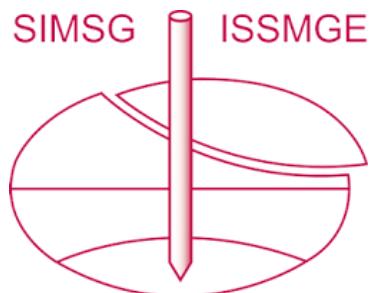


INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND GEOTECHNICAL ENGINEERING



This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

<https://www.issmge.org/publications/online-library>

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.

Theories and Hypotheses of General Character, Soil Properties, Soil Classification, Engineering Geology

Théories et hypothèses de caractère général, propriétés et classification des sols, géologie technique

Chairman / Président: Prof. E. DE BEER, Bruxelles, Belgique

Vice-Chairman / Vice-président: Prof. O. FRÖHLICH, Technische Hochschule, Wien, Austria

General Reporter / Rapporteur général: Prof. A. CASAGRANDE, Harvard University, Cambridge, Mass., U.S.A.

Oral Discussion / Discussion orale:

Dr. L. Bjerrum, Oslo, Norway

Prof. L. Zeevaert, Mexico City, Mexico

Mr. L. Wolpert, Tel Aviv, Israel

Mr. A. B. A. Brink (presented by Mr. G. W. Donaldson),
Pretoria, Union of South Africa

M. U. Nascimento, Lisbonne, Portugal

Prof. M. P. dos Santos, Laurenço Marques, Portuguese East Africa

Prof. A. W. Skempton, London, Great Britain

M. A. Mayer, Paris, France

M. M. Rocha, Lisbonne, Portugal

Written Discussion / Discussion par écrit:

Mr. H. H. von Esbeck-Platen, Aachen, Germany



Prof. A. Casagrande, U.S.A.

General Reporter Session 1 – Rapporteur général Session 1

The Chairman

We open now the discussion on the First Session, theories and hypotheses of general character, soil properties, classifi-

cation and engineering geology. May I ask the General Reporter, Professor Casagrande, for his introduction to this subject.

The General Reporter

Fellow Members of the Third International Conference! After the highly inspiring meeting this morning, it was necessary to come down to earth again, both figuratively and literally speaking. I believe that you will agree with me that after Professor Terzaghi's very instructive talk we can now buckle down to the tasks for which we have gathered here from many parts of the world.

Before I start on the subject of this afternoon's meeting, I should like to say a few words about the way these meetings should be run. I hope I am not too indiscreet by mentioning that just before Professor Terzaghi's lecture the General Reporters met our hard-working Secretary, Dr. von Moos, in order to discuss how these meetings are to be conducted. Perhaps the only point on which we agreed is that I would act as a guinea-pig, and that after this meeting we would meet again and discuss how we could do better in the subsequent Sessions. It was agreed that as much time as possible should be allowed for the discussions and that the Reporter should use only little time for his introduction, a few minutes perhaps, up to a quarter of an hour. I will take only a few minutes.

The scope of this Session covers many divergent topics. When reviewing the papers in my Session, I realized that a re-study of the classification of papers was urgently needed for the purpose of conducting future conferences. Therefore, I have taken the liberty to mention in my report, as the first topic that would deserve discussion, although not in this meeting, a study of the difficulties which have arisen from the lack of a clear definition of the topics belonging to each section. I believe that this is very important because the contributions to certain topics are now divided among two or more Sessions. For example, shear strength is covered in this Session, in tomorrow's Session and in one additional Session. It does not matter whether we discuss this subject in several Sessions, but I believe, from the standpoint of a review of this subject by the General Reporter, he encounters difficulty if he does not know what is contained in the papers on the same subject which have been submitted to other Sessions.

In this Session there are a number of contributions on soil classification and identification. Regional soil studies and engineering geology have been covered in several papers. Consolidation problems have been discussed in two papers. The shear strength of clays is covered in three papers in this Session and in a number of papers in other Sessions. The subject of soil moisture variations in the ground under buildings and under pavements, an important problem in many countries when dealing with fat clays, has been discussed in several papers. Finally, there was a paper on the effects of electro-osmotic flow, a topic which is also covered in papers submitted to other Sessions.

I took the liberty of recommending in my report that the discussions be concentrated firstly on the strength of clays, and secondly—not so important—on soil identification and classification. I believe that the shear strength of clays is the most difficult chapter in soil mechanics. If we note how competent investigators arrive at quite different ideas on this subject, we must realize that there is still a great deal about this subject that we do not know. One difficulty which I found when reading papers on this subject, not only the papers in this Session, but papers published elsewhere or in other Sessions of this conference, is that it is very difficult to understand fully the background of a hypothesis, unless one knows all the details of a test procedure. I realize that this would require very lengthy papers. But without full knowledge of every detail of a test on the strength of clays, it is virtually impossible for me to understand why I obtain results and arrive at conclusions which are different, fundamentally different at times, from those obtained in other laboratories.

Tout d'abord M. Casagrande annonce que les Rapporteurs Généraux ont décidé de consacrer la majeure partie des séances aux discussions et de résumer leurs introductions le plus brièvement possible.

Le sujet de cette Session comprend des sujets différents et le Rapporteur est d'avis qu'une nouvelle classification s'impose (voir Rapport Général, Comptes Rendus 1953, vol. II, p. 307).

De plus le fait que des mémoires se rattachant à un même sujet, par exemple la résistance au cisaillement, se trouvent épars dans plusieurs Sessions empêche le Rapporteur de se former une vue générale sur les travaux présentés.

