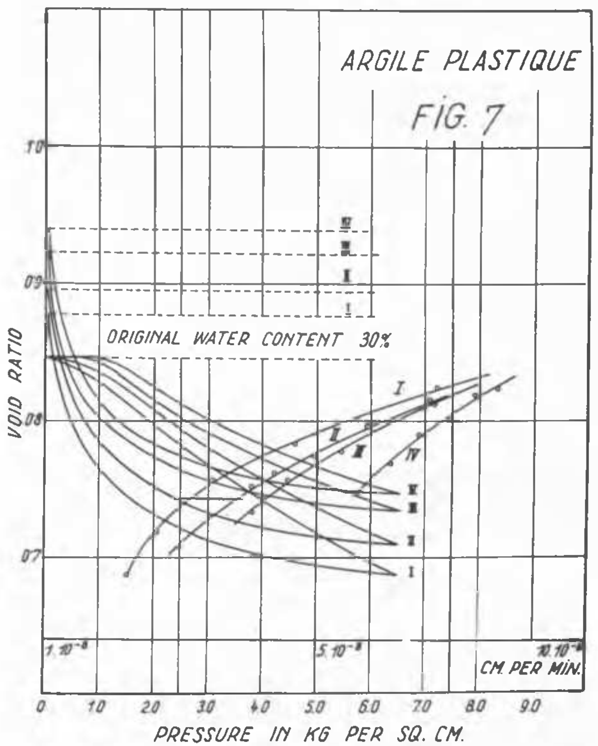


FIG. 5



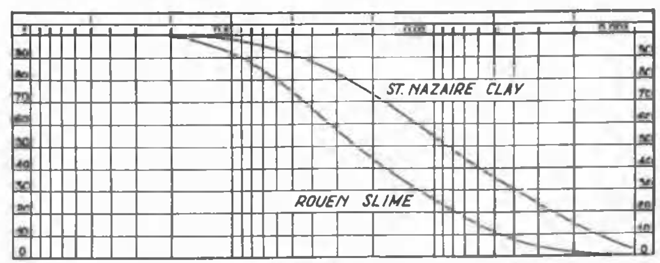


FIG. 8

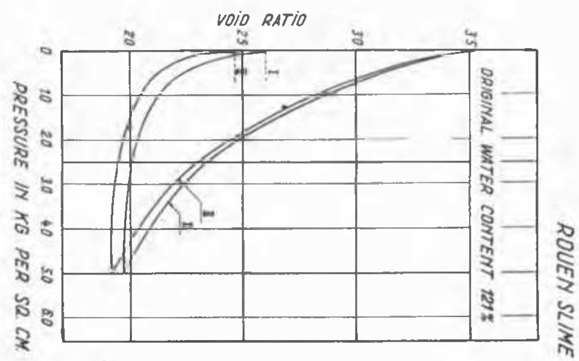


FIG. 10

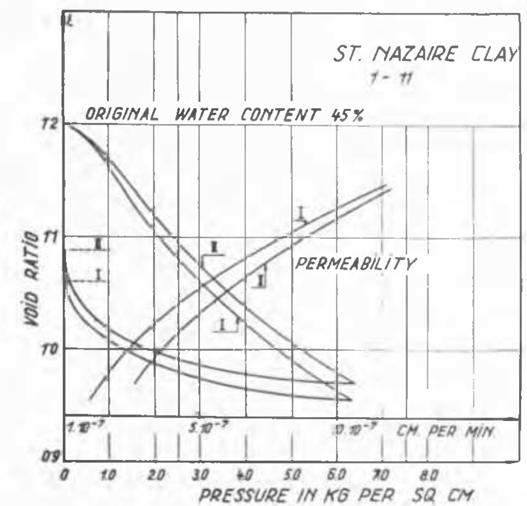


FIG. 9