Laboratory Investigation, Including Compaction Tests, Improvement of Soil Properties

Recherches de laboratoires, y compris essais de compaction, amélioration des propriétés des sols

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Oral Discussion / Discussion orale:

Mr. *W. G. Shockley* (presented by Mr. *W. J. Turnbull*), U.S.A.
Dr. *M. J. H. Hvorslev*, U.S.A.
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Written Discussion / Discussion par écrit:

Dr. *C. L. Dhawan*, India
Dr. *H. Peterman*, Germany
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General Reporter Session 2 – Rapporteur général Session 2

The General Reporter

As an introduction to this morning's discussion, I should like to summarise the conclusions of my general report first. The papers belonging to this session could be divided roughly into 5 groups:

(1) The behaviour of soil at increasing stresses;
(2) Electro-osmotic, electro-chemical and physico-chemical phenomena;
(3) Physical properties and phenomena;
(4) Dynamic properties;
(5) Improvement of laboratory techniques.

All 28 papers were reviewed and comments were given on the contents, which can now be used as a basis for discussion.

However, before entering examining these specific points in detail, I feel that, after informal talks with colleagues on the subject of shear strength, I should explain my attitude towards the most current method of studying the effects of shear stresses on soil behaviour. Like Dr. *Casagrande*, and independently of him, I arrived at the conclusion that, in this most important field of soil mechanics, we still lack a proper definition of the relation between the testing technique and the application of test results to design.

In order to make myself quite clear, I wish to give an example taken from my practical experience, which may serve as a standard. I recorded this case, in one of my papers for the Second International Conference in Rotterdam 1948.

The lateral pressure of a 50-ft. soil layer against a row of closely spaced bored piles had been computed by the designer, on the basis of usual shear strength data, obtained from quick tests. Though a safety factor was taken into consideration, the piles failed *two years* after the structure had been completed.

When this case had been recorded, it seemed that the bridge abutments built on point-bearing piles through thick layers of clay and peat had been subjected to slow lateral movements for quite a long time.

Apart from this, this case taught me one most important fact. Like every soil mechanics engineer, up to that time I had been quite content to apply the ultimate shear stress, divided by a factor of safety, more or less arbitrarily chosen, but rather conventional. I became aware of the fact that cohesive saturated soils exhibit the peculiar property of resisting strongly the application of sudden deformations and yield slowly at low values of the shear stress.

From that time on, I started to develop laboratory techniques on the strength of what is now commonly known as rheological concepts. My first attempt consisted in a number of very slow tests, in an ordinary shear box. However imperfect these tests were, considering the inhomogeneous state of stress and the gradual changes in the boundary condition during the test, the results were rather encouraging, as the phenomenon of pile failure by lateral pressure could be explained quantitatively, from the point of view of bending strength of the piles as well as from the *effect of time* on the deformation shear-stress relations of the cohesive soil. These tests were also put on record in the Conference of Rotterdam 1948.

Soil mechanics is closely related to many other sciences, as for instance physico-chemistry, and it is only when certain difficulties arise that we realize the independence of these sciences. In this case, I got to know the outstanding work performed in the field of rheology by my colleague from the Delft Technical University, Dr. *Burgers*. The principles of rheology, as a matter of fact, proved to be an excellent basis for the study of the flow effects of cohesive soils.

At the London Conference on the Shear Strength of Soils, in 1950, an attempt was made, by Mr. *Tan Tjong Kie* and myself, to compare results obtained in the Dutch cell-test and the standard triaxial test on a pottery clay. Though comparative results were not conclusive, they certainly showed us the merits and deficiencies of both methods.

Now, shortly before our present Conference, the Second International Congress of Rheology was held in Oxford. A paper "On the Mechanical Behaviour of Clays" was presented by Mr. *Tan Tjong Kie* and myself, on test results obtained by applying torsional forces to thin-walled clay tubes, which allowed us to measure time shear relationships at different shear stresses. As you know, the torsion of a hollow tube represents a case where hydrostatic applied stresses are absent at small deformations. This is the reason why it was chosen as the theoretically best method of studying the effects of shear stresses on deformation characteristics of saturated cohesive soils.

The following points are considered representative of the most important features of the test results:

(1) The clay exhibited a first yield value f_1, below which the deformation was so slight that it could not be detected in the very sensitive apparatus used. This threshold value was found to be greater than 54 g/cm² at 47.5% water content (*L.L.*

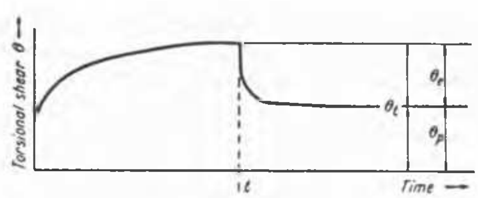

Fig. 1 Torsion Shear vs. Time
Résistance à la torsion en fonction du temps

93.5%, *P.L.* 27.4%). This value can also be measured in a triaxial apparatus, if certain precautions are taken.

(2) The amount of recoverable deformation can be measured in the period following upon the removal of the shear stress (Fig. 1). With a loading time of more than 5 hours, the recovery part of the deformation θ_e obtains a nearly constant value; with a loading time of less than 5 hours, this magnitude increases. It attains 85% of θ_t with loading times of 1 or 2 minutes (Fig. 2).

(3) A third point of interest is the behaviour at small stresses in long duration tests. Then θ_e is comparatively large, and it

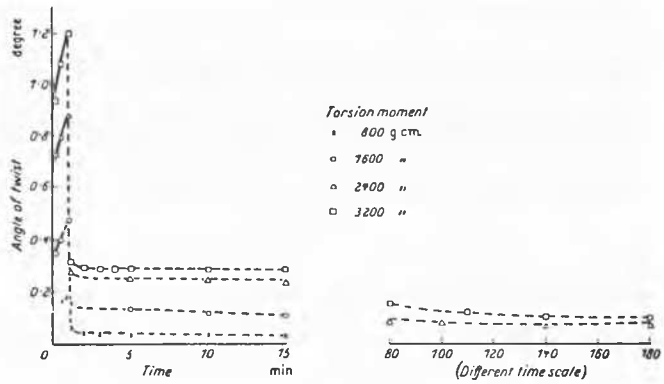

Fig. 2 Deformation vs. Time
Déformation en fonction du temps

might even be suggested that, at a second threshold value f_2, slightly over f_1, complete recovery might also be obtained in the majority of cases (Fig. 3).

(4) The viscosity characteristics of the material are given in the next diagram (Fig. 4), showing the test results. The ordinates are expressed in terms of the rate of total shear $d\gamma/dt$, as the amount of elastic recovery was found to become constant, after 5 hours' loading, so that

$$D = \frac{d\bar{\gamma}}{dt} = \frac{d\gamma}{dt}.$$

This flow diagram demonstrates all the interesting points. At point A a threshold value f_1 is found, below which there is no deformation. This value may become extremely important in foundation engineering, as it represents the ultimate value of shear stress for zero rate of deformation. The AB section of the straight line gives the rate of deformation as a linear *Bingham* viscosity relation in terms of

$$(\tau - f_1) = \eta \frac{d\gamma}{dt}$$

where η = *Bingham* viscosity. Beyond the point *B*, structural disintegration occurs, resulting in so-called shear rate thinning.

The linear part allows us to estimate the increase of the deformation by flow following upon the instantaneous deformation at the application of a shear stress. In the case of the example given, a flow shear deformation γ = 25% would be obtained at point *B*, at the ultimate value of the shear stress f_3 = 0.27 kg/cm², one year after the application of the stress.

However, it must be considered that our remoulded material does not show any favourable effect of structural viscosity, nor of prestress, due to preconsolidation effects.

I welcome this opportunity to recall *Casagrande*'s and *Wilson*'s important results, obtained in long-time loading tests, which showed that failure may occur even after months. The

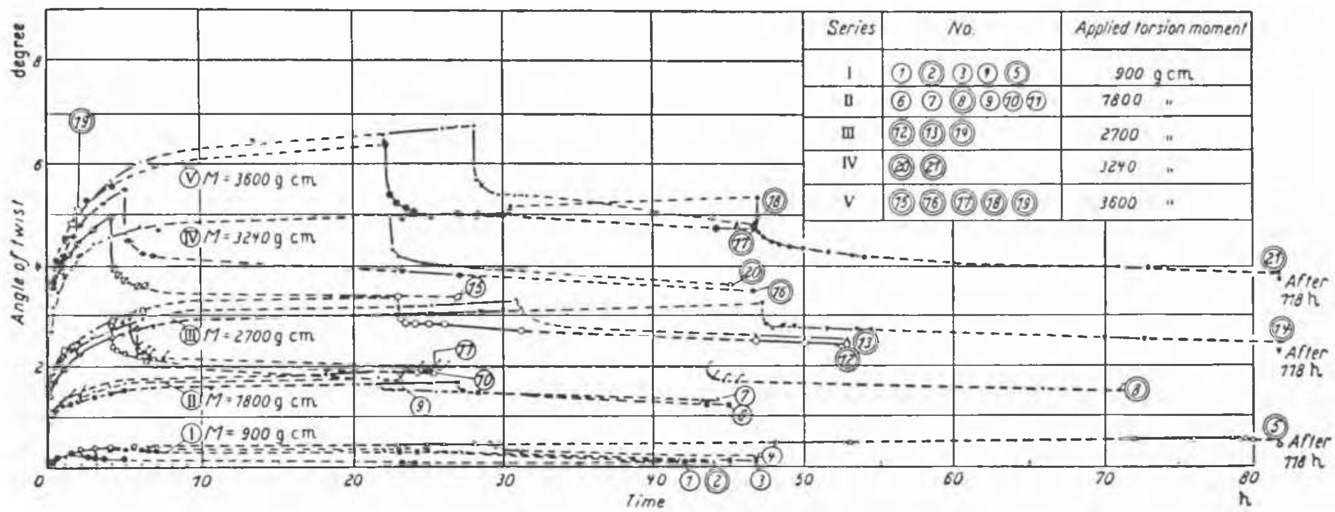

Fig. 3 Diagram Showing Shear vs. Time
Diagramme donnant la résistance au cisaillement en fonction du temps

results of *Haefeli* and *Schaerer*, also obtained in triaxial compression tests, leading to the more simple relation

$$\tau = \mu \frac{d\gamma}{dt}$$

are similar to our test results. In quoting Dr. *Haefeli*'s results, I have another strong reason for deploring his very regrettable absence to-day.