Le Rapporteur propose de délimiter la discussion à deux sujets: tout d'abord, la résistance au cisaillement qu'il considère comme le point crucial de la mécanique des sols, et ensuite l'identification et la classification des sols, pour autant que le temps le permettra. Le Rapporteur souligne la divergence des résultats auxquels sont arrivés différents auteurs et insiste sur le fait qu'il est impossible de comprendre la base d'une hypothèse sans avoir en mains les détails des essais sur lesquels elle repose. Tout en admettant que ce procédé

requiert des mémoires d'une longueur considérable le Rapporteur estime qu'il n'est pas possible d'arriver autrement à des conclusions probantes et de comprendre la raison pour laquelle les résultats obtenus par différents laboratoires divergent parfois essentiellement.

The Chairman

Au nom de l'assemblée le président remercie M. Casagrande de son rapport approfondi ainsi que du bref résumé qu'il vient d'esquisser. Considérant le bien-fondé des remarques du rapporteur, le président décide de passer immédiatement à la discussion sur la résistance au cisaillement et donne la parole au Dr Bjerrum.

The Chairman thanks Prof. Casagrande for his General Report; further he seconds his motion and trusts that both themes will be treated in the course of the afternoon. Finally Mr. de Beer gives the floor to Mr. Bjerrum.

Dr. L. Bjerrum¹⁾

Professor Casagrande has proposed to concentrate the discussion on the basic elements of the shear strength of clays. This question has been treated thoroughly in papers published by different members of Imperial College in London, and the result of the investigations confirmed in principle the validity of Hvorslev's theories, published in 1937.

At the Laboratory for Hydraulic Research and Soil Mechanics at the Swiss Federal Institute of Technology, we made independently a comprehensive investigation on the shear strength of a number of Swiss clays. The results are surprisingly similar to those reported by Gibson (Proc. 1953, vol. I, p. 126) in a paper to this conference. For instance we took a great number of measurements on the inclinations of the failure planes in compression tests and compared the angle of true internal friction so obtained with the value determined by the method developed by Hvorslev.

For five Swiss clays, ranging from plastic to silty clay, we found good agreement, the deviations being of the order of

¹⁾ Only addresses of authors who did not attend the Conference are included. See addresses of Conference Members page 7

L'adresse ne sera mentionnée que pour les auteurs n'ayant pas pris part au Congrès. Voir adresses des Congressistes page 7

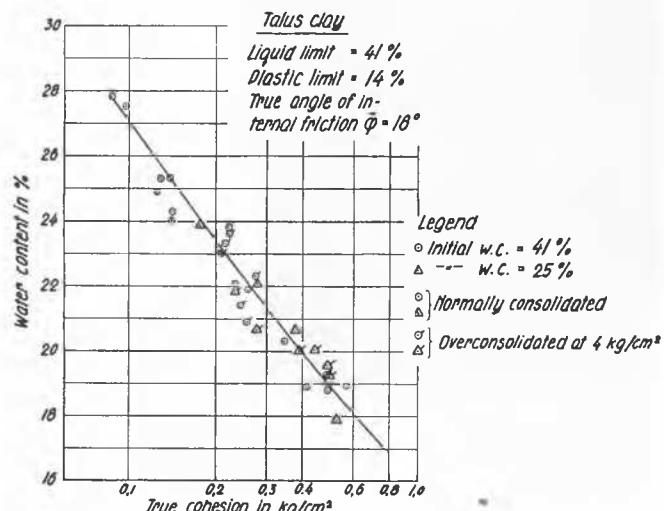


Fig. 1 Relationship Between Water Content and True Cohesion in a Clay
Teneur en eau en fonction de la cohésion vraie

$\pm 1.5^\circ$. Referring to Casagrande's report, I should like to emphasize that all tests were performed with conventional laboratory rates of loading.

Furthermore, we carried out a great number of shear tests with a remoulded clay. In one series of tests, samples were consolidated normally from a slurry. In a second series the clay was consolidated starting with much smaller initial water contents. In a third series of tests we determined the shear strength of the same clay, when first preconsolidated at high pressures, and then afterwards unloaded.

These tests indicated that for the same effective normal stress on the failure plane, the same clay can show extremely different shear strength, dependent e.g. on the initial water content or on the stress history. But, if, from all the different values of the total shear strength, we subtract the true internal friction, we find that the rest, which we call the true cohesion, was proved to be only a function of the water content at failure.

Fig. 1 shows a graph of the true cohesion plotted against failure water content. This clay showed in compression tests an inclination of the failure plane which corresponds to the true angle of internal friction of 18° . Using this value, the frictional resistance was calculated for all drained tests and subtracted from the total shear strength. The cohesion so obtained is then in the diagram plotted against the water content at failure.

The figure shows that all points fall close to a single curve, independent of whether the clay is normally consolidated, shown by round and triangular markings, or whether over-consolidated—indicated by a dash and dot line. Also the cohesion-water content relationship is the same for clay samples consolidated from the liquid limit (41%)—plotted by round markings—or from a smaller water content (25%)—shown by the triangular markings.

We therefore conclude that by the aid of Hvorslev's failure criterion we may account for the difference in shear strength of a clay resulting from different initial conditions or from different stress history. No other theories have succeeded in doing this, and, so far, we consider Hvorslev's failure criterion as the most satisfactory fundamental basis for shear strength research available at the present stage of development.

Les résultats des recherches entreprises aux laboratoires de l'E.P.F. à Zurich sur la résistance au cisaillement de différentes argiles suisses ont donné des résultats similaires à ceux auxquels est arrivé Gibson (Comptes Rendus 1953, vol. I, p. 126).

