ıte on top, and cause the piston to rise. It is unlikely that this slow w of water will cause any change in the void ratio of the unfrozen ɾt of the sample, but the rise of the piston should be observed and water accumulated on top of the sample measured in order to ermine whether the piston was in actual contact with the top of sample during the foregoing observations.

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UNDISTURBED CLAY SAMPLES AND UNDISTURBED CLAYS*

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INTRODUCTION

SINCE it is impossible to secure perfectly intact samples of clay (Terzaghi, 1936) the computation of the settlement of structures founded above beds of clay requires assumptions concerning those properties of the clay which cannot be determined by laboratory tests. The following paper deals with these assumptions. It also deals with the writer's conception regarding the physical causes of the difference between perfectly undisturbed clays and those of so-called undisturbed samples. For the sake of simplicity the discussion will be limited to clays which have never been under a pressure in excess of that produced by the weight of the existing overburden. Clays of this type are called normally consolidated clays.

THE VIRGIN COMPRESSION CURVE

Current conceptions regarding the consolidation of clay strata due to the weight of superimposed buildings are based on laboratory experience and on the results of settlement observations. In order to understand the origin of these conceptions let us assume that we have secured an undisturbed sample of a normally consolidated clay from

*This paper covers in general the subject of the paper entitled "The Critical Load on Strata of Clay beneath Foundations" presented at the January meeting of the Society.

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a certain depth below the surface of the ground. The water content of the sample is 45 per cent, corresponding to a void ratio $e_0 = 1.21$ The effective vertical pressure which acted upon a horizontal section through the sample, prior to its removal from the ground was $p_0 = 2$ tons per sq. ft. Since the deposit is normally consolidated the sample has never been acted upon by a higher pressure. In Fig. 1*a*

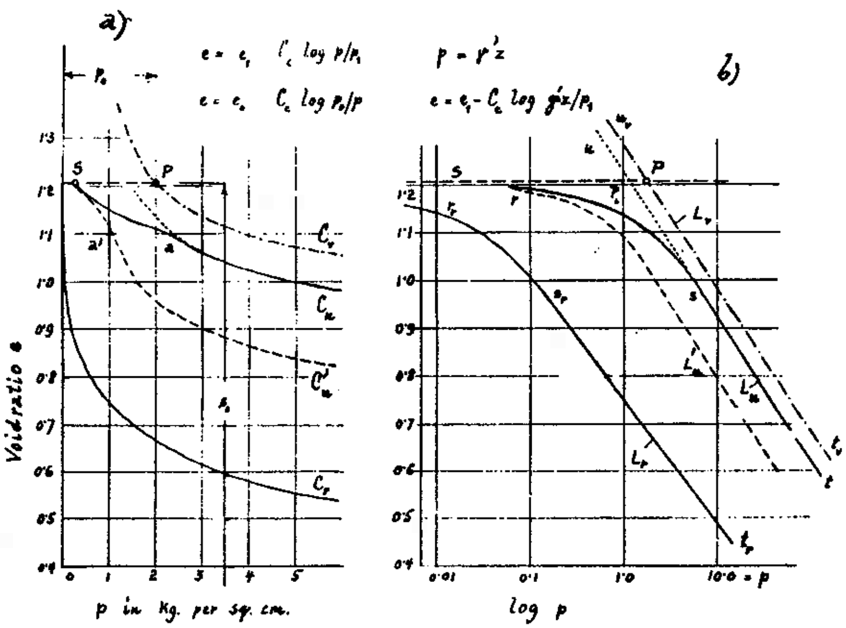

FIGURE 1*a*.

the abscissas represent the effective vertical pressure on the clay and the ordinates the corresponding void ratio. The original state of the clay is indicated by the point *P*. During the operation of sampling the void ratio remains practically unchanged, but the effective vertical pressure on the sample is considerably reduced. This state is shown by point *S*, the ordinate of which is equal to that of point *P*. The next step consists in submitting the sample to a consolidation test. The results of this test depend to a large extent on the degree to which the bond between the clay particles has been injured during the process of sampling. If the sample has been taken carefully, we obtain the curve C_u. At point *a* the curve shows a gentle break.

A. Casagrande (1932) has shown that the abscissa of point *a* is approximately equal to the pre-consolidation pressure, that is, to the greatest pressure to which the sample has ever been subjected in its previous history. For a normally consolidated clay this pressure is identical with p_0, Fig. 1*a*.

If the sample has been somewhat disturbed during the process of sampling a pressure void ratio Curve C'_u which is entirely located below C_u is obtained. The abscissa of the break a' in the curve is appreciably smaller than p_0. Finally, if the sample is tested after having been completely remolded at unaltered water content, the curve C_r is obtained. This curve has no break.

To show the characteristics of the test results more clearly the semi-logarithmic plot shown in Fig. 1*b* is useful. In this plot each of the curves L_u, L'_u and L_r, corresponding to the curves C_u, C'_u and C_r respectively, consists of a gently sloping, curved upper portion and a steep, straight lower portion. The straight line portion of each curve, as for instance the straight portion *st* of the curve L_u, can be described by the empirical equation

$$e = e_1 - C_c \log \frac{p}{p_1} \qquad (1)$$

wherein p_1 is an arbitrary reference pressure, e_1 is the void ratio corresponding to the pressure p_1 on the straight line *tu* and C_c is an empirical coefficient, called the compression index. Rutledge (1939) has demonstrated that the value of C_c increases in a general way with increasing values of the natural water content of a clay. For a clay with a natural water content of 45 per cent such as that represented by Fig. 1 the value of C_c ranges between 0.3 and 0.6. The curve L_u represents a clay for which $C_c = 0.3$. The value of C_c decreases slightly as the degree of remolding increases. For normally consolidated clays the extension *su* of the straight portion *st* of the pressure void ratio curve obtained from undisturbed samples always intersects the horizontal line *PS* at a point P_t which is located in the vicinity of the point *P*. The more the bond between the clay particles has been damaged during the process of sampling, the more the point P_t moves towards the left.

On the basis of observations similar to those shown in Fig. 1 it has become customary to assume that the decrease of the void ratio produced by increasing the load on a stratum of normally consolidated clay from the overburden pressure p_o to a given pressure p could be computed by substituting $p_o = p_1$ and $e_o = e_1$ into Equation 1. Thus we obtain

$$e = e_o - C_c \log \frac{p}{p_o} \qquad (2)$$

In Fig. 1*b* this equation is represented by the straight line $u_v t_v$ which is parallel to ut. The line $u_v t_v$ has been given the name ***virgin compression*** curve. The curve C_v in Fig. 1*a* represents the same curve in an arithmetic plot. If Equation 2 were justified, every increase of the load on the clay should initiate a normal process of consolidation. The settlement due to increasing the load on the clay by Δp should be greater than one-half of the settlement produced by an increase of $2\Delta p$ and the process of consolidation should take place in every bed of clay beneath the site covered by the surcharge, regardless of the depth at which such a bed is located.