Mr. Chairman, before terminating this part of my introduction, which I have had to lengthen with a review of my latest results, owing to the fact that a paper on this subject could not be prepared in time, for reasons well-known to my fellow General Reporters, I would ask your permission to proceed for another few minutes, in order to establish the points of our discussion.

The results of the tests which I have just described are far from being complete. They have, however, given definite proof of the existence of a possibility to improve our definition concerning the permissible shear stress in cohesive soil masses.

If I have criticised the triaxial test in my General Report, it is for the now apparent reason that the state of failure in this test does represent a state of disintegration and, consequently, we need not concern ourselves with the magnitude of a shear stress leading to this state. On top of this, the arbitrarily chosen rate of deformation, usual in this test, may greatly influence the ultimate value of the shear stress.

On the other hand, I am perfectly aware of the fact that thin-walled clay tubes cannot be properly made from undisturbed samples, without introducing an element of disturbance. The triaxial technique, however, could easily be adapted to serve my purpose, so as to procure the necessary data for the admissible shear stress:

(1) by giving the threshold value;
(2) by giving the $D - \tau$ relation;
(3) by giving the transition to the state of disintegration.

I certainly realise that the compression of a cylinder represents an involved state of stress distribution, even for a material with linear behaviour at small deformations. The changes in this state of stress on deformation will increase the possibilities of a transfer of stresses from the liquid part to the solid part. Therefore I suggest that this point would be a theme suitable for our first discussion.

I also suggested that the effects of subsequent deformations should be studied in order to estimate their influence on the flow properties if only one specimen is used; but first of all I would like to have your opinion on my definition of the permissible shear stress, as based on the elastic part of the deformation and the effects of time on deformation, and which may serve to introduce a new way of linking up the necessary elements of the design of earth structures with the results of laboratory investigations.

To my mind, Dr. *Casagrande* has made two excellent suggestions. First, the setting up of a Committee for Classifications of subjects dealt with in the various sessions. Our General Secretary will remember our correspondence exchanged on this subject long before this Conference! Dr. *Casagrande*'s second suggestion concerns the topic of shear strength which should be covered in a special discussion. Needless to say, I strongly favour this suggestion, but I would even go further and suggest that a Committee whose activity should embrace the periods between Conferences should be elected.

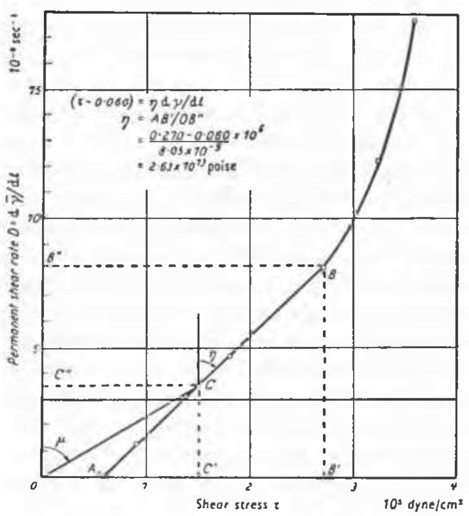

Fig. 4 Viscosity Characteristics
Caractéristiques de la viscosité

Le Rapporteur général démontre avec exemples à l'appui l'influence exercée par le facteur temps sur la résistance au cisaillement et il est d'avis que les phénomènes de fluage des sols cohérents pourraient être avantageusement étudiés sous l'angle de la rhéologie. Des recherches en cette matière sont actuellement en cours aux Pays-Bas. Ces travaux ont permis l'élaboration d'une théorie nouvelle de la résistance au cisaillement qui montre que, au delà d'une certaine limite des contraintes et des déformations, un matériau se comporte comme un liquide visqueux, indépendamment du temps. Une déformation de la structure, semblable à celle obtenue dans l'appareillage de cisaillement triaxial, ne se produit que sous des contraintes considérables.

Mr. W. G. Shockley[1]) (presented by Mr. W. J. Turnbull)

Messrs. *A. W. Bishop* and *G. Eldin* (Proceedings 1953, vol. I, p. 100) have presented an interesting paper giving the results of an extensive series of triaxial tests on sands. In 1950, the Waterways Experiment Station conducted triaxial tests on a uniform sand (uniformity coefficient 1.4 and effective size $D_{10} = 0.23$ mm) in which careful measurements were made of the volume change of the specimens at various degrees of axial strain. The results of these tests were disturbing, to say the least, and tend to cast doubt on the validity of the triaxial test on sands as applied to behaviour of the field prototype.

The tests were conducted on dry sand specimens 2.8 inches in diameter at constant lateral pressure (by applying partial vacuum). Careful techniques were developed to ensure placement of the specimens at initial densities ranging from dense to very loose. The rubber membrane was carefully marked with horizontal circumferential lines spaced about one centimeter apart vertically. At various degrees of axial strain under load the diameters of the test specimen at each line were measured as well as the vertical distance between the horizontal lines. This permitted determination of the volume contained between any two horizontal lines and the change in volume with increasing strain.

The results of the large number of tests conducted in this manner showed that, regardless of the initial density of the specimen, the volume at the two ends decreased continually (density increased) with increasing axial strain, whereas the middle portion of the specimen increased in volume (density decreased). For example, a loose specimen prepared at an initial density of 91 lbs./cu.ft. showed the following densities (from top to bottom of specimen) at an axial strain of 10%: 96, 93, 88 and 96 lbs./cu.ft.

The results of these determinations obviously are valid only if the sand contained between any two lines on the membrane remains between those lines during the test. This assumption was checked by freezing specimens at various percentages of axial strain, cutting them horizontally into segments, and measuring the density of each by displacement methods. The freezing tests gave a good qualitative check on the density variations determined by direct measurement of the specimens.

These tests strongly indicate that the net volume change measured in triaxial tests on a sand is not indicative of the changes which occur in the failure zone of the specimen and may not be indicative of the changes which occur in a sand mass in nature. More tests are needed to determine conclusively the volume changes occurring during triaxial tests on sands. It is hoped that other investigators will have an opportunity to explore this problem and assist in reaching a final answer.

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L'auteur se réfère aux essais triaxiaux exécutés en 1950 par le Waterways Experiment Station sur des sables à grains uniformes, dont les résultats soulevèrent des doutes sur la validité des essais triaxiaux sur les sables. La question se pose de savoir si de tels essais de résistance au cisaillement représentent encore exactement les phénomènes correspondants dans la nature.

The General Reporter

If you will allow me to say some words on the discussion presented by Mr. *Turnbull* on behalf of Mr. *Shockley*, I would like to say that we have observed the same phenomena in critical density tests on sand in our laboratory. It may serve the suggestion made in point 4 of my General Report. To my mind, it proves the fact that the state of stress in the compression type of apparatus is not homogeneous and therefore we understand it is definitely feasible, why these differences in the distribution of density were found by Mr. *Shockley*. It may also be observed that these tests showed that in fact those differences are more pronounced, I would say, in dense sands than in loose sands. I have noted this point as point 4 of my proposals for discussion as I consider it an important one.

Le Rapporteur général souligne qu'il a fait des observations semblables à celles mentionnées par M. *Shockley*.

Dr. M. J. Hvorslev

Some papers and discussions of the first and second sessions refer to a hypothesis for the shearing strength of soils proposed by the writer in 1936. Since that time I have not had any opportunity to conduct further experiments or detailed investigations of this problem, and I appreciate the efforts of several institutions and individuals who have planned and executed their experiments in such a manner that the results may be used for investigating the validity of or for amplifying the hypothesis. I refer particularly to the investigations performed in the Federal Institute of Technology in Zurich and at Imperial College in London, and to the results published in papers by Drs. *Haefeli*, *Bjerrum*, *Skempton*, *Gibson*, and *Bishop*.

In both theoretical and practical investigations of the stability of slopes and the bearing capacity of foundations it is necessary to use rather simple expression for the shearing resistance of soils; i.e., the *Coulomb* equations with internal friction and cohesion acting alone or combined. However, neither the angle of internal friction nor the cohesion are true physical constants, but they vary with the stress conditions and stress history of the soil and depend also on other factors. More detailed and complicated expressions for the shearing resistance are required in order to determine, for given conditions, the proper values of the coefficients in the *Coulomb* equation, to delimit the range of validity of these values, and to explain the causes of deviations from the simplified rules.

It is probable that the true angle of internal friction for fine-grained soils varies but very little with the stress conditions, and such variations were neglected in the writer's hypothesis. The cohesion is caused by intrinsic forces, which probably are created by overlapping of spheres of partially bound water surrounding the clay minerals and are a function of both current and preceding stress conditions. It was assumed that the magnitude of these forces and thereby of the cohesion primarily is a function of the current void ratio, or the water content in case the soil is fully saturated. By replacing the void ratio with the corresponding pressure in the virgin pressure-void ratio diagram, called the equivalent pressure p_e, it was demonstrated experimentally that the cohesion, e, can be expressed approximately by a simple linear equation, $c = \varkappa p_e$, or by an exponential function of the void ratio. It was also shown that not only the cohesion but also the total shearing resistance of a normally consolidated clay can be represented by a function of the void ratio or water content. The relationship between cohesion and void ratio can be determined without use of the equivalent pressure, but introduction of the latter is a mathematical expedient which facilitates visualization of the relationships and particularly the evaluation and averaging of test results with a normal amount of scatter.