Des essais sur des échantillons d'argile non remaniés ont montré que pour la même tension efficace normale agissant sur le plan de rupture, la même argile offre une résistance au cisaillement variant avec la teneur en eau initiale et l'historique des contraintes. Cependant lorsque l'on soustrait le frottement interne vrai des valeurs de la résistance au cisaillement totale obtenue dans les divers essais, on constate que le reste, c'est-à-dire la cohésion vraie, est uniquement une fonction de la teneur en eau au moment de la rupture. Pour plus amples détails voir Fig. 1.

L'auteur conclut que les résultats obtenus démontrent la validité de la théorie de Hvorslev.

Prof. L. Zeevaert

The ratio of the horizontal to the vertical pressure of natural unconsolidated sedimentary deposits is an important point for discussion, and usually leads to controversy among soil mechanics engineers.

Messrs. Habib and Puyo (Proc. 1953, vol. I, p. 32) have performed a series of laterally-confined compression tests in the attempt to determine the ratio of the horizontal to the ver-

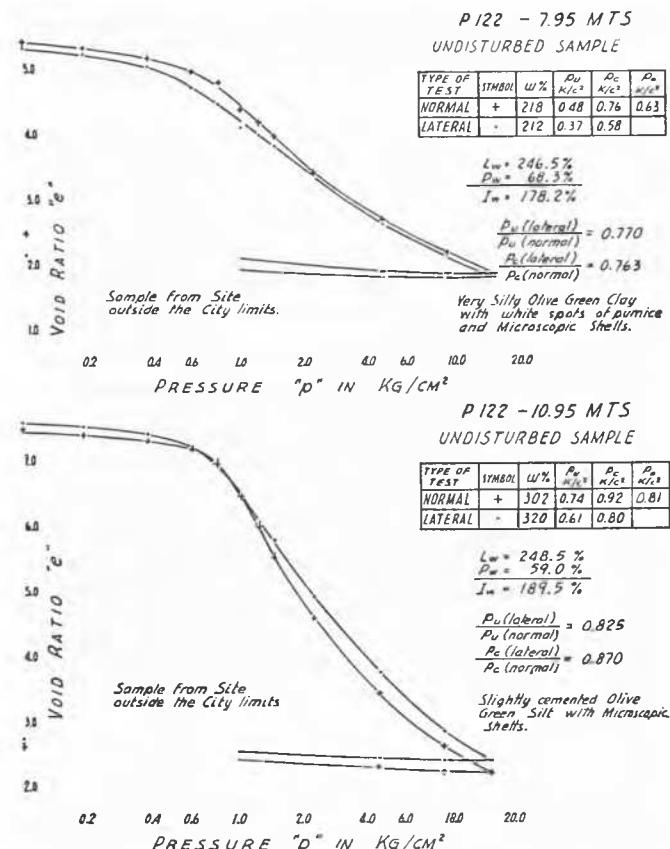


Fig. 2 Consolidation Curves
Courbes de consolidation

tical pressure after the consolidation processes. They compared the standard laterally-confined compression test with a series of compression tests made applying for each of them different lateral horizontal pressures. They reached the conclusion that the compression test with equal horizontal and vertical pressures, showed a compressibility curve very close in shape with that obtained by means of the standard confined compression test, with zero lateral deformation.

It appears that some ideas along this line may be obtained by comparing the vertical and horizontal preconsolidation loads of normally loaded soil deposits, thus avoiding the effect of friction forces that develop during the consolidation tests. Fig. 2 shows standard laterally-confined compression tests made normal and parallel to the sedimentation planes with exactly the same sample. The samples used were undisturbed clayey silt samples from Mexico City. The preconsolidation load was determined using Casagrande's method. The results shown in Fig. 2 yielded values, for the ratio of the horizontal to the vertical preconsolidation loads, of 0.763 for the sample at 7.95 m depth, and 0.870 for the sample at 10.95 m depth, both of the same borehole.

Messrs. Habib and Puyo reached the conclusion that in their tests the ratio of horizontal to vertical pressures after consolidation should be unity. Other investigators have reached the conclusion that this ratio should be close to $\frac{1}{2}$. It is the writer's belief that the ratio of vertical stress to horizontal stress in natural deposits is mainly dependent on the natural process of sedimentation and stratigraphical details of the entire deposit.

Therefore, in order to investigate the ratio of the horizontal to vertical pressures in natural clay deposits, the investigation

should contain a study of the stratigraphy of the deposits and of the surroundings in which it was formed, so as to ensure a proper interpretation of the results of the tests performed in the laboratory.

It is extremely important to investigate the above-mentioned materials that may be geologically considered preconsolidated, either due to eroded material that caused in the past higher overburden pressures, or because of desiccation. Materials that in spite of normal loading have developed high bonds in their structure because of active mineral properties, which give them the appearance of a material of the preconsolidated type, must also be considered.

Acknowledgement is due to Mr. H. Vogel for his collaboration in the performance of the tests shown in the figures.

L'auteur traite du rapport entre les pressions verticales et horizontales agissant sur un élément d'un sédiment naturel intact. Il a trouvé des valeurs différentes pour ce rapport en examinant deux échantillons du sous-sol de la ville de Mexico prélevés à 7,95 et 10,95 m de profondeur. Les échantillons ont été examinés parallèlement et perpendiculairement à la stratification et indiquent une proportion de préconsolidation variant de 0,763 à 0,870 (voir Fig. 2). Ces résultats diffèrent de ceux obtenus en laboratoire par MM. Habib et Puyo et d'autres auteurs. Ce fait montre que, pour établir ce rapport, il est nécessaire, en cas d'échantillons non remaniés, de mettre en évidence l'histoire géologique du sol en question.