The Sedimentation Compression Curve

The present state of every sedimentary, normally consolidated clay has been preceded by a gradual increase of the pressure from zero to the overburden pressure p_o. If Equation 1 were valid for both the process of loading in the laboratory and the process of loading by sedimentation in nature it should be possible to determine by means of this equation the relation between depth and void ratio. At any depth z below the surface of a submerged deposit of normally consolidated clay the overburden pressure p_o is equal to

$$p_o = z\zeta' \qquad (3)$$

wherein ζ' is the submerged unit weight of the clay. For a soft clay which has never carried any surcharge the upper end of the straight line portion of the semi-logarithmic consolidation curve corresponds to a pressure of not more than about 200 lbs. per sq. ft. In a submerged bed of clay this overburden pressure exists at a depth of about $z_1 = 3$ ft. below the surface. Introducing the value 200 lbs. per sq. ft. for p_1 and the value of $z\zeta'$ for p into Equation 1, the following equation is obtained

$$e = e_1 - C_c \log \frac{z\zeta'}{200} \qquad (4)$$

wherein e is the void ratio of the clay at a depth of 3 ft. below the surface of the deposit, e is the void ratio at a depth z meters below the

surface and ζ' is the submerged unit weight of the clay in lbs. per cu. ft.

The writer has knowledge of three reliable records concerning the relation between the depth and the void ratio of submerged deposits of normally consolidated uniform clays. Yet none of them shows any resemblance to what should be expected on the basis of the theory represented by Equation 4. Hence it is necessary to distinguish between the virgin compression curve derived from the results of laboratory tests, which is a hypothetical curve, and the *sedimentation compression curve* which represents the real relation between the void ratio and the very slowly increasing pressure due to the weight of the overburden in a clay deposit during the process of sedimentation.

The first information concerning the relation between depth and void ratio of a bed of clay in a state of very slow growth by sedimentation was secured by the author in 1924. Samples were taken in boreholes drilled to a depth of about 80 feet below the surface into the deposit of soft organic clay which covers the bottom of the Golden Horn near Istambul. The drill holes were located at a short distance off the shore near Aiwan Serai. The liquid limit of the samples ranged between 56 and 65 per cent. The water content of the uppermost samples which were obtained at a depth of about 5 feet, were approximately equal to the liquid limit. According to the results of consolidation tests on remolded samples the water content of the samples from a depth of 80 feet should have been about 20 per cent below the liquid limit. Yet, these water contents were almost as close to the liquid limit as those of the samples obtained near the surface. These findings as compared to the results of the consolidation tests confronted the author with what appeared at that time to be an insoluble puzzle.

A second record was obtained from borings made in 1939 at the author's request in a deposit of micaceous lacustrine clay at the southern end of the Lago di Resia in upper Italy. The clay originated from a terminal moraine and the surface of the deposit was covered with shallow water. Altogether thirty samples were secured from 7 drill holes having a maximum depth of 60 feet. The liquid limit of most of the samples ranged between 50 and 75 per cent. With few exceptions the water content of each sample was close to the liquid limit. Yet the difference between the liquid limit and the natural water content had no tendency to increase with depth. In this respect the record is a duplicate of that obtained at the Golden Horn for a marine clay.

A third record was published by B. Fellenius (1936). It shows the relation between the water content and the depth below the surface of a remarkably uniform deposit of soft clay in the Gota River in southern Sweden. The surface of the deposit is covered by about ten feet of water. Curve C_s in Fig. 2 represents Fellenius' data, recalcu-

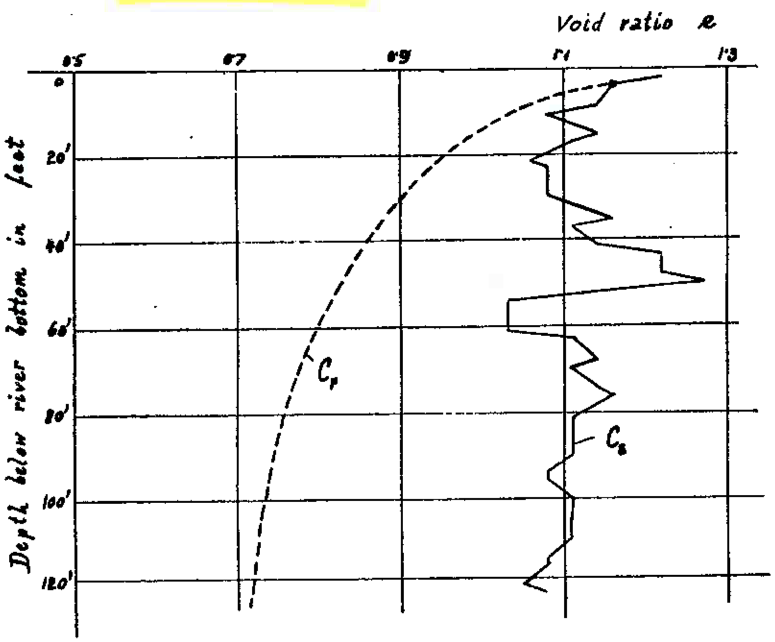

FIGURE 2.

lated on the assumption that the average unit weight of the solid soil particles is 2.7 gm. per cu. cm. As stated previously, the compression index C_c for a clay with a natural void ratio of 1.2 ranges between 0.3 and 0.6 (Rutledge, 1939). The average immersed weight of the clay is about 0.82 gm. per cu. cm. Assuming the lowest value, 0.3, for C_c we obtain by means of Equation 4 for the relation between the depth and the void ratio the dashed curve C_r in Fig. 2. In contrast to the empirical curve it indicates that the water content should decrease rapidly with depth. For higher values of C_c the difference between the theoretical and the empirical curves would be still more conspicuous.

In order to account for the records described above we are compelled to assume that the average slope of the sedimentation compression curve is extremely small compared to that of the curves obtained from laboratory experiments. Fig. 3 illustrates this state-

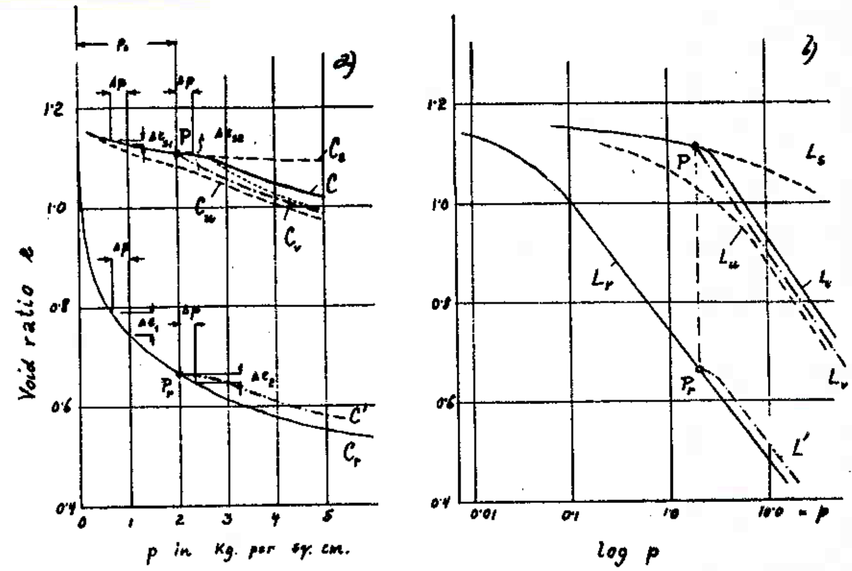

FIGURE 3.