Objections have been raised to the use of the equivalent pressure because the position of the virgin pressure-void ratio curve depends to some extent on the testing conditions, on the degree of disturbance of sample, and for remolded and reconsolidated soils also on the intial water content. It is true that the value of the coefficient $\varkappa$ depends on the position of the virgin consolidation curve, but the product $\varkappa p_e = c$ is independent thereof when the slope of the curve in a semi-logarithmic plot and the corresponding compression index are substantially correct. For remolded soils this slope varies but very little with the testing conditions and the initial water content, and the same values of the cohesion and internal friction are obtained whether the test results are evaluated by use of the equivalent pressure or by comparison of the shearing resistances of test specimens with the same void ratio at failure but different normal pressures on the plane of failure. In case of soils in natural condition, the slope of the virgin consolidation curve depends to some extent on the degree of disturbance of the sample, but so does the shearing resistance, and the consolidation curve can probably be used to determine the relationship between void ratio and cohesion for this particular degree of disturbance. It is probable that disturbance primarily causes a decrease of cohesion and that a decrease of the effective angle of internal friction is very small unless slickensided surfaces of failure are formed.

The writer's original hypothesis and method of evaluating the test results were based on the results of tests with two types of remolded clays, and they constitute a simplification of actual conditions and relationships, as also emphasized in the original paper. The hypothesis applies primarily to saturated or nearly saturated soils with water contents between those corresponding to the liquid and shrinkage limits. The influence of other factors must be considered in further and more detailed investigations, particularly of undisturbed soils. Some of these factors are discussed in two interesting papers presented at Sessions I and II of this conference.

Mr. *Jakobson* (Proceedings 1953, vol. I, p. 35) demonstrates that inundated clay deposits, even though never subjected to external overloads or drying, may have an appreciable shearing resistance at the surface where the external effective stresses are zero, and this shearing resistance is defined as origin cohesion. The data presented indicate and the author states that the upper soil layers have a void ratio which is smaller than that corresponding to the external effective stresses, and it is suggested that the superficial soil has been consolidated under action of internal pressures or intrinsic forces. The data presented by Mr. *Jakobson* are a distinct contribution to the knowledge of sources of shearing resistance of undisturbed soils, but the writer doubts that the described conditions also exist in test specimens of remolded and recently reconsolidated soils.

It appears to the writer that the decrease in void ratio of the surficial soil layers is comparable to that caused by ordinary overconsolidation and that at least a part of the origin cohesion, as defined by Mr. *Jakobson*, can be accounted for by this decrease in void ratio and the relationship between void ratio and cohesion. It is possible that a part of the cohesion of an undisturbed soil, which for a long period has been subjected to slow chemical changes and/or secondary consolidation, may be independent of the void ratio, and this part should then be entered as a soil constant in the general expression for the shearing resistance, in addition to the frictional resistance and the part of the cohesion which varies with the void ratio. It is difficult to estimate, from the data available, how large a part of the superficial shearing resistance corresponds to the actual void ratio and which part, if any, is independent of the void ratio and the external effective stresses. Consolidation tests on samples from the surface and various depths and determination of the preconsolidation pressures would provide data of interest. When evaluating Mr. *Jakobson*'s diagrams in Figs. 2 to 4, it should be taken into consideration that the properties of the surface layers differ to some extent from those of the soil below, as indicated by the liquid limits, and extrapolation of data from the underlying and more uniform strata may be influenced by disturbance of the soil during advance of the vane, The influence of a hypothetical increase in degree of disturbance with increasing depth is indicated in Fig. 5.

Dr. *Gibson* suggests (Proceedings 1953, vol. I, p. 126) that the energy represented by volume changes or movements of the surface of the test specimen be taken into consideration; that is, a part of the total shearing resistance is represented by the energy consumed or supplied by volume changes and should be considered separately from cohesion and internal friction. The writer fully agrees with Dr. *Gibson*'s thesis but would suggest that, when such refinements are introduced, additional sources of supply and expenditure of energy also be considered. A part of the energy represented by consolidation and the corresponding downward movement of the surface of the test specimen and the external load is consumed in forcing water out of the soil and in producing internal strains. On the other hand, a part of the energy required for expansion of a strongly

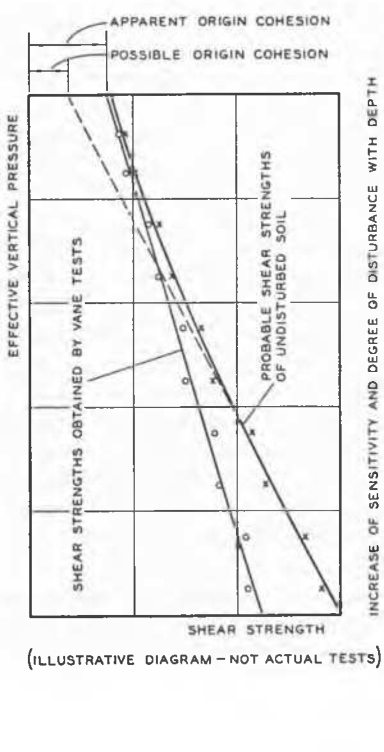

Fig. 5 Influence of Partial Disturbance of Soil During Vane Tests
Influence du remaniement partiel du sol au cours d'essais au moyen de l'appareil à palettes

overconsolidated soil during a shear test may possibly be supplied by internal energy, in the form of structural deformations, produced and stored up during the overconsolidation and released by partial disturbance during the test. When these factors are considered, it is probable that the corrections will be smaller than indicated in the paper by Dr. *Gibson*.

It is to be hoped that basic research on the shearing resistance of soils, particularly of undisturbed soils, will be continued and receive adequate encouragement and support of university and government agencies. It is time-consuming work which requires utmost patience and attention to detail on the part of the investigators, and there are many difficulties ahead in separating the influence of minor variables and of inherent sources of error in testing procedures, as also indicated in the excellent general report by Professor *Geuze*.

Foremost among the above-mentioned difficulties is the nonuniform distribution of stresses and deformations in test specimens, the change of this distribution during the test, and the consequent internal migration of pore water and irregular changes in void ratios and water contents, also in case of constant volume tests. Efforts to develop direct shear equipment with a more uniform distribution of stresses in the test specimen and to investigate this distribution theoretically, as in the excellent paper by Mr. *Roscoe* (Proceedings 1953, vol. I, p. 186), are to be highly commended. Results of recent test at the Waterways Experiment Station show that the nonuniformity of deformations and volume changes in triaxial test specimens is much greater than indicated by theoretical considerations, and further investigations of this problem are badly needed. Because of these nonuniformities, void ratios and water contents should be determined for various parts of a test specimen and particularly for material from the zone of failure. In many cases it is extremely difficult and even impossible to evaluate properly the results of tests for which only the average void ratios or water contents of the entire test specimen have been determined.

Most natural or undisturbed soils are more or less stratified, and the writer demonstrated in his original investigations that even in remoulded, reconsolidated, cohesive soils the clay particles have a preferred orientation perpendicular to the direction of the major principal stress during consolidation, and that this orientation influences both the strength of the soil and the inclination of the planes of failure. *Casagrande* and *Carillo* have developed a theory for determination of the influence of stratification or anisotropy on the strength characteristics of soils, but this problem has not yet been investigated experimentally in sufficient detail. The influence of anisotropy may be neglected in many practical tests, but it should be considered in other cases and assuredly in basic research.

Time or available space does not permit a discussion of many other problems in the research on shearing resistance of soils, but in closing the writer would like to mention that all basic research on the physical properties of undisturbed soils would be greatly facilitated if easily accessible and sufficiently uniform deposits of the principle types of soils could be located, and if undisturbed samples from them be made available to those engaged in the research. It appears to the writer that the location of such deposits should be given priority in any extensive programme of basic research, and it may well be made the object of a cooperative effort of national and even international committees and organizations.

Il découle de recherches théoriques et pratiques que l'angle de frottement interne étant que la cohésion dépend de l'historique des sols et du type des contraintes, et, par conséquent, ne sont pas des constantes physiques. Pour les sols cohérents à granulométrie fine, la cohésion peut être considérée approximativement comme une fonction linéaire d'une pression de consolidation équivalant au volume des pores.

L'auteur présente une explication alternative des résultats d'essais obtenus par M. *Jakobson* (Comptes Rendus 1953, vol. I, p. 35), qui arrive à la conclusion que la cohésion se subdivise en deux parties: une partie constante (cohésion d'origine) et une partie dépendant de l'état actuel du sol. L'emploi de l'appareil à palettes pour ces essais ne semble pas indiqué.

Plus loin l'auteur est d'avis que si une amélioration doit être apportée à l'hypothèse de travail pour l'interprétation du cisaillement, comme suggéré par M. *Gibson* (Comptes Rendus 1953, vol. I, p. 126), cette hypothèse devrait également prendre en considération des autres facteurs affectant l'énergie. Il espère que les recherches de base sur le cisaillement apporteront des éclaircissements sur les questions se rattachant à ce domaine.

L'auteur commente ensuite le mémoire de M. *Roscoe* (Comptes Rendus 1953, vol. I, p. 186). Les essais exécutés par le Waterways Experiment Station montrent que la non-uniformité des déformations et les variations de volume trouvées dans les essais triaxiaux dépassent considérablement les valeurs correspondantes trouvées par des recherches théoriques. L'auteur conclut qu'il faudrait déterminer le volume des vides et la teneur en eau après la rupture en plusieurs points de l'échantillon pour être en mesure de caractériser ces dernières avec précision.