Mr. L. Wolpert

I would like to make a few remarks on the paper by Messrs. Croney and Coleman (Proc. 1953, vol. I, p. 13), on soil moisture suction properties and their bearing on the moisture distribution in a soil. Croney and Coleman have suggested an equation: $aP - s = u$ to describe the equilibrium moisture condition at a point in an unsaturated soil. In this equation P is the overburden, u is the pore water pressure determined by the position of the water table and s the stress free suction. The authors also suggest that this equation describes the stress equilibrium in the pore water, and that the pore water carries a fraction a of the overburden. But there is at present, however, no experimental evidence to show that the pore water in an unsaturated soil actually carries any of the overburden. All that is measured is the change in the negative pore water pressure, which may be due to the change in pore/space relationship. If the equation is regarded as describing the stresses acting in the soil water, as suggested by the authors, one arrives at a conclusion which is in contradiction with the theory of consolidation, namely that the pore water in a saturated consolidated soil at equilibrium carries all the overburden. These considerations do not invalidate the equation by any means, but merely suggest perhaps that the equation is better regarded simply as an algebraic relation between the three relevant quantities, P , u and s , which a having such a value so as to make the equation hold. aP may then be regarded as the change in pore water pressure due to the application of the overburden P which brings about a change in the pore space relationships.

L'auteur critique la formule de MM. Croney et Coleman (Comptes Rendus 1953, vol. I, p. 14), pour autant que cette formule est considérée comme démontrant l'équilibre des forces dans l'eau interstitielle. Jusqu'à présent nous n'avons pas de moyen expérimental pour mesurer et prouver que la pression de l'eau interstitielle dans un sol non saturé porte une partie de la surcharge. S'il en était ainsi, la formule serait en contradiction avec la théorie de la consolidation car dans un sol saturé l'eau interstitielle porterait toute la surcharge. L'auteur est d'avis que cette formule démontre seulement les relations algébriques entre les différentes quantités.

Mr. A. B. A. Brink

Observations of the effect of a building on the moisture content of the soil, similar to those discussed by L. A. Du Bois (Proc. 1953, vol. I, p. 8), have recently been made in the expansive soil area of Vereeniging, South Africa.

Before the construction of the experimental building described by Jennings (Proc. 1953, vol. I, p. 390), natural moisture content profiles were taken on the site, during June and July, 1950, from a number of holes which were drilled for the installation of the thermocouples and nylon moisture units. Three of these profiles are reproduced in the accompanying Fig. 3, boreholes 1, 2 and 3. The building was erected a few months later, during March 1951. Three years later, during July 1953, three more auger holes were put down from inside the building, adjacent to the earlier three holes as shown in Fig. 1. Moisture content profiles were again observed and these are plotted for comparison in Fig. 3 (boreholes 1a, 2a and 3a) next to the original profiles.

Although Wolpert¹ has found that variations in the natural moisture content, as measured by direct tests, are of the order of $\pm 2\%$ at any particular point, a significant increase in the moisture content is seen to have taken place since the construction of the building. The greatest increase appears beneath the centre of the building, where a moisture content increase of approximately 5% is seen to have taken place throughout the profile generally.

This series of observations confirms the observations from the nylon moisture meters described by Jennings (Proc. 1953, vol. I, p. 390), and establishes beyond doubt that the heaving which has taken place is due to an increase in moisture content in the foundation soil. The increase is considerably greater than that found by Du Bois (Proc. 1953, vol. I, p. 8) and this may be due to an originally drier profile, which may be characteristic of South African soils.

¹⁾ Wolpert, L. (1952): "The Direct Measurement of the Natural Moisture Content of Unsaturated Soils." National Building Research Institute, Bulletin No. 9, December 1952.

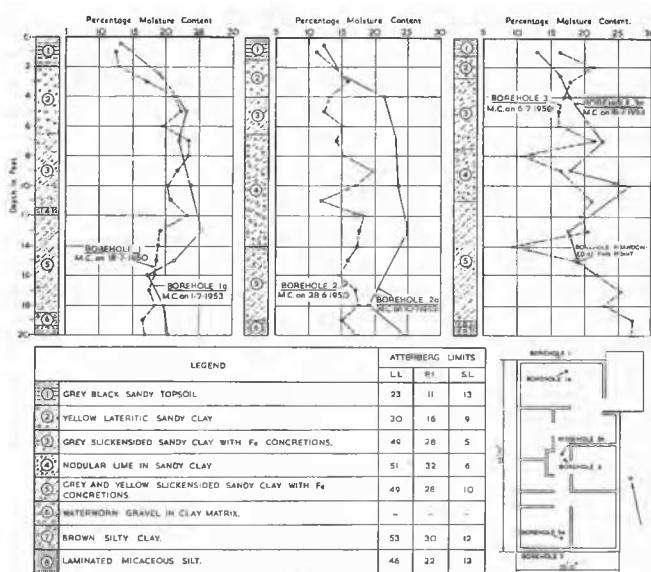


Fig. 3 Comparison of Moisture Content Profiles Taken Before Erection of an Experimental Building on Expansive Soils, and Three Years After Erection

Comparaison de la teneur en eau dans le sous-sol d'un bâtiment d'essai avant et trois ans après la construction

L'auteur décrit des recherches exécutées sur un sol expansif de l'Afrique du Sud; ces essais se rapprochent de ceux de M. L. A. Du Bose (Comptes Rendus 1953, vol. I, p. 8).