ment. In this figure the curve C_r represents the pressure-void ratio curve for a remolded specimen of a clay having an initial water content, w_o=45 per cent, equal to the liquid limit. Assuming that the water content of the clay at a depth of 3 ft. below the surface is also equal to the liquid limit, the curve C_s is the steepest sedimentation compression curve which appears to be compatible with the aforementioned field records.

The conspicuous difference between the trend of the two curves, C_s and C_r in Fig. 3a becomes comprehensible if we consider the difference between the rate of loading which corresponds to these two curves and the influence of this difference on the compression produced by the load. It is known that every soil particle is surrounded with a very viscous film of adsorbed water the presence of which delays the establishment of a direct contact between the solid soil constituents (Terzaghi, 1925). Before this contact exists the films

TABLE 1: *Rate of deposition and thickness of some Quaternary and Pliocene argillaceous sediments*

	Thickness of deposit m	Rate of deposition m/1000 yr	Reference
DELTAIC			
Mississippi, Holocene	55	120	McClelland (1967)
Rhone, Holocene	65	17	Lagaaij (*in litt.*)
Orinoco, Holocene	40	8	Kidwell & Hunt (1958)
ESTUARINE			
Avonmouth, Holocene	13	2·5	present paper
Tilbury, Holocene	16	2	present paper
Pisa, Holocene	10	2·5	present paper
MARINE, shallow water			
Oslofjord, Holocene	—	0·8	Richards (*in litt.*)
Po Valley, Pleistocene	2000	1·2	present paper
Po Valley, Pliocene	3000	1·0	present paper
Kambara, Pliocene	2600	0·9	present paper
MARINE, deep-sea			
Caribbean, Pleistocene	—	0·03	Rosholt *et al.* (1961)

deposition one order of magnitude less for cores of red clay from the Atlantic and Indian Oceans. It can readily be accepted that such slow accumulations could scarcely give rise to any excess pore water pressures.

With the great thickness of shallow-water marine deposits often encountered in deep borings the problem is more complex. In the Po Valley, for example, Pliocene and Quaternary sediments extend to a depth of 5000 m (Fig. 19) and have been deposited at an average rate of the order 1 m/1000 yr. Now if these sediments were argillaceous throughout, then, on theoretical grounds, high excess pore pressures would be expected; and it is well known that such 'abnormal' pressures do indeed exist in some formations (Tkhostov 1963). Nevertheless pore pressures have been measured in the Pliocene deposits of the Po Valley (see section 4 (E)) and they are not more than about 15 to 20 per cent above hydrostatic values. The explanation of these relatively low excess pressures is to be sought, almost certainly, in the presence of sand and silt beds which permit more rapid consolidation. Under such conditions calculations are exceedingly difficult and the pore pressures in deep formations should be measured in the field if reliable values of effective overburden pressure are required.

2. Recent sediments

In this section of the paper summaries will be given of case records for several argillaceous sediments recently deposited on the sea-bed and in tidal flats.

of adsorbed water act as a lubricant. Therefore the rate at which the pressure on the clay is increased must have an influence on the resistance against intergranular slippage. This conclusion is in accordance with some laboratory tests which have been carried out by Langer (1936) on a stiff plastic clay from the vicinity of Paris, France. Fig. 4 is a semi-logarithmic plot of the test results. It shows that

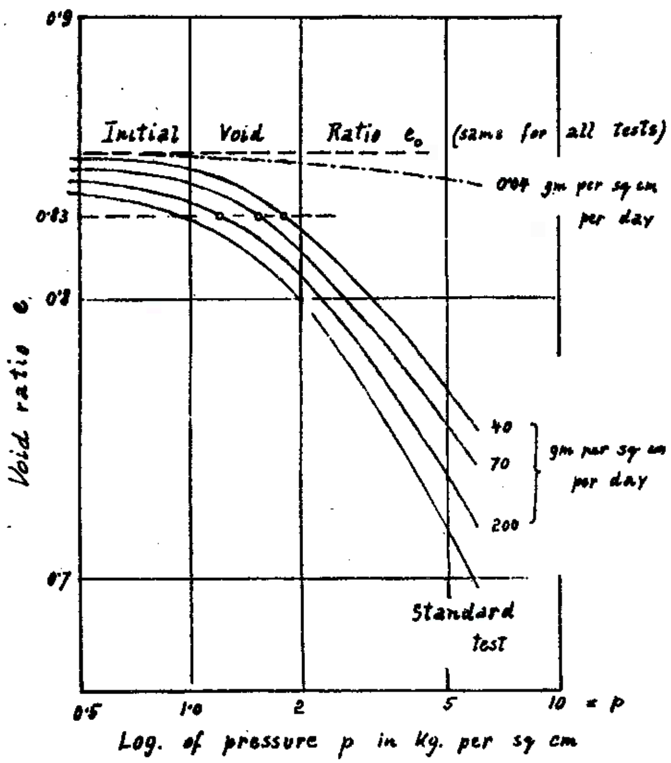

FIGURE 4.

the pressure p required for reducing the void ratio of the clay to a given value, for instance 0.83, increases with decreasing rate of loading. The lowest rate of loading, 40 gm. per sq. cm. a day, is about one ~~ten~~ thousand times as rapid as the rate at which nature increases the pressure on a bed of clay during the process of sedimentation. The dash-dotted curve illustrates the writer's opinion concerning the shape and the trend of the curve which one would obtain if loading the

clay at a rate of 0.04 gm. per sq. cm. a day. This rate corresponds to the rate of loading on a clay during the process of sedimentation in nature. For two other clays which have also been tested, a reduction of the rate of loading from 200 to 40 gm. per sq. cm. a day had no measurable effect on the relation between pressure and void ratio. This seems to indicate that the effect of the rate of loading on the relation between pressure and void ratio is different for different clays. In order to investigate this effect in the laboratory, very much lower rates of loading than the lowest one in Langer's series should be used.

The following investigations will show that the existence of adsorbed films is likely to account not only for the difference between the general trend of the two curves, C_s and C_r in Fig. 3*a*, but also for the other differences between the properties of undisturbed and remolded clays.