Dr. J. Kerisel

Puisque Messieurs les Rapporteurs généraux, aussi bien M. *Casagrande*, que M. *Geuze* souhaitent que la discussion soit ouverte sur la force de cisaillement, je voudrais – et je m'excuse de cette coincidence avec l'intervention de M. *Hvorslev* – parler du rapport de M. *B. Jakobson* (Comptes Rendus 1953, vol. I, p. 8). M. *Jakobson* a fait des essais de cisaillement sur des argiles qui n'ont jamais été soumises à des précontraintes de compression, constamment sous l'eau, et qui n'ont pas été soumises à des efforts de succion de la part d'une végétation quelconque. Ce sont des argiles à haute limite de liquidité, avec une teneur en eau supérieure à la limite de liquidité, et ces facteurs vont en décroissant avec la profondeur. Il a reporté les cisaillements, mesurés avec l'appareil à palettes, en fonction du total de la pression que causent les grains solides, déjaugés dans l'eau, et il a trouvé une ordonnée à l'origine qu'il appelle la cohésion d'origine. Il trouve que cette cohésion d'origine est de l'ordre de 50 à 60 g/cm². J'aimerais, pour ma part, apporter une observation semblable à celle de notre collègue suédois et dire que pour certaines vases africaines, notamment les vases de Tunis, j'ai pu faire une observation semblable, observation qui est la synthèse de nombreux essais de laboratoire. Si l'on reporte de la même façon les cohésions en ordonnées, et en abscisses les profondeurs multipliées par les densités déjaugées, on trouve encore que la courbe ne passe pas par l'origine et qu'il y a une cohésion que j'appellerai avec notre collègue suédois une cohésion d'origine qui est aussi de l'ordre de 50 à 60 g/cm². Ces observations montreraient la necessité d'apporter un tempérament à ce concept qui était proposé par le Rapporteur général, M. *A. Casagrande*, hier, a savoir le concept du cisaillement considérée comme une force résiduelle, selon lequel, il n'y a résistance au cisaillement, que s'il y a eu une précompression, exercée soit par les surcharges, soit par le poids des grains, cette force au cisaillements disparaissant après quelques déformations. Je pense, quant à moi, qu'il est d'autant plus nécessaire d'apporter ce tempérament au concept: cisaillement force résiduelle, après l'exposé très brillant de M. *Geuze*, ce matin, qui montre que dans l'étude des déformations par cisaillements il faut considérer une première phase qu'il appelle «la phase élastique» assez restreinte qui est suivie par une deuxième phase visqueuse. Et je pense aussi que nous ne devons pas ignorer les recherches extrêmement précises qui sont développées par les chercheurs qui s'intéressent à la physico-chimie des argiles. Dans la constitution des grains des argiles, ils ont mis en relief avec une précision accrue l'existence des feuillets de 7, 10 et 14 Ångströms, qui existent entre les architectures moléculaires des atomes de SiO_2 et Al_2O_3 en structures octoédriques ou hexaédriques. Je pense que c'est dans la constitution des grains qu'il faut chercher l'explication de cette cohésion d'origine et de cette phase élastique à déformations fonction de temps, exposée par M. *Geuze*, et en particulier dans les feuillets ou se logent les intervalles entre cations échangeables, et ions OH. N'est-ce pas l'occasion de remarquer à ce propos que c'est à partir de plans parallèles laissant entre eux un liquide visqueux, que l'on a définit dans les traités de physique l'unité de viscosité. Je souhaiterais pour ma part que les essais qui ont été rapportés par notre collègue suédois et par M. le Rapporteur général soient développés de façon à rechercher s'il n'y a pas une relation d'une part entre cette cohésion d'origine, les épaisseurs des feuillets et la nature des cations.

The author refers to the article by Mr. *Jakobson* (Proceedings 1953, vol. I, p. 35) and mentions similar observations made by himself in Tunis. He draws attention to the fact that the theory on the application of rheology formulated by Mr. *Geuze* in order to throw light on the properties of shear strength will not lead to a solution of the problem if soil structure and cristallography are not sufficiently taken into consideration in their relationship to the physical and chemical phenomena.

Le Rapporteur général

Je félicite Dr. *Kerisel* de son exposé. Ce qui m'a beaucoup intéressé dans son exposé, c'est la notion de l'origine de la cohésion qui serait due aux forces moléculaires de l'eau entre les feuilles des minéraux d'argile. Nous avons exécuté des études similaires à Delft, en prenant, d'une part, les résultats des études physico-chimiques donnant la relation de l'énergie de l'absorption de l'eau aux grains, et, de l'autre, le comportement de ces argiles du point de vue des relations entre les forces mécaniques et les déformations. Nous rencontrons de grandes difficultés dans ce domaine. Ces relations, du point de vue physico-chimique, sont données en termes d'énergie, d'énergie d'absorption. C'est un sujet qui sera discuté plus tard par M. *Salas*. Nous nous trouvons donc devant la difficulté suivante: d'une part la teneur en eau qui est liée aux grains, peut être déterminée par des méthodes d'absorption et, d'autre part, les grandeurs qui nous intéressent, c'est-à-dire la déformation et les forces, sont données en termes de force; jusqu'à présent, il est très difficile de relier ces deux domaines, et peut-être serons-nous obligés de transformer la base de nos études de l'énergie et de transformer les termes de force en termes d'énergie. J'admets sans réserve l'importance de ce sujet d'études, et désire seulement attirer l'attention sur ce que M. *Kerisel* a dit, peut-être par hasard, c'est-à-dire que la grandeur de la cohésion serait de l'ordre de grandeur de 60 g/cm². C'est exactement ce que j'ai trouvé dans les études que j'ai mentionnées ce matin. Je pense que ce fait est attribuable au hasard, car l'on peut bien s'imaginer que beaucoup d'argiles ont été soumises à des contraintes plus grandes que celles de l'argile de M. *Kerisel* ou que celles des expériences du laboratoire de Delft. On peut s'imaginer que, pour les densités plus grandes, les efforts de cisaillement, les forces moléculaires agissant entre les grains des argiles minérales seront aussi plus grandes. C'est pourquoi je crois que la méthode que j'ai montrée ce matin est une méthode qui pourrait donner les résultats que demande M. *Kerisel.* D'autre part, il nous faudra poursuivre nos études sur l'origine de cette cohésion. Pour moi, je considère comme une hypothèse l'assertion que la façon d'appliquer les forces de cisaillement des différents types d'argile nous permet d'en déduire des conclusions importantes sur les relations entre les forces et les déformations, la grandeur de la déformation élastique, et sur les effets du temps sur les déformations.

The General Reporter draws attention to the fact that Mr. *Kerisel*'s investigations on origin cohesion lead to results similar to his own results.

Mr. Jimenez Salas

I want to present a short remark, in support of the recommendation of the General Reporter, contained in the third proposal for discussion. As we have indicated already (Proceedings 1953, vol. I, p. 192), it is very improbable that the particles of clay kneaded with water, or another polar liquid, will become in actual contact between them. As a matter of fact, even the films of adsorbed water will get into contact one with the other only under very significant external pressures. Therefore, speaking about internal friction in a clay that has its pores filled with water or other polar liquid, is only a fiction. This fiction is sometimes useful, but has been proved confusing and has led very often to misunderstandings. Consequently, I will state my opinion on the convenience of selecting more appropriate coefficients for the definition of the physical properties of the clay, as p.e. those suggested in the proposal of the General Reporter.

I want also to note the relationship, in this field, between the works of Mr. *Habib*, to which Mr. *Mayer* has referred, and ours. The coincidence is a check, and it is therefore a satisfaction for my co-worker, Mr. *Serratosa*, and for me. I am only surprised at the constancy of the dry density of a clay kneaded with different liquids, as found by Mr. *Habib*. We found different results, as it is shown in Fig. 10 of our paper (Proceedings 1953, vol. I, p. 197). Perhaps the explanation of this difference is that we experimented with a far more colloidal clay than Mr. *Habib*'s, and therefore had to deal with more exaggerated and as a result more easily measurable phenomena. We have also the impression that our explanation of the mechanism of the influence exerted by the nature of the pore liquids being an explanation that takes into account the role of the polar momentum of the liquid, is very comprehensive and therefore, we hope that it will lead to more farsighted developments in the future. There is also an inter-relationship between our points of view and Mr. *Jakobson*'s origin cohesion which Mr. *Cassel* has mentioned. This origin cohesion is a pure thixotropical phenomenon which we have found also in the flocculated silty clays of Huelva, a harbour in South Spain. These silty clays are coagulated by the action of the acid waters that come from the pyrite treatment plants of the mines of Riotinto. We have found an origin cohesion of 80 g/cm².

We want finally to present our most sincere thanks to the General Reporter for summarizing our paper, which he did with great completeness and exactitude. But, nevertheless, as our work has been considered as "highly speculative", we may point out, once more, that soil mechanics has grown from an art into a technical subject, by the use of the knowledge, previously acquired by its elder sisters: hydraulics and pure mechanics. On the basis of this former experience, we think that in the future soil mechanics will use in a practical and realistic way the knowledge that other connected sciences, like physico-chemistry and electro-chemistry, are reaping every day, as, by instance, through the works of Mr. *Overbeek*, of Utrecht, on the double layer.

Les recherches entreprises par l'auteur démontrent que le terme de frottement interne ne saurait être appliqué aux argiles, ainsi qu'on le fait pour caractériser la force de cisaillement, tout particulièrement lorsque les pores sont emplis d'eau. Dans ces recherches sur les argiles espagnoles l'auteur – de même que M. *Jakobson* – a observé une cohésion constante (cohésion d'origine). Il attribue ce phénomène aux qualités thixotropiques des sols.