L'augmentation de la teneur en eau dans le sous-sol d'un bâtiment d'essai est illustrée par la Fig. 3. Cette augmentation, de l'ordre de 5%, a été observée dès le début de la construction; elle est particulièrement accentuée au-dessous du centre du bâtiment et a causé un soulèvement. Les recherches sus-mentionnées confirment les observations décrites par M. J. E. Jennings (Comptes Rendus, vol. I, p. 390).

Mr. U. Nascimento

Notre contribution se rapporte à l'article de MM. Croney et Coleman (Proc. 1953, vol. I, p. 13).

Je tiens d'abord à exprimer mon admiration pour l'œuvre entreprise par le «Road Research Laboratory» dans l'étude du comportement de l'eau dans les sols, tout particulièrement les travaux de Croney, Coleman et Bridge.

Les commentaires que je désire faire sur cette question se rapportent à des idées qui se trouvent déjà exposées d'une façon détaillée dans un article intitulé: «Capillarity and Soil Cohesion» qui n'a pu être publié dans les Comptes Rendus du congrès, vu qu'il a été envoyé trop tard. Un certain nombre d'exemples ont été distribués aux congressistes au commencement des sessions.

Dans cet article l'équation de Terzaghi est généralisée par l'introduction de la succion capillaire ω :

$$\sigma = \bar{\sigma} + u - \omega \quad (1)$$

d'où

$$(\sigma - \bar{\sigma}) + \omega = u \quad (2)$$

si l'on pose

$$\sigma - \bar{\sigma} = \alpha \sigma \quad (3)$$

on obtient

$$\alpha = \frac{\sigma - \bar{\sigma}}{\sigma} = \left(1 - \frac{\bar{\sigma}}{\sigma}\right) \dots \quad (4)$$

dans laquelle α représente le coefficient de MM. Croney et Coleman. Dans notre article nous revisons la notion de tension superficielle, tout en montrant que celle-ci est considérée dans la physique, depuis un siècle environ, comme agissant dans la direction de la normale à l'interface entre les phases et non plus comme une force agissant suivant la tangente à l'interface, ainsi qu'on l'admet encore couramment en mécanique des sols. Tout en prenant comme base la première conception de tension superficielle, on fait l'analyse du phénomène de la succion capillaire en établissant son rapport avec l'épaisseur de la pellicule de contact des phases solide, liquide et gazeuse, et aussi avec la porosité du sol.

L'hydrostatique des eaux capillaires est établie en considérant l'action simultanée du poids gravitaire et celle du poids nommé capillaire dû à la succion capillaire.

L'équation (1) est employée comme point de départ pour une analyse du phénomène de la cohésion et de la réversibilité des déformations dues à la consolidation. De cette analyse on conclut que la résistance à la traction, vu la cohésion physique, est égale à la succion capillaire, et que la réversibilité des déformations dues à la consolidation est une conséquence de cette succion. On montre qu'un drain placé dans un terrain homogène au-dessus de la nappe phréatique ne recueille pas les eaux pluviales, ce que nous avons d'ailleurs vérifié au Laboratoire Nacional de Engenharia Civil à Lisbonne.

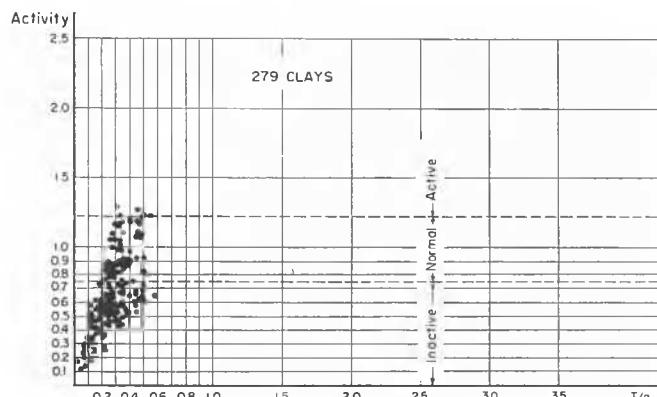


Fig. 4 Relationship Between Skempton's Activity of Clays and dos Santos' Constant "a"

Rapport entre l'activité des argiles d'après Skempton et la constante «a» introduite par dos Santos

On conclut que seule la phase solide d'un sol subit la poussée hydrostatique d'Archimète si la succion capillaire est constante en profondeur.

The author refers to his work on "Capillarity and Soil Cohesion", in which the notion of suction, ω , is generalized and applied to Terzaghi's equation: $\sigma = \sigma + \mu - \omega$.

Prof. P. P. dos Santos

I should like to make some brief comments on papers by P. P. dos Santos (Proceedings 1953, vol. I, p. 47) and by A. W. Skempton (Proceedings 1953, vol. I, p. 57). It appears that there is a possible relationship between Professor Skempton's activity of clays, defined as the quotient

Plasticity Index

Percentage minus 2 microns

and the quotient

Plasticity Index

a

a being the constant introduced by myself in paper 1/11, related to the sand fraction of the grain size distribution curve. Data have been collected for 279 clays of various origins in Portuguese East Africa. They are shown in Fig. 4.

It may be remarked that there are not yet sufficient data available in the case of active clays.

L'auteur désire attirer l'attention sur la possibilité d'une relation entre l'activité des argiles d'après A. W. Skempton et la constante «a» faisant l'objet de son mémoire 1/11 (voir Fig. 4). (Comptes Rendus 1953, vol. I, p. 47.)