Solid and Lubricated States of Clays

Fig. 5 is a graphic representation of the author's conception regarding the interaction between adsorbed layers. (Terzaghi, 1925, 1926.) It represents magnified ideal sections through the vicinity of a point of contact between two soil particles. In the immediate vicinity of the surface of the solid particles the adsorbed water is solid and its density is far above normal. With increasing distance from the surface of this solid film both the density and the viscosity of the water decrease and beyond a certain distance d the properties of the water are normal. The distance d depends both on the chemical properties of the solid and on those of the substances other than water which are present within the zone of adsorption. Thus, for instance, if the water in the voids of a bentonite specimen contain sodium salts in solution, the adsorbed layers are very much thicker than those in one saturated with pure water. The views regarding the nature of the effect of the solid on the water are still in a controversial state. Yet the existence of the layers and the conspicuous deviation of their physical properties from those of normal water have been conclusively demonstrated by numerous and very different methods of investigation.

When two soil particles are pressed together with a force Q the outer zones of their adsorbed layers merge as shown in Fig. 5*a*. The

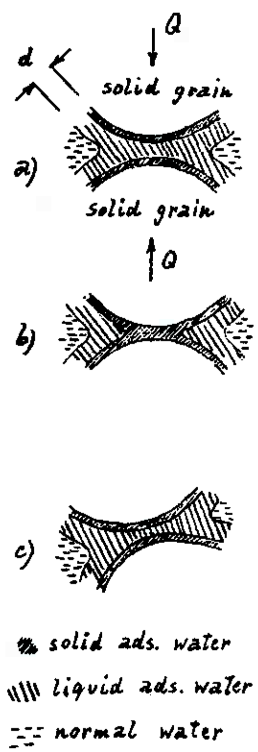

FIGURE 5.

event occurs instantaneously. However, in this state they are still separated from each other by a liquid though very viscous layer of adsorbed water. The further approach between the particles takes place at a rapidly decreasing rate, until finally the solid portions of the adsorbed films come into actual contact.

If p is the intensity of the effective vertical pressure per unit of area of a sample of clay, the average pressure Q per point of contact between two clay particles is equal to p divided by the number of points of contact per unit of area. The value Q represents the statistical average of the real contact pressures. The individual values may be greater or smaller than mean value. Hence at some points of contact the rigid bond will be established much more rapidly than at others. The gradual development of a solid bond between the clay particles will be called the *process of solidification*. Before the clay is com-

pletely solidified, some of the particles are already connected with each other by a solid bond, due to direct contact between the solid parts of the adsorbed layers. Others are held together by the highly viscous, yet liquid portion of the adsorbed layers. Both types of bond participate in the transmission of the pressure from grain to grain. Hence it is necessary to subdivide the effective stress in a clay into two parts. One part, the *solid bond stress*, is entirely carried by the bond between the solid portions of the adsorbed layers. The second part, the *film bond stress*, is carried by the viscous resistance of the adsorbed films. As long as part of an effective stress consists of a film bond stress, the stress produces a slow, viscous intergranular movement. If the entire stresses in a mass of clay are carried by the solid bond the clay is in a *solid state*. This state is preceded by the *lubricated state* in which part of the stresses are carried by the film bond. As the solid state is approached, the *degree of lubrication* of the clay decreases.

The process of solidification involves the lateral displacement of adsorbed water from the zones of potential solid bond between the soil particles towards the intergranular voids. The rate at which solidification at a given load per unit of area proceeds depends undoubtedly on the quantity of highly viscous, adsorbed water which needs to be displaced in order to establish the solid bond and on the pressure per unit of area of the adsorbed films. If we make an approximately plane horizontal section through a loaded layer of sand in such a manner that it does not intersect any grain, practically the entire section will pass through normal water and the pressure produced by the load per unit of area of the section through the adsorbed water will be extremely high. On the other hand, if we make a similar section through a layer of clay which carries the same load per unit of area as the sand, the major part of the section will be located within adsorbed water and the pressure due to the load per unit of area of the section through the adsorbed water will be relatively low. Therefore one should expect, that the time required for a given pressure to establish a given degree of solidification should for a clay be very much longer than for a sand.

The influence of grain size on the rate of solidification may account for the known fact, that a powder consisting of scale-like mineral particles such as mica or hematite flakes cannot be transformed into a plastic substance by mixing it with water, unless the size of the particles is smaller than about 0.002 mm. A coarser powder solidifies during the test, while the finer powder remains in a lubricated state.

The same influence may also account for the well-known statistical relation between grain size and the angle of shearing resistance. In a general way, the smaller the effective grain size the smaller is the angle of shearing resistance determined by means of slow laboratory tests, such as slow shear tests or slow triaxial compression tests. In a slow test the neutral stress in the clay at the instant of failure is practically equal to zero. However, the degree of solidification in a highly colloidal clay is likely to be far less advanced at the instant of failure than in a leaner clay. If this is the case the increase of the shearing resistance due to a given increase of the normal stress on the surface of sliding should decrease with decreasing grain size.

As the state of failure in a shear test on clay is approached, the presence of films of liquid water between the clay particles should cause the clay to flow at constant stress like a viscous liquid. This conclusion has been repeatedly confirmed by experiment. (Terzaghi, 1931; 1932; Hvorslev, 1936).

Consolidation Due to Increasing the Load on a Normally Consolidated, Sedimentary Clay

Thus far our investigation has been of a purely theoretical nature. In order to make the step towards phenomena of practical importance we must analyze the effect of a transition from the slow process of loading applied by nature to the relatively rapid process associated with the construction of a building. In order to visualize the mechanical consequence of this transition we make one real and one imaginary consolidation test. Both tests are made on a remolded clay, the initial water content of which is equal to 45 per cent and equal to the liquid limit. In the real experiment the load is applied in equal increments Δp at the rate of 1 ton per sq. ft. per week and in the imaginary experiment the load is increased at a rate of 1 ton per sq. ft. per 500 years. The two tests will be referred to as rapid and slow tests respectively. Tests by Langer (1936) have shown that a process of loading by increments leads to practically the same pressure-void ratio curve as a continuous process of loading, provided the average rate of load application is the same in both tests. Therefore we are justified in assuming

that the compression curve which corresponds to the slow application of the load, is identical with the sedimentation compression curve C_s in Fig. 3*a*. The curve obtained from the rapid test is represented by the curve C_r.

In the rapid test an increase of the pressure from p to $p+\Delta p$ reduces the void ratio by Δe_1 and in the slow test by Δe_{s1}, which is very much smaller than Δe_1. The probable reason for this difference has already been explained. During the slow test every load increment acts on the clay for a period of many decades. As a consequence, contact is established between the solid portions of the adsorbed layers and the clay assumes the characteristics of a cemented material like concrete. This bond represents a true cohesion. It accounts for the high modulus of elasticity of slowly sedimented clays and for the absence of consolidation under very low excess pressures.