The General Reporter

I should like to assure Mr. *Salas* that I am extremely interested in his approach to the importance of the physico-chemical phenomena, especially in connection with the role played by pore water in relation to the properties of minerals. I think his approach is justified by the results he has obtained. I have already alluded to a matter which I consider worth discussion; I think that we should draw out a survey including the most important elements of inter-action between grains and water. If I have made some criticisms on Mr. *Salas*' important contribution, it is because in our test we have found that the simple *Freundlich* relation cannot be wholly maintained for a number of clays, especially when the electrolyte contents of the pore water come into the picture. I think that Mr. *Salas* will agree with me that if the forces acting between the water and the grains are somewhat influenced by what is commonly called a double-layer effect due to the ionisation of the water, they may also have a considerable influence on the initial resistance due to shearing strength of the water bound to the grains, and I think that he has realized that this is exactly what we are looking for. We are trying to establish a connection between the properties of the bound water, in order to explain any cohesion which may exist in soils. I think that the present concept of the viscous fluid is the most suitable one for our purposes. By "viscous fluid" I understand grains and fluid together as the results obtained on the entire mass of soil. I think that the role played by the fluid between the mineral grains follows fundamentally the same laws as mixtures of water and grains, and I think this would justify our approach to the determining of the magnitudes and the nature of the cohesion. Again I have been struck by the fact that the cohesion as found by Mr. *Salas* which, if I understood rightly, is of about 80 g/cm^2, is of the same order of magnitude as so many other clays, and it may be that the limited range of those magnitudes as mentioned by M. *Kerisel*, by Mr. *Salas* and myself, enables us to arrive at a certain conclusion concerning the composition of the soil and the distance between the particles and the nature of the pore water. I thank again Mr. *Salas* for his most important contribution to our knowledge about these phenomena.

Le Rapporteur général souligne l'importance des influences chimico-physiques sur le cisaillement, notamment en relation avec l'eau interstitielle.

Mr. S. Steuerman

Three papers in this Session (Proceedings 1953, vol. I, pp. 118, 152 and 406) deal with the causes and extent of energy dissipation in granular soil masses subjected to vibrations, a topic which has been proposed by the General Reporter.

These three papers discuss only vertical dynamic forces and neglect completely the horizontal ones. Horizontal dynamic forces are able to produce forces and movements in each grain which are reversible in direction whereas vertical dynamic forces produce in each grain, in all practical cases except earthquake, forces and movements directed always downwards. Horizontal vibrations first loosen up the grain structure permitting sieving of finer grains which are pressed downwards by their own weight, that of the super-imposed layers and of the foundation, thus compacting the sand. Vertical dynamic forces wedge the grains one between each other by a tamping action. Dissipation of energy is materially different in these two cases. These two vibration components are seldom completely separated, but one of them is predominant.

The temporary loosening up of the sand structure is partially the cause of the "liquid" behaviour of sand described by *T. Mogami* and *K. Kubo* (Proceedings 1953, vol. I, p. 152).

For energy dissipation in granular soils still another physical factor is important, the water content of the soil. Assuming that the sand-water mixture is free of air, it can be stated that every movement of sand grains must be accompanied by a corresponding motion of the adjacent water. Movement of the water relative to the grains changes the static friction between the grains to a much smaller hydrodynamic one. This facilitates grain movement as can be seen from the test results given by *W. Eastwood* (Proceedings 1953, vol. I, p. 118).

An interpretation of this informative paper would be further enhanced if, in addition to the volume weight (108.1 lbs./cu.ft.) of the tested sand, its relative density, grain size curve and moisture content were given. Phenomena observed by Prof. *Eastwood* on water-saturated sand—lower damping effect, behaviour "more like a solid bed of rock" (p. 121)—are indicative of high relative density of the tested sand.

It would be interesting to hear from Prof. *Lorenz*, whether his formulas (Proceedings 1953, vol. I, p. 406) are appliable for a two-phase system and for horizontal vibrations. Footings subjected to horizontal vibrations, so-called "rocking" foundations, are the most troublesome ones, although their vertical forces may be relatively small.

Dissipation of energy is an important factor for future design of foundations; the direct problem of the design engineer is still to predict settlements of a given foundation on a given soil. Another way to pose the problem would be: under consideration of the expected energy dissipation in different layers of substrata, to specify for a given foundation what densities of these strata are required to eliminate the settlement of the foundation or reduce it to a tolerable amount.

Construction site observations in U.S.A. indicate that 70–80% relative density precludes foundation settlements, even under heavy vibratory loads, but these are only first, by no means final, conclusions.

L'auteur souligne les propriétés des vibrations horizontales, prenant particulièrement en considération les matériaux sableux. La teneur en eau joue également un rôle déterminant dans le compactage par vibration.

The General Reporter

I think we have to thank Mr. *Steuermann* especially for his remarks on what I consider to be the fundamental problem in the effect of vibrations in Foundation engineering, as the combined influence of lateral and vertical vibrations should be taken into account in any problem concerning vibrations and as the distribution of vibrations and movements in soil is a three-dimensional problem. This would indicate in principle the best way of dealing with these questions. A second point is that I also agree with the importance of the behaviour of saturated sand, as is especially pointed out by Messrs. *Mogami* and *Kubo*. What we have in mind is to study the behaviour of saturated sand at different vibrations and thereby study especially the effect of the initial density, along the same lines as my discussion on the effect of static forces on saturated soil this morning. I think that would procure us the data which we need, in order to estimate the danger of excessive settlements, which are caused by a decrease of internal friction of the saturated soil mass. I think there we have an approach which might produce practical results.

Le Rapporteur général remercie M. *Steuerman* et fait ressortir que les points sur lesquels a porté la contribution de ce dernier occupent une place de premier rang parmi les thèmes de la Session.

Dr. C. L. Dhawan[1])

While assessing the suitability of soils for different engineering works, the following soil constants are usually determined:

(i) Mechanical analysis;
(ii) Compaction characteristics;
(iii) Consistency limits;
(iv) Shrinkage limit;
(v) Shear characteristics.

In order to classify soils on the basis of a mechanical analysis, further work is necessary to differentiate between the real size limits for clay, silt and sand as far as the engineering properties of soils are concerned and to correlate them with the other soil characteristics.

Atterberg and his co-workers were among the first to study the changes in the cohesive properties of soils as a function of moisture. Regional empirical equations should be investigated in connection with the different size particles and, finally, soils should be classified and compared on an International basis.

A syringe with holes $\frac{1}{8}''$ in diameter has been designed in India for the determination of the plastic limit.

Regarding compaction characteristics, we must be clear as to the purpose, i.e. hydraulic or highway, for which the soils are to be used. Curves showing density, compaction moisture and the compactive effort needed to attain a particular density should be drawn first for the guidance of the field workers.

An apparatus for the determination of shrinkage factor of soils has been investigated and a paper has been published in the Journal of Scientific and Industrial Research, India, Vol. 11B, No. 21 by Dr. *Dhawan*.

For shear characteristics we must determined the relationship between the direct shear and the triaxial shear. For Punjab soils (India) we have developed an empirical equation.

Angle of internal friction:

$$\varphi = \frac{C}{7} + \frac{F.Si}{5} + \frac{C.Si + F.Sa}{3} + \frac{C.Sa}{2.5}.$$

where C	=	Clay%	(particles below .002 mm)
F.Si	=	Fine Silt%	(between .002 mm and .01 mm)
C.Si	=	Coarse Silt%	(between .01 mm and .02 mm)
F.Sa	=	Fine Sand%	(between .02 mm and .2 mm)
C.Sa	=	Coarse Sand%	(greater than .2 mm but less than 2.0 m)

L'auteur passe en revue les différentes méthodes de classification des sols.

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Dr. C. L. Dhawan[1])

The rate of water adsorption varies considerably with the nature of the colloid, its silica/sequioxide ratio and with the nature of the adsorbed cation. Sodium soils produce the maximum swelling pressures and the lateritic soils swell the least, irrespective of the nature of the adsorbed ion.

A number of clay stone samples from the area near Bhakra Dam were examined for their swelling pressure. This ranged from 0.3 tons/sq.ft. to about 1.4 tons/sq.ft. It was very interesting to find that the swelling pressure of clays was mostly below 1.5 tons/sq.ft. These results were in agreement with those obtained with Burma soils and described in a paper "Movement in the Desiccated Alkaline soils of Burma" by *F. L. D. Woolterton* published in the Proceedings of American Society of Civil Engineers 1950, vol. 76. *L. F. Cooling* in his paper "Some Foundation Problems in Great Britain" presented to the Building Research Congress 1951, stated that the swelling pressure of a particular clay was of the order of 1 ton/sq.ft.

The close agreement of the results obtained in India and those of other workers lead us to the conclusion that the swelling pressure of all clays are a more or less constant value. These results require further confirmation by further tests carried out on different types of soils saturated with different bases after compacting at varying densities and moisture contents. It is well known that the greater the density, the higher is the swelling pressure. It would be interesting to know the observations made in this direction by workers in other countries.

L'auteur indique l'ordre de grandeur des pressions de gonflement enregistrées sur des échantillons de pierre argileuse provenant de la région du barrage de Bhakra (Indes). Les résultats concordent avec ceux obtenus dans d'autres pays et l'auteur souhaite que les ingénieurs étrangers fassent connaître le résultat de leurs observations.

Dr. H. Petermann

This paper is concerned with the results of shearing tests on medium sand recently carried out in a single shear apparatus. In those experiments the internal strain (i.e. the shearing displacement and the variation of density of the specimen) was specially examined. The results are founded on the observations made on three points on the stress-strain curve (Figs. 6, 7).

Reference is not made here to the true strength nor to the ratio shearing stress/normal stress.

Fig. 8 shows, in relationship to one another, the values of shearing displacement and density measured during the shearing movement and the sliding phase. The curve connecting the test points is intended to show that these points represent moments of the same test; it does not show any direct connexion during the test. Loosening or densifying in shearing correspond to the initial density and to the normal stress.