Prof. A. W. Skempton

I would like to mention very briefly two points that are concerned with shear strength of clays. The first of these points is largely a matter of emphasising how much I agree with what Dr. Bjerrum said in his discussion a little earlier. He mentioned that, at Imperial College, Dr. Gibson has been doing work on shear strength to investigate, by means of tests on a number of clays, covering quite a wide range of type, the criterion of failure as proposed in 1937 by Dr. Hvorslev. In all, we have so far worked on 8 or 9 different clays, and with these we find, without exception, that the angle of true internal friction (by which I do not mean the slope of the Mohr envelope in undrained or slow tests, but the true angle as defined in Hvorslev's

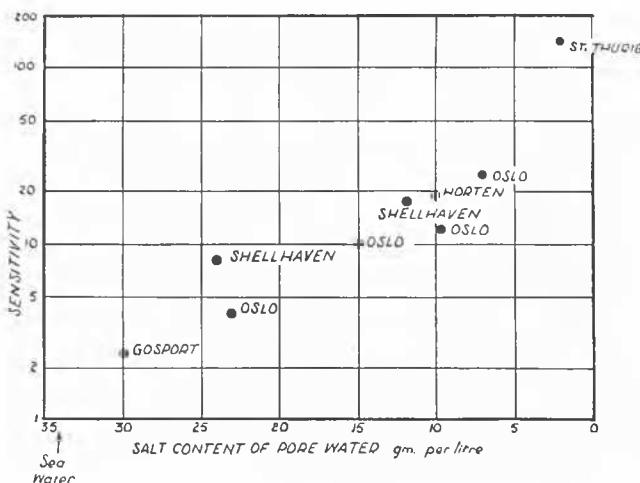


Fig. 5 Relation Between Sensitivity and Salt Concentration in the Pore Water of Marine Clays
Rapport entre la sensibilité et la teneur en sel de l'eau interstitielle des argiles marines

equation) agrees quite closely with what one would deduce from the inclination of the slip planes in the compression specimens. These angles vary from a value of 2° or 3° for Bentonite, up to angles of about 25° or so for silty clays. And in the whole of that range we get, as I mentioned, quite good agreement between the angle, as measured by a technique first suggested by *Hvorslev*, which involves, as you know, measuring shear strength of the clay at the same water content, but under two very different effective pressures, and the angle of the slip plane in a compression specimen.

I am aware that, in his general report, Professor *Casagrande* has mentioned that the speed of loading appears to play an important part in altering the inclination of the slip planes in a compression specimen of clay. This phenomenon we have not investigated, our total range of rate of loading being not more than the ratio of about 1:100. This influence of the extremely rapid rate of loading on the inclination of the shear slip planes is a very interesting point, and it seemed to me to suggest some interesting possibilities with regard to the structure of clays, but I do not wish to touch on this now.

Another point arising from these tests in which we are investigating *Hvorslev's* criterion failure was that equally, of course, we could get the true cohesion, again as defined by *Hvorslev*, and for these various clays we found that the proportion of the shear strength which could be attributed to true cohesion, was fairly closely correlated with the mineralogy of the clay. This proportion of the shear strength, contributed by true cohesion, varied in the clays we tested from about 10 per cent for Kaolin up to about 80 per cent for Bentonite, and we feel bound to conclude, when we consider our tests together with those carried out here in Zurich, by Dr. *Bjerrum*, a year or two ago, that certainly, as a first approximation and for normal rates of testing, *Hvorslev's* criterion has a very great deal to recommend it.

The second point on which I want to comment concerns the sensitivity of clays. You probably know that in 1946 Dr. *Rosenquist* in Norway made the suggestion, so far as I know for the first time, that the sensitivity of clays was due to a leaching out of some of the salt from the pore water in marine clays. Independently, at Imperial College, Dr. *Northey* was reaching the same conclusion, while doing some experiments which had been initiated with a view to investigating thixotropy. We,

through the courtesy of Mr. *Brinch Hansen* of Denmark, got to know of this work (which is written in a Scandinavian language which I am afraid we cannot read) and we then extended our experiments. These led to two conclusions. The first, which is very tentative and no more than a hypothesis, was that the particles in clay are in contact. I do not mean to say that there is a solid, mechanical contact. There may well be, say, four or five water molecules between the particles, but these few water molecules are held on very rigidly, and so we can say that there is effective contact. The experiment which led us to that conclusion was as follows: we had a marine clay which we had brought to equilibrium under load in a special form of oedometer and we then leached this sample. On leaching out part of the salt from the pore water, we observed practically no change in volume (that is to say in water content or packing of the particles) nor any appreciable change in the undisturbed strength of the material. We noticed, however, a very large drop in the remoulded strength. Now, the obvious conclusion, to my mind, from that experiment is that the particles must have been effectively in contact, because if they were not, how is it possible that, by changing the concentration of salt in the pore water by a very appreciable amount we got no change in the volume of the clay, nor any change in the undisturbed strength. The change in remoulded strength is quite easy to understand, and in a paper already published Dr. *Northey* and I touch on that problem.

The other interesting thing which emerged from this work on sensitivity was that having in this way produced artificially in our laboratory, a sensitive clay, we then naturally looked for field evidence. At the time our paper was published, two years ago, we had information on only four clays. They showed that, the more a marine clay was leached, the more sensitive it was; but since then data on five more clays have become available, and in Fig. 5 I show the relation between sensitivity and salt concentration in the pore water for all nine marine clays.