After the load on the specimen represented by the curve C_s in Fig. 3*a* becomes equal to p_o the rate of loading in this test is changed from 1 ton per sq. ft. in 500 years to 1 ton per sq. ft. in one week. When the first increment in excess of p_o is applied the clay cannot possibly know whether this increment will be followed by another one in a day or after 20 years. Hence the decrease Δe_{s2} caused by this increment cannot be greater than the decrease determined by the slope of the curve C_s at point P. The next increment which is added to $p_o+\Delta p$ after a day or two causes a heavier taxation of the strength of the clay. Yet the bond which has been built up during centuries of intimate contact between the clay particles cannot be expected to break down abruptly. Hence the effect of the second load increment will be intermediate between that for a perfectly solidified clay, determined by the slope of the curve C_s and that for a lubricated clay, determined by the slope of C_r. Finally after several load increments have followed each other in relatively rather rapid succession, all the clay particles will be separated from each other by films of liquid adsorbed water whereupon the clay particles are able to slide along each other with the same ease as those in the clay subjected to a rapid test. Hence the pressure-void ratio curve representing the imaginary test should ultimately become at least as steep as the curve C_r. Thus we arrive at the following conclusion. When changing the rate of loading at the state represented by point P in Fig. 3*a* from very slow to rapid the slope of the corresponding void ratio-pressure curve C should consist of two distinctly different parts. The lower one, corresponding to the higher pressure, should have the same general trend as the curve C_r and the upper one should represent a gradual transition from C_s into C as shown in the figure.

In reality the first part of our imaginary experiment indicated by the curve C_s takes place during the deposition of a bed of clay. Point P represents the clay in its natural state. The subsequent rapid process of loading is performed by man, when constructing a building above the bed of clay. In order to get information regarding the decrease of the void ratio of the clay due to the weight of the building we proceed in the following manner. We first remove the clay from its original location whereby we reduce the load on the clay from p_0 to a small value. Then we transfer the specimen into a consolidation device and increase the load on the clay again from zero to p_0. During this process the void ratio of the clay decreases by several per cent. Hence, the void ratio-pressure curve C_u which we obtain while increasing the pressure to values of more than p_0 cannot possibly be identical with the curve C. It must be located at a lower level as shown in Fig. 3*a*. Hence, no matter how careful the sample has been secured and how conscientious the test in the laboratory has been made, the test result does not inform us on the character of the transition between the sedimentation consolidation curve C_s and the surcharge consolidation curve C, nor does it permit any definite conclusion regarding the position of the curve C with reference to the theoretical curve C_v. The plain line in Fig. 3*a* merely represents the result of a guess. The curve may as well occupy the position indicated by the dotted line.

As a final step in our investigation let us assume that a remolded clay has been consolidated rapidly to the state corresponding to P_r on the curve C_r in Fig. 3*a*. Then we leave the clay under the pressure p_0 for a period of say 500 years, whereupon we resume the process of loading at the original rate of 1 ton per sq. ft. per week. During the intermission the contact between the clay particles changes from the state illustrated by Fig. 5*a* into that shown in Fig. 5*b*. Furthermore during the same period the bond between the adsorbed layers is strengthened by molecular adjustments within the zone of contact, known as thixotropic processes. Hence, if we continue our experiment at the original rate of loading, the initial slope of the following section of the pressure-void ratio curve should be as small as that of

the tangent to the curve C_s at point P. The trend of the curve cannot become identical with that of curve C_s until the clay has passed from the solid into the lubricated state. Thus it appears most likely that the pressure-void ratio curve will be similar to C' in Fig. 3*a* and to L' in Fig. 3*b*.

SECONDARY TIME EFFECT

After the solid bond between the clay particles has been destroyed, for instance, by a load in excess of the critical load, the clay particles are temporarily separated from each other by films of very viscous adsorbed water. Hence the clay does not pass again into the solid state until the solid bond between the clay particles, illustrated by Fig. 5*b*, is re-established. In this connection we must consider two possibilities. Either the solidification of the clay occurs simultaneously with the consolidation or else the solidification continues after the consolidation is complete. In the first case the time-settlement curve should be in agreement with the theory of consolidation while in the second case the settlement due to consolidation should be followed by a supplementary settlement which cannot be accounted for by a process of consolidation. Both field and laboratory experience demonstrates that we have to deal almost exclusively with the second case. Figs. 6*a* and *b* show the relationship between per-

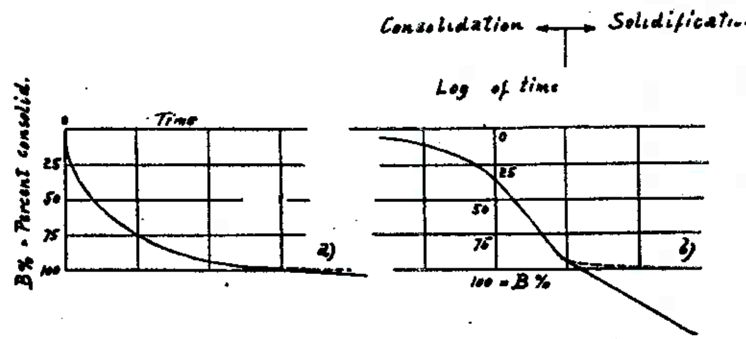

FIGURE 6.

centage consolidation and time for a given load increment which is applied in performing a consolidation test on clay. In Fig. 6*a* the relationship is shown on an arithmetic diagram and in Fig. 6*b* on a semi-logarithmic diagram. In both diagrams the plain curve represents the test results and the dashed curve, that part of the corresponding theoretical time-consolidation curve which does not coincide with the test curve. The difference between the ordinates of the dashed and the plain curves indicates the decrease of the void ratio associated with the process of solidification. As long as consolidation proceeds, part of the load on the clay is carried by an excess hydrostatic pressure in the water and the balance chiefly by film-bond stresses, because the relative movement between the clay particles maintains a state of lubrication. After the excess hydrostatic pressure in the water has become practically equal to zero the film bond is gradually replaced by the grain bond. This is the process of solidification which follows the process of consolidation. During the process of consolidation the effective stresses increase at the expense of the neutral stresses until the neutral stresses become equal to zero. During the period of solidification the grain-bond stresses increase at the expense of the film-bond stresses, until the film-bond stresses disappear. As long as part of the load is carried by film-bond stresses, the decrease of the void ratio of the clay continues. The settlement associated with this decrease is called secondary compression.

Thus far the only attempt to formulate a theory of the secondary compression has been made by Taylor and Merchant (1940). The theory has been worked out for the purpose of predicting the secondary time effect in the field on the basis of data obtained from consolidation tests in the laboratory. The theory is based on assuming that the rate of secondary compression follows a simple law, resembling that for viscous flow or for creep. In reality this law is likely to be very complicated, for two reasons. First of all, the viscosity of the liquid part of the adsorbed layers decreases with increasing distance from the surface of the solid particles. Second, as the rate of slippage at the points of contact decreases the resistance due to the viscosity of the adsorbed films combines with a supplementary resistance due to thixotropic stiffening, resulting from the building up of molecular bonds within the adsorbed layers. Hence, before accepting the theory for practical usage it would be advisable to investigate the importance of the error due to the basic assumption by comparing the computed and the real secondary time effect in several typical cases. This could even be done for existing buildings for which reliable settlement records are available.