In Fig. 9 the values of shearing displacement s_1 and s_2 are roughly averaged at different normal stresses. With increasing normal stress in the shear plane, the ratio required (shearing displacement/strength) is more favourable.

In Figs. 10 and 11 we can see changes in density during the process of shearing to reaching the shearing or sliding strengths compared to the density.

A uniform process of flow would take place if the points for

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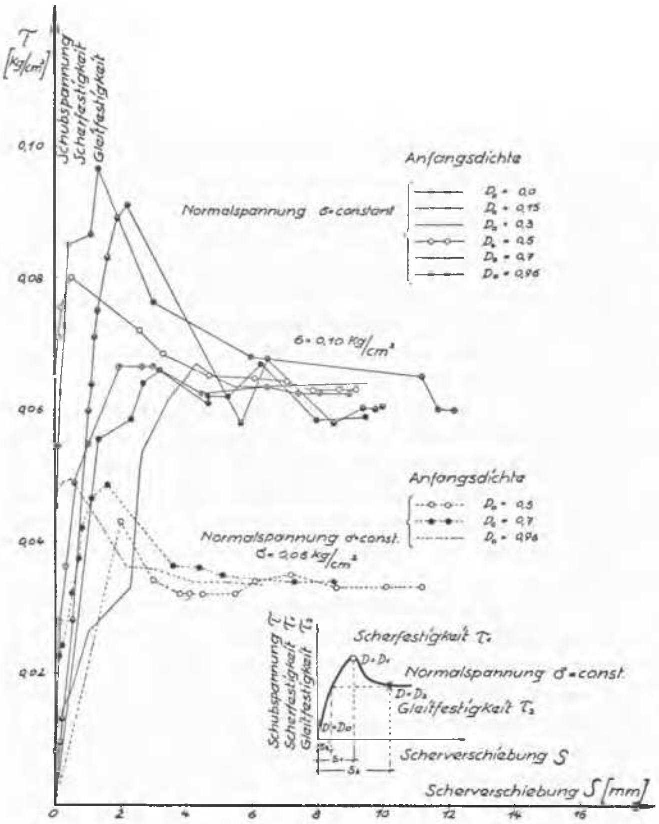

Fig. 6 Shearing Displacement vs. Shearing Stress, Shearing Strength and Sliding Strength
Déplacement au cisaillement en fonction de la contrainte de cisaillement, de la résistance au cisaillement et de la résistance au glissement

Legend for Figs. 6–11
Légende des Figs. 6–11

Normalspannung	= Normal Stress	contrainte normale
Schubspannung	= Shearing Stress	contrainte au cisaillement
Scherfestigkeit	= Shearing Strength	résistance au cisaillement
Gleitfestigkeit	= Sliding Strength	résistance au glissement
Scherverschiebung	= Shearing Displacement	déplacement au cisaillement
Verschiebung	= Displacement	déplacement
Dichte	= Density	densité
Anfangsdichte	= Initial Density	Densité initiale
Bruchzustand	= Moment of Shearing	moment de cisaillement
Gleitzustand	= Phase of Sliding	phase de glissement
Auflockerung	= Loosening	expansion
Verdichtung	= Densifying	compression

D_1 and D_2 on the test curve for equal initial density corresponding to the normal stress were along the diagonal.

In practice we always ought to make clear whether, with the existing earth stresses and the changes brought about by external forces, a loosening or a densifying of the soil structure takes place until the shear strength that has been foreseen has developed. We must also ascertain whether an extension is generally possible or whether the shearing zone is artificially limited. The latter would effect an increase in the shearing strength and a decrease of the shearing displacement required for the ultimate strength values.

In many practical cases the full strength developed in the soil by corresponding displacement cannot be fully utilized because the building can only stand displacements of lower magnitude or because it undergoes only small displacements when the external forces acting on it do not cause considerable soil displacements.

The tests described above lead to qualitative conclusions only. Interpretation of further similar tests is needed in which grain size, thickness of the shearing zone, density, shearing

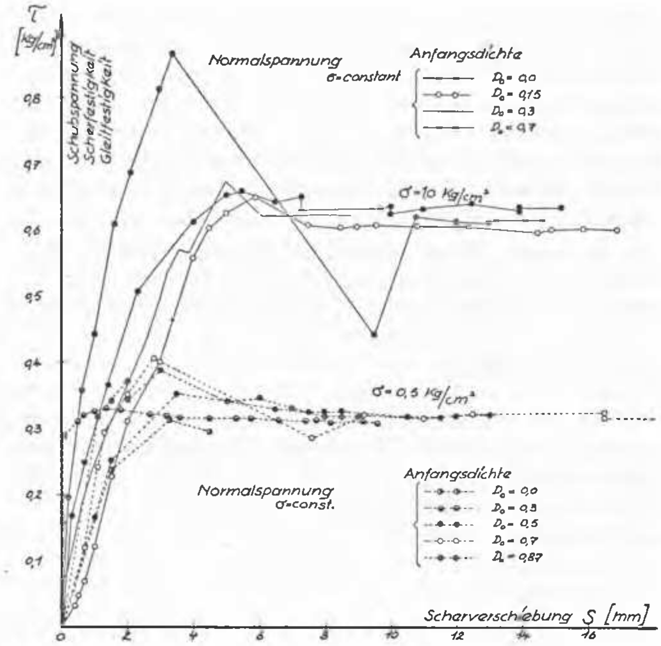

Fig. 7 Shearing Displacement vs. Shearing Stress, Shearing Strength and Sliding Strength
Déplacement au cisaillement en fonction de la contrainte de cisaillement, de la résistance au cisaillement et de la résistance au glissement

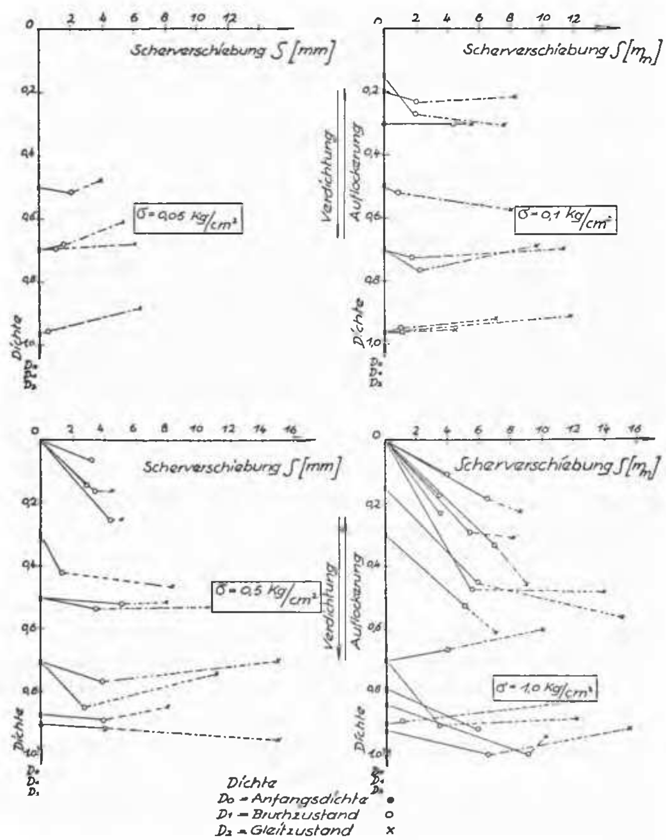

Fig. 8 Shearing Displacement vs. Density
Déplacement par cisaillement en fonction de la densité

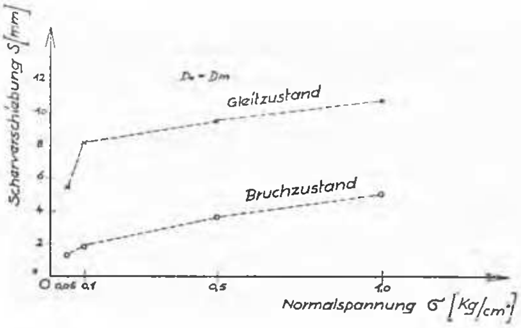

Fig. 9 Shearing Displacement vs. Normal Stress
Déplacement par cisaillement en fonction de la contrainte normale

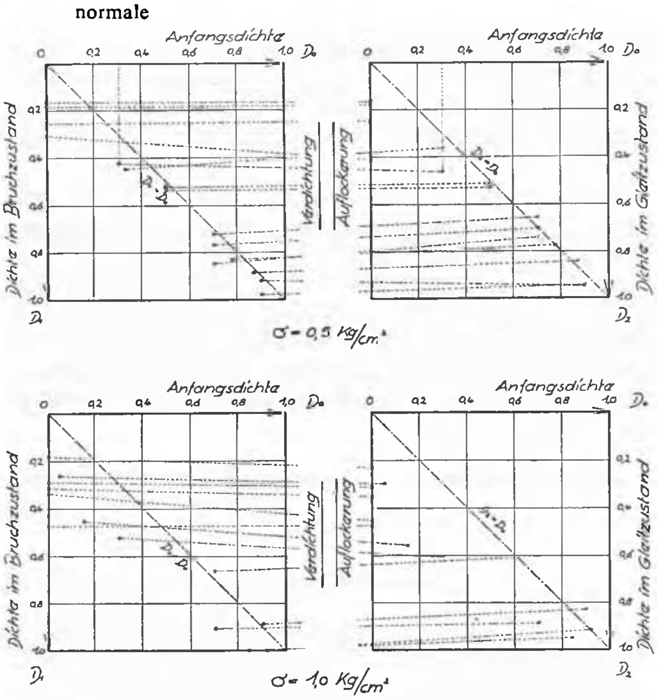

Fig. 10 Relationship Between Density and Density when Sliding
Relation entre la densité et la densité au cours du glissement