Des essais sur la résistance au cisaillement faits par le Dr *Gibson* à l'Imperial College London sur 8-9 argiles ont montré que l'angle vrai de frottement interne était en étroite relation avec l'inclinaison des plans de glissement. Les angles varient de 2° à 3° pour la bentonite, et mesurent jusqu'à 25° pour les argiles limoneuses, corroborant ainsi la théorie de *Hvorslev*.

L'influence de la vitesse de chargement sur l'angle des plans de rupture dans l'essais de compression n'a pas été étudiée.

Les essais sus-mentionnés ont également montré que la proportion de la résistance au cisaillement attribuable à la cohésion vraie était en relation avec la minéralogie de l'argile. Dans le cas des argiles indiquées plus haut, la résistance due à la cohésion était d'approximativement 10% pour le kaolin et s'élevait jusqu'à 80% pour la bentonite. En conséquence, pour une première approximation et des vitesses de chargement courantes, le critère de *Hvorslev* est recommandable.

Des essais sur la sensibilité des argiles entrepris à l'Imperial College London ont abouti à deux conclusions.

Premièrement à l'hypothèse que les particules d'argile sont en contact par l'intermédiaire des molécules rigides de l'eau. Cette hypothèse est fondée sur les essais suivants: après avoir délavé une partie du sel de l'eau interstitielle contenue dans une argile marine amenée à équilibre par chargement dans un oedomètre de forme spéciale on n'a observé ni modification de volume ni modification appréciable de la résistance à l'état non remanié du matériel, mais, par contre, une diminution importante de la résistance à l'état remanié. D'où nous avons conclu que les particules doivent être en contact.

Deuxièmement: nous avons cherché des exemples parmi les argiles existantes. Pour plus amples détails sur le rapport entre la sensibilité de différents argiles et la teneur en sel de l'eau interstitielle voir Fig. 5.

M. A. Mayer

La présence de M. *Habib* à la table des interprètes me fait intervenir pour discuter ou plutôt rectifier un point du rapport général de M. *Casagrande* qui concerne la communication de M. *Habib* (*Comptes Rendus* 1953, vol. I, p. 28). Le professeur *Casagrande* a indiqué qu'il s'agissait d'essais de compression simple. En réalité il s'agit d'une série extrêmement importante d'essais triaxiaux dont la communication publiée ne représente qu'une toute petite partie; c'est la thèse de doctorat de M. *Habib* qui a été publiée dans les Annales de l'Institut Technique et, si M. *Casagrande* n'a pas eu l'occasion de la connaître jusqu'ici, j'espère qu'il voudra bien, à l'occasion de ce congrès, en prendre connaissance. Dans cette thèse M. *Habib* a examiné à l'appareil de cisaillement triaxial la résistance au cisaillement des sables, des limons et des argiles au moyen d'essais à la compression, d'essais à la traction, qui n'avaient jusqu'ici, je crois, pas été réalisées dans les sables, et d'essais de torsion. De cette manière, il a pu établir l'influence dans les sables de la contrainte intermédiaire sur la résistance au cisaillement. M. *Habib* n'a trouvé qu'une faible influence de la contrainte intermédiaire sur la résistance au cisaillement des argiles. J'aime mieux, en ce qui me concerne, employer le terme de «résistance au cisaillement» que de parler «d'angle de frottement et de cohésion», qui sont des termes mal définis, lorsque les courbes de résistances au cisaillement ne sont pas des droites. Dans cette étude, M. *Habib* a également examiné l'influence sur la résistance au cisaillement des argiles de l'histoire des matériaux, de leur consolidation préalable, l'influence des conditions d'application des efforts selon qu'elles sont lentes ou rapides, l'influence enfin du liquide interstiel, selon que le liquide est de l'eau ou un liquide organique dont le coefficient de mouillage des matériaux solides est différent de celui de l'eau. On obtient une différence essentielle, en ce sens que, lorsqu'on remplace l'eau par du benzène par exemple, la perméabilité du matériau augmente considérablement et les vitesses de consolidation varient de ce fait. Une argile à l'état pulvérulent dans laquelle l'eau a été remplacée par un liquide qui ne mouille pas les grains, se comporte comme un sable. Lorsque, au contraire, on introduit un liquide qui mouille les grains, on voit tomber progressivement l'angle de frottement et apparaître la cohésion. L'angle de cisaillement tend vers zéro, valeur à laquelle on arrive toujours dans le cas du cisaillement très rapide. Voilà toutes les observations que je voulais vous présenter aujourd'hui.

Mr. *Mayer* points out that the paper by Mr. *Habib* (*Proc. 1953*, vol. I, p. 28) is a part of his doctor thesis in which he studies the behaviour of sands, silts and clays by means of triaxial tests, compression tests, tensile strength tests, torsion tests, etc. Mr. *Habib* has found that the intermediary stress (contrainte intermédiaire) has only a little influence on the shear strength of clays.

Other experiments with various liquids have shown that clays behave like sand if the liquid does not wet the grains, whereas the shear strength diminishes when a wetting agent is used.

M. M. Rocha

Nous avons préparé une communication dans laquelle nous avons établi les conditions de similitude mécanique qui doivent être remplies dans l'étude des problèmes de mécanique du sol sur modèle. Malheureusement, elle n'est pas arrivée au Secrétariat dans le délai convenable pour être publiée dans les *Comptes Rendus* du Congrès. Cependant, le Secrétariat a bien voulu faire distribuer un certain nombre d'exemplaires de cette communication, et j'en ai encore quelques-uns que je remettrai

volontiers aux personnes qu'intéresserait le travail dont je vais vous présenter un très bref sommaire.