Settlement under Small Loads

Whatever the position of the real compression curve C in Fig. 3 with reference to the theoretical compression curve C_v may be, the percentage difference between the amount of compression determined by these two curves rapidly decreases with increasing values of p-p_o. This may account for the satisfactory agreement between the computed and the measured amount of settlement of heavy structures, covering large areas. On the other hand, if the increase of the pressure on a bed of clay due to the weight of a superimposed building is small, the settlement is likely to be considerably smaller than what the computation based on the theoretical curve C_v in Fig. 3 would lead to expect. However, quantitative information regarding this difference could only be obtained by combining laboratory tests with settlement observations on light buildings and the available data do not yet permit any definite conclusions.

With increasing depth beneath the base of a building, the excess pressure produced by the weight of the building on horizontal sections decreases with increasing depth. At a certain depth the excess pressure enters the range within which the slope of the real compression curve C, Fig. 3*a* is insignificant, while that of the theoretical curve C_v is steep. Hence, below this depth, the compression of the clay due to the weight of the building should be very much smaller than the compression computed on the basis of the curve C_v. Since the shape and the extent of the flat part of the curve C in Fig. 3*a* cannot be determined by laboratory tests, information regarding the depth to which consolidation proceeds can only be obtained by means of settlement observations on underground reference points. In this connection the following observation is of interest. At the author's request, three underground reference points were established at depth of 15, 45 and 100 feet below the level of the pile points of the Charity Hospital in New Orleans. The subsoil consists of a succession of strata of sand and of normally consolidated clay which has been explored by test borings to a depth of 150 feet below the pile points. The natural water content of the clay did not decrease with depth and according to the customary method of computing the settlement due to the consolidation of the beds of clay all the underground reference points should have moved down. Yet, the lowest reference point did not move at all and the intermediate reference point descended much less than it should have, according to theory.

At the present state of our knowledge, the estimate of the depth limit to the compression of clay due to the weight of superimposed buildings can only be based on judgment, which may be very misleading.

In many cases the construction of a building is preceded by the excavation for subbasements. The amount by which this operation reduces the pressure on the beds of clay located beneath the site decreases with increasing vertical distance between the clay and the bottom of the excavation. Fig. 7 illustrates the author's conception

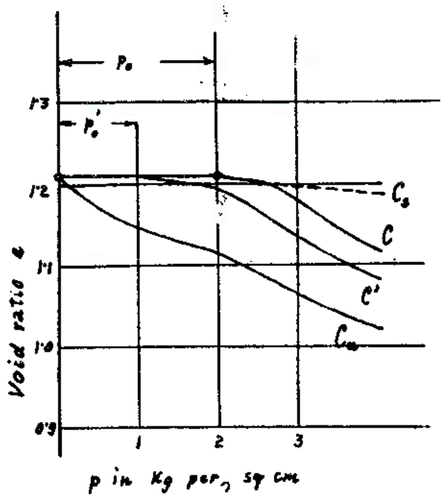

FIGURE 7.

of the effect of a temporary reduction of the pressure on the compressibility of a bed of clay. In this figure the curve C_u represents the results of a consolidation test on an undisturbed sample. Curve C shows the relation between a steadily increasing pressure such as the pressure produced by the construction of a building without subbasements and the corresponding void ratio. The curve C' represents the same relation on the assumption that the application of the pressure has been preceded by a temporary reduction of the pressure to a value $p'_o < p_o$. Since neither of the curves C and C' can be obtained by means of laboratory tests, the conception expressed by these curves can only be confirmed or modified on the basis of field experience.

Conclusions

The preceding analysis has led to the following tentative conclusions:

(1) If the state of stress in a clay has remained constant at least for several decades, or if the state of stress changes at a very low rate, the effective pressure on the clay is transmitted from grain to grain exclusively by a bond between the solid part of the adsorbed films which surround the clay particles. As long as this solid bond is intact the clay is in a solid state. In this state it has the physical characteristics of a porous, cemented material such as a typical loess. The modulus of elasticity is high and its volume compressibility is smaller than that of a dense sand.

(2) When a bed of clay in a solid state is loaded beyond a certain limit, for instance by constructing a heavy building above the clay, the rigid bond between the clay particles breaks and the particles become temporarily separated from each other by liquid films of adsorbed water. This is the lubricated state of the clay. In this state the clay is highly compressible. The increase of the load on a clay in a lubricated state is first of all followed by a period of consolidation the length of which depends both on the physical properties of the clay and on the thickness of the bed of clay. During this period the building located above the clay undergoes settlement due to consolidation. Yet, after the major part of the excess water has drained out of the clay, the clay is still in a lubricated state. Therefore the sliding of the clay particles along each other continues until finally the solid parts of the adsorbed layers come into contact with each other, whereupon the clay has again attained a solid state. The corresponding settlement of the structure is called settlement due to secondary compression. The relation between the time and the secondary compression per unit of depth depends on the grain size, on the chemical composition of the clay and of the adsorbed layers and on the length of the period of consolidation which preceded the period of secondary compression. However, in contrast to the rate of consolidation it is independent of the thickness of the bed of clay. The transition of a clay from the lubricated into the solid state may require years, decades or centuries, depending on the nature of the clay.

(3) After the consolidation of a clay is practically complete all the stresses in the clay are effective stresses. Yet these stresses

are partly carried by the highly viscous liquid films of adsorbed water which surround the clay particles and partly by a direct bond between the solid portions of the adsorbed layers. Therefore it is necessary to subdivide the effective stresses in the clay into two parts each having different characteristics: the film bond and the grain bond stress. A constant film bond stress is associated with a viscous flow which occurs at a constant rate while a constant grain bond stress does not produce any movement.

(4) During the process of consolidation the stresses in the clay consist mainly of neutral and of film bond stresses, while the grain bond stresses are negligible. During the subsequent period of solidification the neutral stresses are negligible while the grain bond stresses steadily increase at the expense of the film bond stresses. Finally the film bond stresses disappear, whereupon the clay is again in a solid state.

(5) The rate of settlement due to consolidation can be estimated by means of the theory of consolidation. This theory is based on the simplifying assumption that the entire effective stresses represent grain bond stresses. An attempt to establish a theory of the secondary compression associated with the process of solidification has been made by Taylor and Merchant. The data required for evaluating the error associated with this theory are not yet available.

(6) Prior to the beginning of construction operations practically every bed of clay in the field is in a solid state unless the structure of the clay has been disturbed by recent soil movements or by artificial changes in its state of stress. As the load on a bed of clay due to an artificial surcharge increases, the clay gradually passes from the solid into the lubricated state, whereby the permanent compression produced by a given increase of the pressure first increases and then decreases.