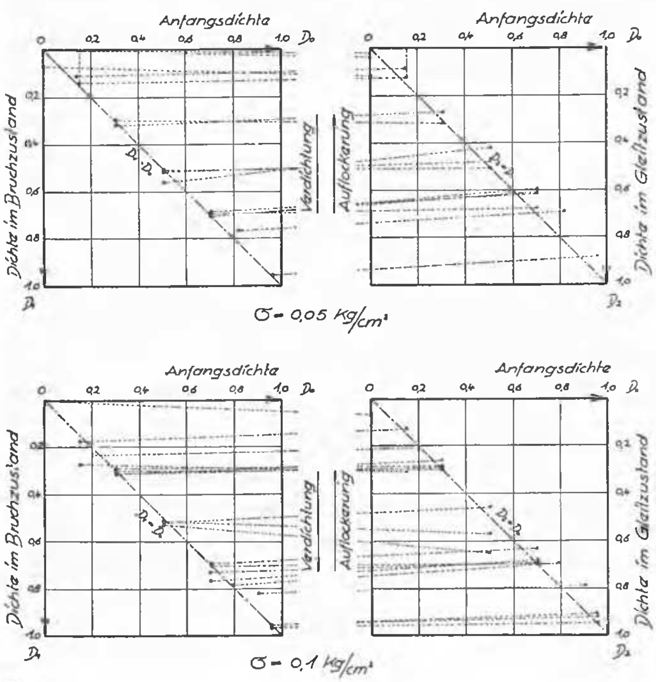

Fig. 11 **Relationship Between Density and Density when Sliding**
Relation entre la densité et la densité au cours du glissement

stress, normal stress and velocity of shearing should be observed and also the effect of vibration.

L'auteur traite de la résistance au cisaillement dans les sables. Le déplacement au cours du cisaillement et les variations de la densité font l'objet d'une étude approfondie.

Mr. Tan Tjong-Kie

It would be interesting to compare *J. A. Jimenez Salas* and *J. M. Serratosa*'s (Proceedings 1953, vol. II, p. 192) hypothesis with my conception of the mechanical behaviour of clays 1): The particles of most clay-minerals are platelike of different sizes and shapes. In 2) 1 layer (montmorillonites, illites) the flat side is charged negatively, whereas the edges may be regarded as being charged positively.

According to the theory of the lyophobic colloids these particles must form a three-dimensional network, with edge to flat side contacts. *Van Olphen* pointed out that the cohesion in the contacts must be attributed to *Coulomb*'s attractive forces and also to *London–Van der Waals* forces. I would like to remark here that the sedimentation volume of a clay suspension is permanently reduced, after being centrifuged. Consequently the clay-micelles must have contacts and therefore a hypothesis based on *Freundlich*'s theory can *only be valid for very dilute suspensions.*

Basing on the above network-conception it is possible to explain the mechanical behaviour of clay (1), (2); the elastic properties must be ascribed to the repulsion occurring between the flat sides of the particles; its retardation is caused by the low permeability of the soil skeleton and the viscosity of the pore water. The viscosity is caused by contacts loosening and reforming at other points. The first yield value can be ascribed to the coagulation of the clay particles between two neighbouring sand particles and to *Coulomb*'s internal friction.

Failure takes place when the equilibrum between breaking and forming of contacts ceases. It would be too lengthy to give more details.

References

Geuze, E. C. W. A., and *Tan Tjong-Kie* (1953): Proc. 2d Int. Congress on Rheology, Oxford.

Tan Tjong-Kie (1954): Investigations on the Rheological Properties of Clays. Forthcoming Doctor's Thesis, Techn. Univ. Delft.

L'auteur compare sa théorie sur le comportement mécanique des argiles à la théorie faisant l'objet de la communication de MM. *J. A. Jimenez Salas* et *J. M. Serratosa* (Comptes Rendus 1953, vol. II, p. 192).

Prof. L. Zeevaert and H. Vogel

The results obtained by Messrs. *Jimenez Salas* and *Serratosa* (Proceedings 1953, vol. I, p. 192) using bentonitic clays mixed with organic liquids are extremely interesting. Specially important to us are the results that show the shape of the consolidation curves.

The straight line relationship in semi-logarithmic paper of the secondary compression has been observed for many other clays and has been classified in soil mechanics as "secondary compression". Fig. 12 shows several consolidation curves made on undisturbed samples.

Strictly speaking, Mexico City volcanic clay may be classified as a clayey silt of very high compressibility with liquid limits

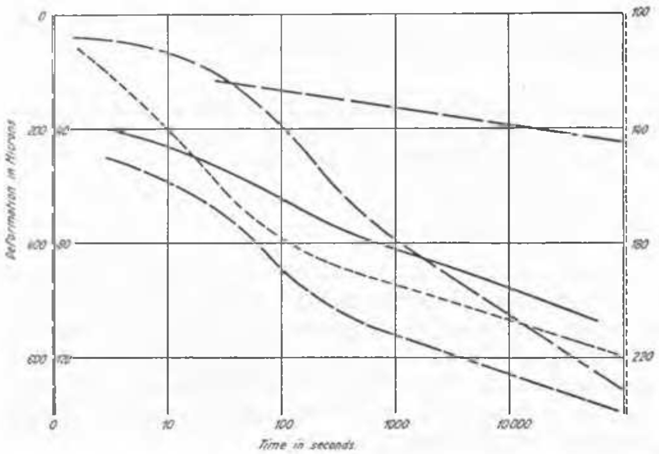

Fig. 12 Consolidation Curves for Undisturbed Clays, Mexico City
Courbes de consolidation d'échantillons d'argile non remaniés, Mexico City

up to 450%. It contains as much as 20% of the mineral montmorillonite; 50% of ashes, diatoms and ostracods, and about 5 to 10% of organic colloids. Because of its very high water content, the material is rather pervious as compared with a compact material that would have the same amount of fine constituents.

Most of the settlement takes place following a logarithmic law. In Mexico City silty clay deposits, the primary consolidation in the field takes place rapidly because of the many pervious layers encountered in the deposit (see Proceedings 1953, vol. II, p. 299), thus leaving us mostly with the secondary time effect.

The secondary compression may be approximately evaluated by:

$$S = S_a + C_t \log \frac{t}{t_a} \quad .. \tag{1}$$

where: S_a is the settlement at time t_a, at which the primary consolidation merges into the logarithmic law. If $\varDelta p$ is the increment of pressure and H is the thickness of the deposit, then:

$$S_2 = \left[\frac{S_a}{H\varDelta p} + \frac{C_t}{H\varDelta p} \log \frac{t}{t_a}\right] H\varDelta p$$

$$S_2 = \left[m_{va} + m_t \log \frac{t}{t_a}\right] H\varDelta p \tag{2}$$

The expression above represents the settlement vs. time after time t_a. Here m_{va} and m_t are compressibility coefficients given in the same units, cm²/kg.

The settlements according to (2) are retarded by the slow process of squeezing the gravitational water out from the material during the primary time effect. This delay may be computed approximately my means of *Terzaghi*'s theory of consolidation $U\% = F(T_v)$, thus during this process the settlement is:

$$S_1 = m_{v1} F(T_v) H\varDelta p \quad .. \tag{3}$$

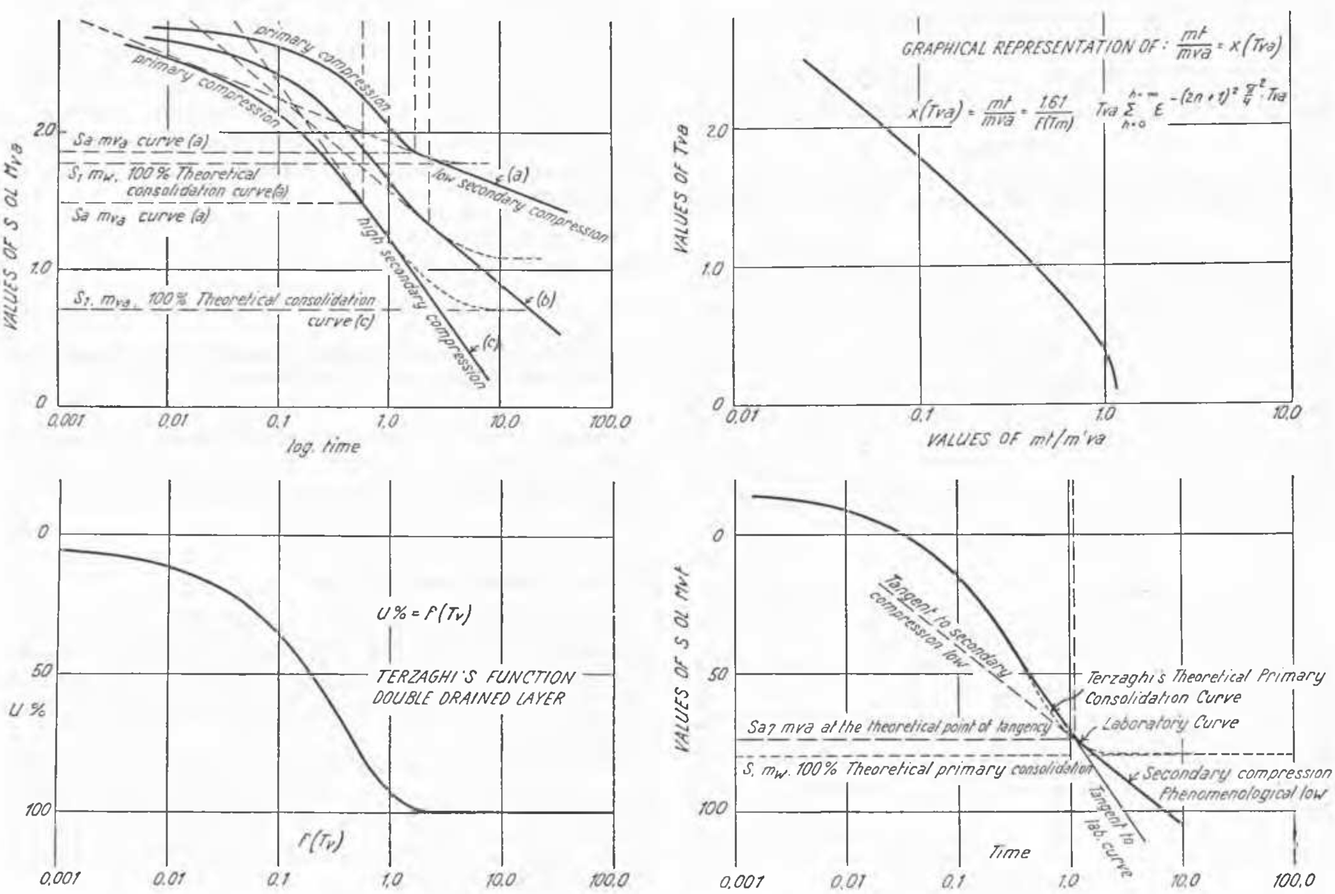

Fig. 13 Graphical Representation of $M_t/M_{va} = x(T_{va})$
Diagramme représentant $M_t/M_{va} = x(T_{va})$