Après avoir mis en lumière l'insuffisance des méthodes analytiques de la mécanique du sol, on présente les conditions de similitude auxquelles doivent répondre les modèles destinés à l'étude des problèmes que pose à l'ingénieur le comportement des massifs.

On déduit, en premier lieu, les conditions générales relatives à des prototypes, constitués par des sols saturés ou non saturés, soumis à l'action de forces de surface et du poids propre. Puis on traite de quelques cas particuliers qui sont importants dans la pratique, tels que les équilibres pour lesquels on néglige la circulation de l'eau qui remplit les pores, le cas où l'on admet que l'état final du massif, en particulier la rupture, ne dépend pas des états intermédiaires, etc. On signale en particulier les cas où les matériaux du prototype peuvent être employés dans la construction des modèles.

Puis on donne des indications générales concernant la marche à suivre dans la recherche des matériaux répondant aux conditions de similitude. On souligne que, d'une façon générale, il n'y a pas lieu d'imposer des exigences rigoureuses en ce qui concerne la vérification des lois de similitude établies.

On signale, en conclusion, les services que peuvent rendre les modèles dans l'étude des problèmes posés par les massifs à l'ingénieur.

Je dois souligner que nous avons déjà entrepris quelques études sur modèles qui nous ont donné entière satisfaction. Ces études sont décrites dans une autre communication, préparée en collaboration avec M. *Folque*, qui n'est pas non plus arrivée à Zurich en temps utile.

I have written a paper in which are derived the conditions of mechanical similarity to be fulfilled in model studies of problems in soil mechanics. Unfortunately, it could not reach the Secretariat in time, so that it has been impossible to include it in the Proceedings. However, the Secretariat has been kind enough to have some copies distributed to members of the Congress. A few copies are still available which I shall be pleased to forward to those who might be interested in this contribution, of which I am going to present a very short abstract.

After the limitations of the analytical methods in Soil Mechanics have been emphasized, similarity conditions are presented which should be fulfilled by models to be used in studying the engineering problems of soil masses.

The general conditions are first presented which should be obeyed by prototypes, consisting of saturated or unsaturated soils and acted upon by surface forces and dead weight. Then some particular cases of practical significance are considered, such as that of equilibria in which the seepage of pore water is neglected, that in which the final state of the mass, more especially the failure, is assumed not to depend on the intermediate states, etc. Cases are given in which the materials of the prototype can be used for the construction of the models.

Some general indications are given on the path to be followed in finding materials that fulfil the requirements for similarity. Emphasis is laid on the fact that, as a rule, it is not necessary to be too exacting in the fulfilment of those requirements.

Finally, attention is drawn to the valuable aid models can give in studying the problems of soil masses.

I should emphasize that we have already carried out some model studies whose results were entirely satisfactory. These studies are reported in another contribution, prepared with the collaboration of Mr. *Folque*, which, like mine, was not received by the Secretariat in time.

Mr. H. H. von Esbeck-Platen

Since a subsoil map is in preparation for the purpose of rebuilding the town of Aachen (Aix-la-Chapelle) I should like

to make a few remarks on this subject considering particularly the future development of the map. *W. Dienemann* does not discuss the application of soil mechanics to subsoil maps. A number of soil mechanics reports have been prepared for engineering projects in this town, and the data obtained have proved most valuable for the preparation of the map. Due to rapid changes in the strata the data obtained from "point tests" are valid only for a very restricted area. Tests carried out in bore holes even at intervals of only a few feet showed rather varying physical characteristics and this would induce us to believe that varying settlements would occur. In reality a local change in the stratification had taken place due to a minor fault. Taking the average data obtained in laboratory tests the settlement of the project site will be uniform.

As *Terzaghi* has mentioned in his introduction, observations will be of the greatest importance for the future development of geotechnique. In this respect attention may be drawn to the installation of levelled control points in the foundations of buildings so as to permit the observation of settlements, and to compare them with laboratory tests. The costs of recording such measurements is very low and the value of the observed data very important.

In order to obtain data for subsoil maps it is suggested that all offices engaged in town planning and other similar organisations should keep records of foundation work in their districts. The following points should be taken into consideration for such records:

- (1) Name of firm and datum of observation.

- (2) Location: street, house number, site sketch, etc.
- (3) Geological description: petrographic and structural situation, etc.
- (4) Groundwater: actual groundwater level and noticeable changes (to distinguish from surface water).
- (5) Drilling record (cores should be examined by a geotechnician).
- (6) Observations of the excavation: angle of slope, slipping, quicksand, swelling, drycracks, etc.
- (7) Field and laboratory tests: recorded data and serial number of the report.
- (8) Observations during and after construction: settlement, tilting, heaving, etc.

Although a subsoil map may already exist, a good subsoil map will never be definitive and additional changes and amendments should be made to tally with the new data observed. This will provide the civil engineer with the basic map he requires.

Sommaire

L'auteur, qui prépare une carte géotechnique d'Aix-la-Chapelle, recommande de coordonner les résultats de laboratoire et les aspects géotechniques, particulièrement dans les emplacements d'une tectonique compliquée. Il propose l'installation de points de repère dans les bâtiments nouveaux, ce qui permettrait de comparer les résultats œdométriques et le tassement réel. Finalement l'auteur dresse une liste des points à mentionner dans une description géotechnique aux fins de rassembler le matériel le plus étendu possible.