(7) During the sampling operations every clay passes from the solid into a partially lubricated state. Hence, information regarding the physical properties of clays in a solid state can only be obtained by means of field observations, for instance by measuring the settlement of structures the weight of which increases the pressure on underlying beds of clay by not more than a fraction of the unconfined compressive strength of the clay, or by observing the movement of underground reference points beneath the foundation of heavy buildings and by comparing the results of the observations with those of the computed settlements. Up to this time the available data do not yet justify expressing a definite opinion regarding the relation between pressure and compression within the range of pressure required for transferring the clay from the solid into the lubricated state.

Acknowledgment

The author wishes to express his gratitude to Professor A. Casagrande for careful perusal of the manuscript and for valuable suggestions.

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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.

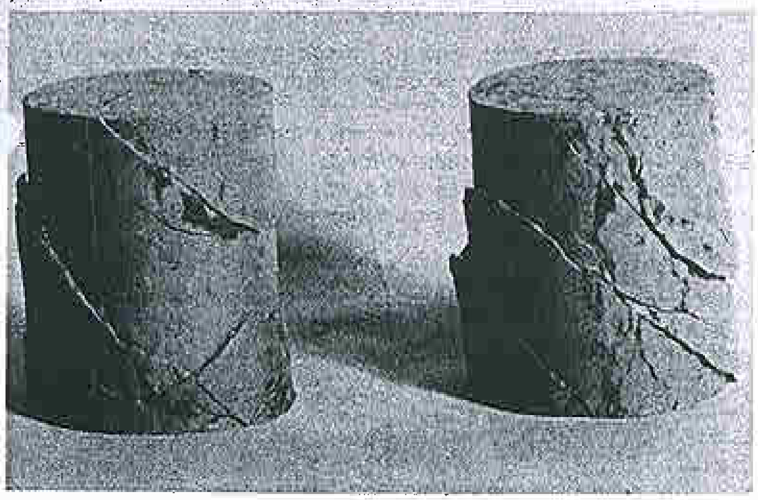
The reason of this observation was experimentally found to be due in a large part to the way the loading was increased in the compression apparatus.

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:

$P = 25^T$

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>. 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.

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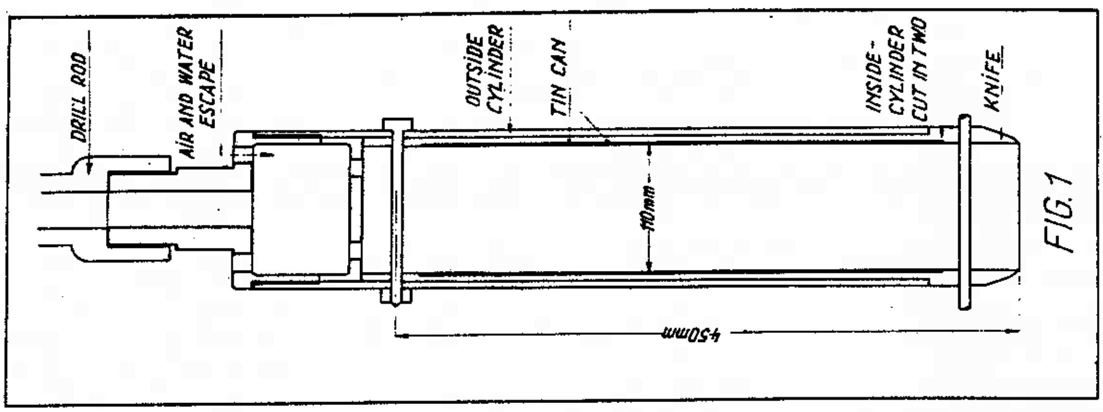


FIG. 1

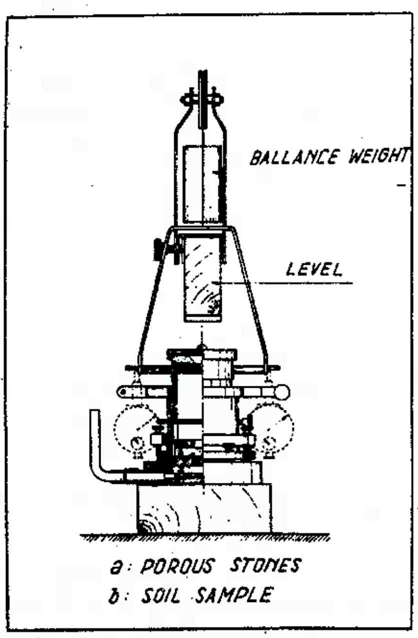


FIG. 2

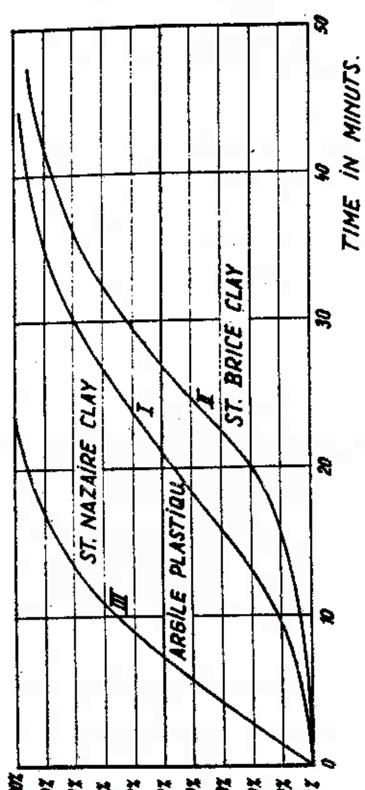
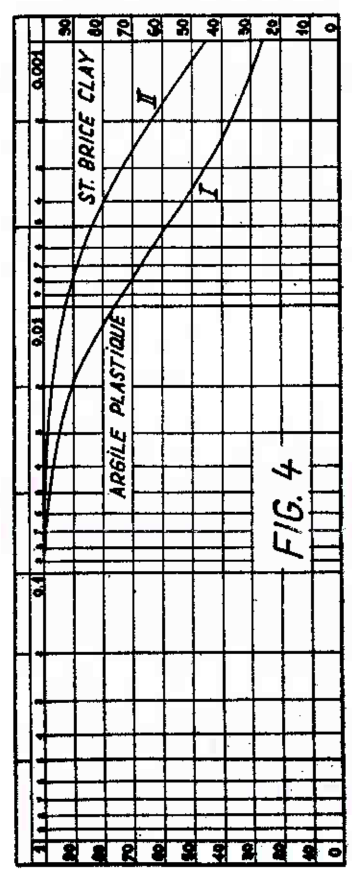