and continues up to the point where

$$\frac{\delta S_1}{\delta_t} = \frac{\delta S_2}{\delta_t} \qquad (4)$$

Therefore, at time $t = t_a$, $U\%$ according to the theory of consolidation is not 100% as generally assumed, since during the secondary time effect, the volume changes require hydrostatic excess pressures in the material to force the gravitational water out to the drainage surfaces.

Condition (4) leads to an auxiliary function Fig. 13:

$$\frac{m_t}{m_{va}} = X(T_{va}) \quad .. \qquad (5)$$

that permits to compute, knowing the ratio m_t/m_{va}, the values of T_{va}, and thus the value of $U\%$ according to *Terzaghi*'s theory of consolidation at which the primary consolidation merges into the secondary. Therefore, the auxiliary function facilitates the computation of theoretical m_{v1} in equation (3):

$$m_{v1} = \frac{m_{va}}{F(T_{va})}$$

corresponding to the virtual 100% theoretical consolidation. Then the coefficient of consolidation may be obtained:

$$c_v = \frac{0.195\,H^2}{t_{50}} \quad \text{for } U\% = 50\%$$

Using this procedure as developed by Dr. *Zeevaert*, the coefficient of consolidation of a clay sample of Mexico City

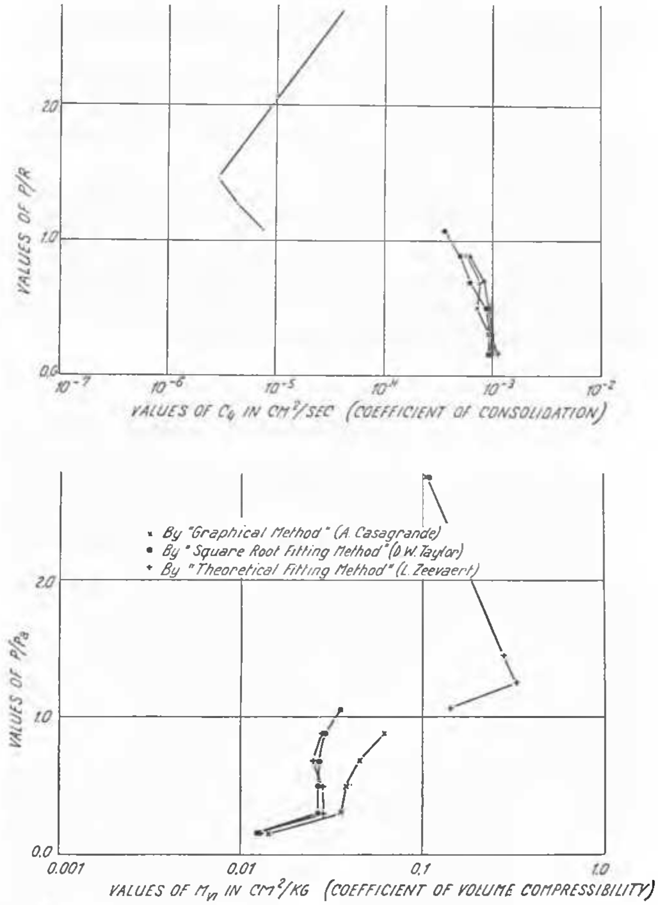

Fig. 14 Coefficient of Consolidation of a Clay Sample Calculated by Different Methods
Coefficient de consolidation d'un échantillon d'argile, calculé d'après différentes méthodes

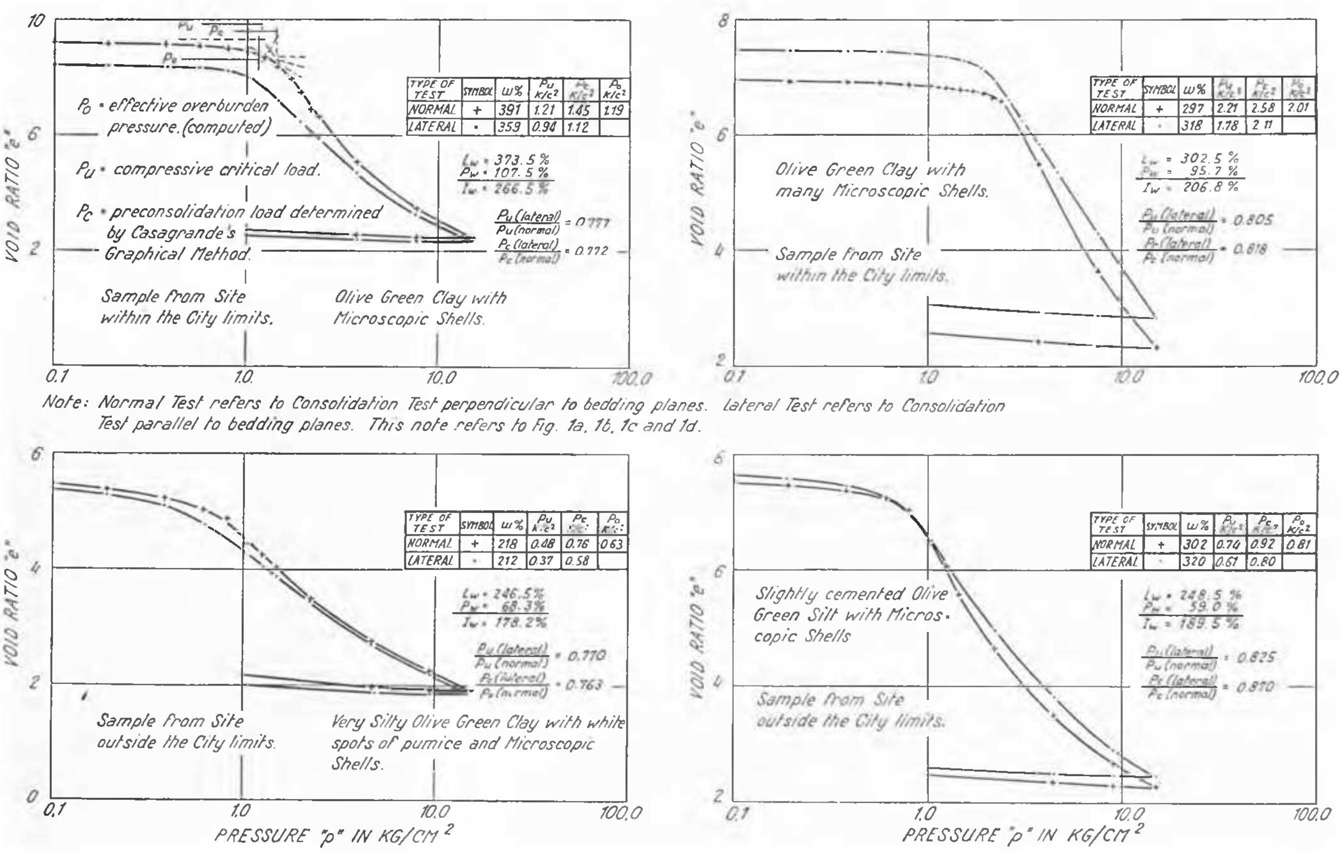

Fig. 15 Change in the Compressibility Coefficients in the Neighbourhood of the Break of the Compressibility Curve
Modification des coefficients de compressibilité aux alentours de la rupture de la courbe de compressibilité

was computed and the results compared with other procedures as shown in Fig. 14.

Consolidation tests were made normal and perpendicular to the sedimentation planes. From the results obtained, it may be noticed that mechanical properties show fairly good isotropy.

It is particularly interesting to notice the large changes in the compressibility coefficients occurring in the neighbourhood of the break of the compressibility curves Fig. 15. Using the fitting method proposed here, the values of the compressibility coefficients can be determined practically for all the ranges of pressures at which the samples were subjected.

Les dépôts argileux hautement compressibles du sous-sol de Mexico City accusent un tassement secondaire élevé qui suit une loi logarithmique. Vu les conditions stratigraphiques le tassement primaire sur place est très rapide. Les argiles bentonites mêlées à des liquides organiques qui ont fait l'objet d'une étude par MM. *Jimenez Salas* et *Serratosa* montrent également un tassement secondaire très nette en accord avec une loi logarithmique. Un procédé inventé par les auteurs, permettant de raccorder les deux courbes de tassement, primaire et secondaire, est discuté et une méthode est proposée pour l'évaluation du coefficient de consolidation dans des argiles à haute compression secondaire. Les figures ci-dessous illustrent la compressibilité de plusieurs argiles bentonites de Mexico City à différents niveaux.