FIG. 3



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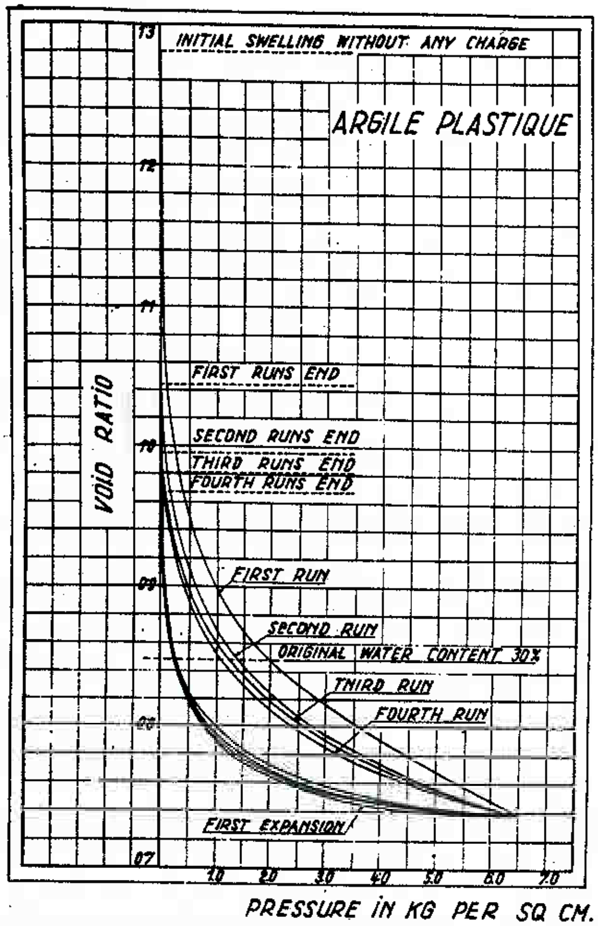

FIG. 5

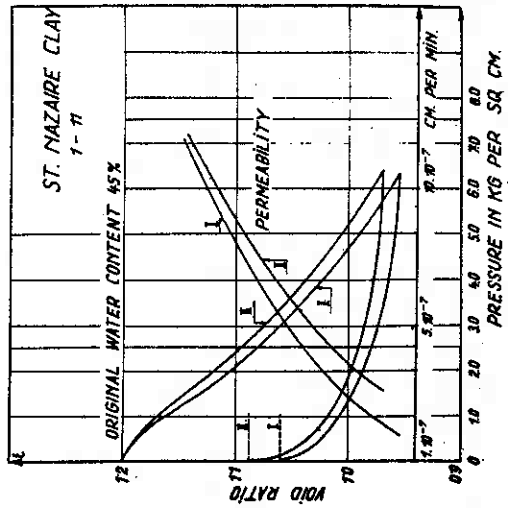

FIG. 9

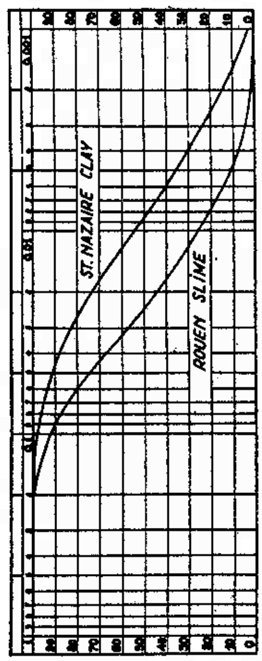

FIG. 8

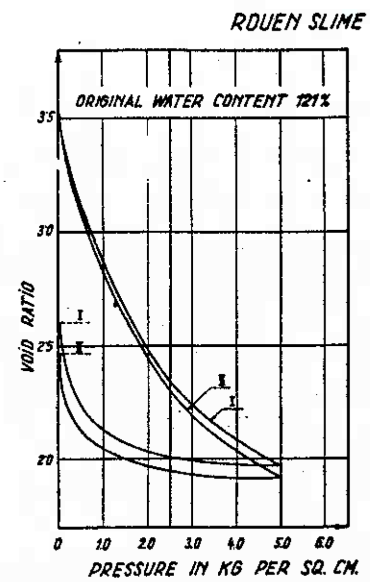

FIG. 10

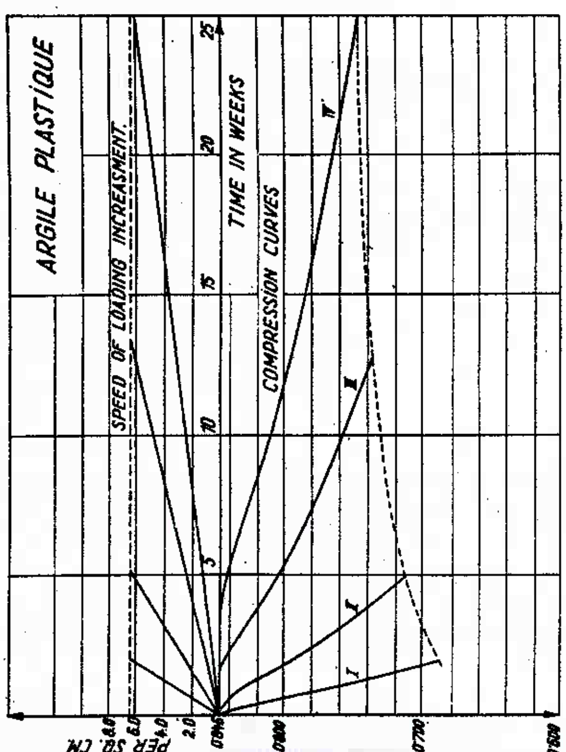

FIG. 6

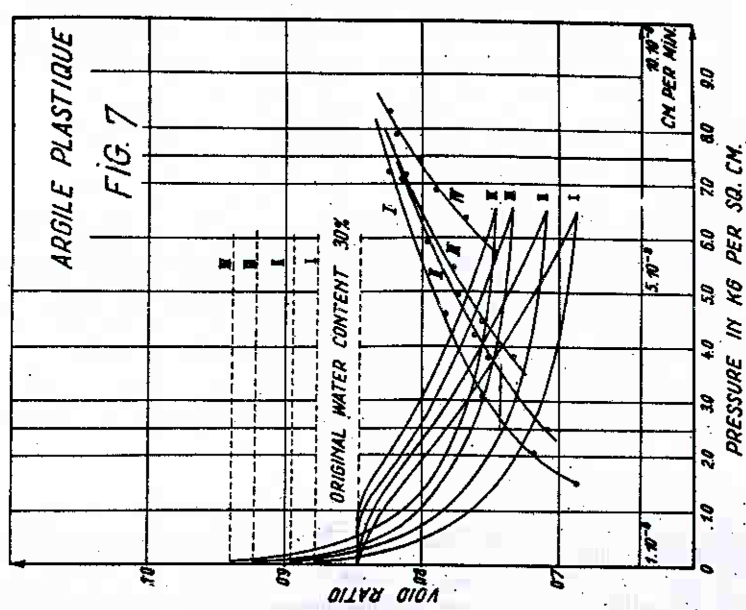

FIG. 7